



“ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
School of Civil and Environmental Engineering
Post Graduate Program Structural Engineering Stream”
Seismic Evaluation of Cost-Effective Houses around Addis Ababa Due to
Current Earthquakes in Awash Region
(Case Study of Condominium Building)

By

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Advisor Dr. Edom Adane

Addis Ababa / September, 2025



**“ADDIS ABABA UNIVERSITY
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“A project Submitted to the School of Graduate Studies of Addis Ababa
University in Partial Fulfillment of the Requirements for the Degree of Master of
Engineering in Civil Engineering”

(Structural Engineering Stream)

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Declaration

I so certify that all of the data in this study was gathered and presented in compliance with ethical standards and academic regulation. I further declare that I have properly cited referenced all information and findings that are not original to this study as required by these rules and conduct.

Name: -Suleyman Mohammed

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Place: Addis Ababa University, September, 2025

ACKNOWLEDGEMENTS

First and foremost, I want to express my gratitude to Almighty God, who has given me the resolve to overcome every obstacle I have faced during my research studies, enabled me to navigate every hallway with courage and zeal, and ultimately enabled me to develop an effective completion of this study.

I want to thank my advisor Dr. Edom Adane for having faith in me and for his priceless guidance during this project. She provided helpful criticism if the study for which I am very grateful.

Lastly, I want to sincerely Thak all of the instructors, staff and friends at AAiT for their encouragement and support.

Abstract

Ethiopia, particularly the Afar region, has experienced a significant increase in seismic activity, including frequent tremors in the Awash region. This heightened seismic risk necessitates a comprehensive evaluation of seismic performance of buildings, especially low-cost housing structures. Seismic evaluation of constructed buildings is mandatory especially in Ethiopia region where the great east African rift valley crosses our country Ethiopia. Studies shows or predicts that the country Ethiopia inclusive of Africa will bisect into two different continents. Recently starting from September to end of October 2024 unusual weekly repeating earthquakes making its initial (epicenter) from awash region is occurring and the ground shake vibration is felt in 10km radius including in Addis Ababa.

The performance of existing building in seismic evaluation area mainly affected by ground excitation. Ground acceleration is the principal cause of collapse in reinforced concrete structures developed in seismically active zone. This might be owing to either the unanticipated nature of the seismic excitation or due to lack of suitable structural design, detailing and construction. It is mandatory to accurately assess the response of the structure during seismic excitation. Codes are unable to foresee the real-time reaction of structures when it comes to response determination. Therefore, the existing structural performance assessment should be done using a reliable and easily applicable analysis procedure.

The revised Ethiopian building code shows a major change on Ethiopia's hazard map in terms of peak ground acceleration. The expected peak ground acceleration (from 0.05g to 0.1g) for Addis Ababa [1]. And also incorporate two types of spectra that depends the surface wave magnitude (Type 1 and Type 2) that happen recently earth quick fluctuate in those two ranges. Many buildings have been designed and build in accordance with the previous building code for the past few years. This change has the significant impact on these structures' performance. Therefor it is necessary to evaluate the effectiveness of the current structures using the revised code's provisions. Because condominiums are built on a large scale across the nation and are places where people congregate, this study focus on them. Premature failures are not anticipated to occur, according to the study's premise.

The main objective of the study is to assess the seismic performance of residential condominium buildings that have been built in Addis Ababa for earthquake design. The existing residential condominium buildings have been chosen for the study.

From the existing 40/60 high rise condominium houses located in Addis Ababa, four samples (Building Typology of $B+G+7$, $B+G+9$, $2B+G+12$ and $2B+G+18$) were chosen for the assessment. Architectural, structural as built structural detail and some design reports were taken from *CITY GOVERNMENT OF ADDIS ABABA SAVING HOUSING HOUSE DEVELOPMENT ENTERPRISE* for modeling purposes. The structural modeling was analyzed and designed by linear static and nonlinear static pushover analysis using *ETABS 22.0.0* (cracked) Engineering software. Furthermore, damage limitation limit state and ultimate limit state have been considered for the linear static analysis.

Base shear, story displacement, story drift, and design spectra were compared in this study in order to analyze the building; The findings base shear, story displacement and story drift significantly changed as the result of revision but spectrum type in the performance evaluation did not significantly change. Lastly, push over analysis was used to evaluate the case study building's performance level.

All four typologies used for the study—Building Typology $B+G+7$, $B+G+9$, $2B+G+12$, and $2B+G+18$)—had sufficient capacity to withstand damage limiting earthquakes, according to the results of linear static analysis. And going additional investigation on nonlinear static analysis showed Only two structures (structure Type $B+G+9$ and $2B+G+18$) had sufficient capacity to ensure life safety but $2B+G+18$ has good performance in Y direction between (A and Io), while the other one building (Building Type $B+G+7$ and $2B+G+12$) had inadequate for IO it is in the range of LS(life safety) near Cp (collapse preventive). But the most important discovery was that every type of building did not pass ultimate limit state evaluation, which is a crucial criterion outlined in the code.

The majority of house construction in the town area is from this construction type and the performance of the buildings against earthquake forces was shown to be poor. Accordingly, it is important to seismically retrofit the existing buildings to mitigate potential disasters to be caused by future earthquakes.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	I
Abstract	II
TABLE OF CONTENTS	IV
LIST OF FIGURES	VI
LIST OF TABLES	VIII
ACRONYM	IX
CHAPTER ONE	1
1.INTRODUCTION	1
1.1Background	1
1.2 Study Area	5
1.3 Scope of the Study	7
1.4 Objectives	7
1.4.1 General objectives.....	7
1.4.2Specific Objectives	7
1.5 Statement of the problem	8
1.6 Significance of the study.....	8
1.7 General Over View	9
CHAPTER TWO	10
2. LITERATURE REVIEW	10
2.1 Introduction.....	10
2.2 Seismic performance Evaluation methods.....	10
2.3 Linear methods of analysis	12
2.3.1 Ultimate limit state (Near collapse-NC)	13
2.3.2 Damage Limit State (DL)	13
2.3.3 Inter story drift limit.....	14
2.4 Nonlinear static analysis	15
2.4.1 Static nonlinear/pushover analysis using ETABS.....	17
2.4.2 Structural modelling.....	19
2.4.3. Finite Element Non-linear modelling of beams and columns.....	21
CHAPTER THREE	23
3.RESEARCH METHODOLOGY	23
3.1 Assessment Of seismic specifications.....	23
3.3 Performance assessment	23
3.3 General analysis strategy	24
CHAPTER FOUR.....	29

4. ANALYSIS AND RESULTS OF CASE STUDY BUILDINGS	29
4.1 Case Study Building	29
4.1.1 Criteria for Regularity in plan and Elevation.....	30
4.2 Inter story drift consideration at damage limitation.....	31
4.2.1. Damage limitation requirement nonlinear static.....	31
4.3 Equivalent lateral forces analysis.....	35
4.3.1 Base shear comparison.....	35
4.3.2 Story Displacement Comparison	36
4.4 Safety compromising based on equivalent lateral force analysis (ELFA).....	39
4.4.1 Demand to capacity Ratio of columns based on ELFA	40
4.4.2 Demand to capacity ratio of beam based on ELFA	43
4.5. Nonlinear static pushover analysis.....	45
A plot of the base shear versus displacement at a control node.....	47
4.6 Analysis results and discussion for nonlinear static pushover analysis	47
4.6.1 The general form of those case study buildings.....	47
4.6.2 Capacity curve	47
4.6.2.1. Capacity curve of case study buildings for push-x and push-y	48
4.7. Performance Point Analysis.....	49
4.7.1 Level of the structure's hinge formation at the performance point	52
4.7.1.1 Plastic hinge formation of case study buildings.....	52
4.8 Structural global performance based on story drift.....	56
CHAPTER FIVE: CONCLUSION AND RECOMMENDATION	59
6.1 Summary	59
6.2 Conclusion	59
6.3 Recommendation	61
References.....	62
APPENDIX A: CASE STUDY BUILDINGS DATA USED, MODELING AND ANALYSIS OUT DRAWINGS.....	64
APPENDIX B: SOFTWARE VERIFICATION.....	76

LIST OF FIGURES

Fig 1.1 Damage to houses in the Afar area [4]	2
Fig 1.2 Magma Intrusions Under Ethiopia Near the Fantale Volcano [5].....	3
Fig1.3 Earth quick report Data of Ethiopia [5]	3
Fig 2.1 location of Performance points.....	17
Fig 3.1 performance level of Hinges.....	26
Fig 3.2 Flow chart depicting the process followed in nonlinear static procedures [13].	28
Figure 4.1 Sample Modeling -Three-dimensional and Elevation of finite element model of the 2B+G+18 building	29
Figure 4.2 Sample Typical floor plan view of 2B+G+18 building.....	30
Figure 4.3 Equivalent lateral force and response spectrum Analysis of story displacement of all topology's	Error! Bookmark not defined.
Figure 4.4 Equivalent lateral force and response spectrum Analysis of story drift.....	39
Figure 4.5 Section Designer Modeling of Existing Structure.....	42
Figure 4.6 Column demand to capacity ratio for 2B+G+12 topology type	43
Figure 4.7 Hinge assignment in the beams	46
Figure 4.8 Capacity curve of B+G+7 for push-x Figure 4.9 Capacity curve of B+G+7 for push-y.....	48
Figure 4.10 Capacity curve of B+G+9 for push-x Figure 4.11 Capacity curve of B+G+9 for push-y.....	49
Figure 4.12 The Performance point, Demand curve capacity curve of structure.	50
Figure 4.13 The Performance point, Demand curve capacity curve of structure	50
Figure 4.14 Plastic hinge distribution of last step for B+G+7 building in X-direction on axis B.53	
Figure 4.15 Plastic hinge distribution of last step for B+G+7 building in X-direction on axis 2. 54	
Figure 4.16 Plastic hinge distribution of last step for B+G+7 building in Y-direction on axis 4. 54	
Figure 4.17 Plastic hinge distribution of last step for B+G+9 building in x-direction on axis E. 55	
Figure 4.18 Plastic hinge distribution of last step for B+G+9 building in x-direction on axis D. 55	
Figure A.1 Resent Ethiopian Earth quicks (image from Volcano Discovery)	64
Figure A.2 Sample Floor Plan 2B+G+18	65
Figure A.3 Stiffness Modifier Factor Assigning Sample for Beam and Wall.	65
Figure A.4 Plastic Hinge Assigning Sample for column.	66
Figure A.5 Axial force moment interaction Diagram for Plastic Hinge Assigning Sample in column.....	66
Figure A.6 The Assigned Plastic B+G+9	67
Figure A.7 General Story Response of B+G+12.	68
Figure A.8 Pushover curve in X direction B+G+7.	68
Figure A.9 Definition of Fiber hinge for column in 2B+G+12.	69
Figure A.10 Definition of Fiber hinge for Beam in 2B+G+12.	69
Figure A.11 Push over curve in X and Y direction for 2B+G+12.	70
Figure A.12 Demand to capacity curve (performance point) for 2B+G+12.	70
Figure A.13 Demand to capacity curve (performance point) for 2B+G+12.	71
Figure A.14 Hinge formation and number of hinges for 2B+G+12.	71
Figure A.15 The formation of Plastic Hinge Sample in columns and Beams B+G+12.	72
Figure A.16 The Plastic Hinge Formation Sample in columns and Beams B+G+12.	73

Figure A.17 The Plastic Hinge Formation Sample in columns and Beams B+G+12.	73
Figure A.18 The Plastic Hinge Formation Sample in columns and Beams 2B+G+18.	74
Figure A.19 Definition of Fiber hinge for column in 2B+G+18.	75
Figure A.18 The Plastic Hinge Formation Sample in columns and Beams 2B+G+18.	75

LIST OF TABLES

Table 1.1 Case Study Building Data to be Represent.....	6
Table 1.2 Selected Building Data	6
Table 3.1 Summary Table: Linear vs. Nonlinear Analysis.....	28
Table 4.1 Regularity in Plan of B+G+9 cheking for slenderness of the building.....	30
Table 4.2 Regularity in Plan of B+G+9 cheking for structural eccentricity	31
Table 4.3 Analysis of B+G+9 story drift result used to compute inter story drift.....	33
Table 4.4 Analysis of 2B+G+18story drift result used to compute inter story drift.....	34
Table 4.5 Linear static analysis of base shear calculation procedures.....	35
Table 4.6 Linear static analysis base shear result of the two codes	36
Table 4.7 Linear static analysis Top story displacement result of the two codes.....	36
Table 4.8 Column demand to capacity ratio for B+G+7 topology type	40
Table 4.9 Column demand to capacity ratio for B+G+9 topology type	41
Table 4.10 Column demand to capacity ratio for B+G+12 topology type	42
Table 4.11 Beam demand to capacity ratio for B+G+7 topology type.....	44
Table 4.12 Beam demand to capacity ratio for 2B+G+12 topology type.....	45
Table 4.13 Hinge status for different Step for pushover on y direction	Error! Bookmark not defined.
Table 4.14 Hinge status for different Step for pushover on X direction.....	51
Table 4.15 Hinge status for different Step for pushover on x direction	51
Table 4.16 Hinge status for different Step for pushover on y direction	52
Table 4.17 Global performance of the structure for B+G+7 building	56
Table 4.18 Global performance of the structure for B+G+9 building	56
Table 4.19 Global performance of the structure for 2B+G+12 building	57
Table 4.20 Global performance of the structure for 2B+G+18 building	58

ACRONYM

<i>2D</i>	Two Dimensional
<i>3D</i>	Three Dimensional
<i>AAiT</i>	Addis Ababa Institution of Technology
<i>ADRS</i>	Acceleration-Demand Response Spectrum
<i>ATC</i>	Applied Technology Council
<i>CP</i>	Collapse prevantation
<i>CMS</i>	Capacity Spectrum Method
<i>DL</i>	Dead Load
<i>ECBS</i>	Ethiopian Building Code Standard
<i>FEMA</i>	Federal Emergency Management Agency
<i>IO</i>	Immediate Occupancy
<i>LL</i>	Live Load
<i>LS</i>	Limit States
<i>M_x</i>	Balanced moment about x-direction
<i>M_y</i>	Balanced moment about y-direction
<i>MDF</i>	Multiple Degree of Freedom
<i>M_{edix}</i>	Design moment about x-direction
<i>M_{Edy}</i>	Design moment about y-direction
<i>MPa</i>	Mega Pascal
<i>NC</i>	Near Collapse
<i>NS</i>	Nonlinear Static
<i>PDLR</i>	Analysis Reference probability of exceedance in 50 years of Reference seismic action for the damage limitation requirement
<i>PGA</i>	Peak ground acceleration
<i>PNCR</i>	Reference probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement
<i>P_{ed}</i>	Design load
<i>P-M-M</i>	Interaction of axial load and biaxial bending moments
<i>q</i>	Behavior factor

RC	Reinforced concrete
SD	Significant Damage
TNCR	Reference return period of the reference seismic action for the no- collapse requirement
TDLR	Reference return period of the reference seismic action for the damage limitation requirement
IMP x	Force of imperfection X-direction.
IMP y	Force of imperfection Y-direction.

CHAPTER ONE

1.INTRODUCTION

1.1Background

An earth quick is a rapid, powerful shaking of the earth caused by crustal movements or volcanic activity. When these motions take place, seismic waves are emitted that have the potential to seriously harm infrastructure, buildings, and natural landscapes. Earthquakes occur when massive slabs of rock called tectonic plates, which make up the Earth's surface, collide, glide past one another, or rip apart along fault lines. Even though earthquakes are unpredictable, early warning systems and earthquake-resistant building techniques can help minimize damage [2].

To make our buildings more resilient, we first need a clear understanding of the seismic risks they face. Probabilistic performance-based assessments are a powerful way to evaluate those risks, and today's numerical tools have made these assessments more capable than ever. Still, the process can be complicated. That's why this study proposes a simpler, probabilistic, displacement-based method designed specifically for reinforced concrete wall structures [3]

As human knowledge and understanding of seismic activity continues to advance, it leads to a deeper comprehension of structural performance under earthquakes. Structures built in the past may not withstand seismic forces as effectively as those constructed today or in the future, due to evolving engineering practices, materials, and design philosophies.

To accurately assess the real-world behavior of buildings after construction, conventional code-based analysis methods may no longer sufficient. Instead, innovative approaches such as pushover analysis, advanced computational modeling, and machine learning-based predictive analysis should be implemented. These modern techniques allow engineers to evaluate dynamic responses, detect weaknesses, and improve resilience, ensuring that future structures can better withstand seismic threats. By integrating data-driven insights with traditional design principles, the engineering community can move towards safer, more adaptive infrastructure in earthquake-prone regions.

Since early September 2024, a series of earthquakes has been shaking the area between the Fanale and Methara volcanoes in central Ethiopia's Awash region, about 150 km northeast of Addis Ababa. Some of these quakes reached close to magnitude 6, causing significant damage to homes. In some spots, cracks have appeared in the ground, with hot water seeping through, creating small geysers and bursts of steam. Satellite images show that the land has risen by as much as 30 cm over a 10 km stretch, suggesting magma may be moving—or has already moved—beneath the surface. While this could lead to a volcanic eruption, it's still unclear if that will happen or if the activity will calm down on its own. As of now, the latest reports indicate that the activity seems to be tapering off [4].

In recent years, Ethiopia has experienced a significant increase in seismic activity, particularly in the Afar region. In Ethiopia, notable weekly earthquakes in the Awash region occurred on January and February 2025 G.C., and the highest record was registered at Mag. 6.0 on Friday, Feb. 14, 2025, 8Km NNE Methara, Ethiopia and since 2025 has had 220 quakes of above 4 and up to 6. [4]. The images in Fig 1.1 (showing damage to houses in the Afar area), Fig 1.2 (depicting magma intrusions near the Fanale volcano), and Fig 1.3 (presenting earthquake report data for Ethiopia) collectively demonstrate significant seismic activity in the region.

Because of this high magnitude and highly reputation have raised concerns about the seismic resilience of existing buildings, especially low-cost housing structures (Condominium)



Damage to houses in the Afar area (image: Addis Standard)

Fig 1.1 Damage to houses in the Afar area [4]

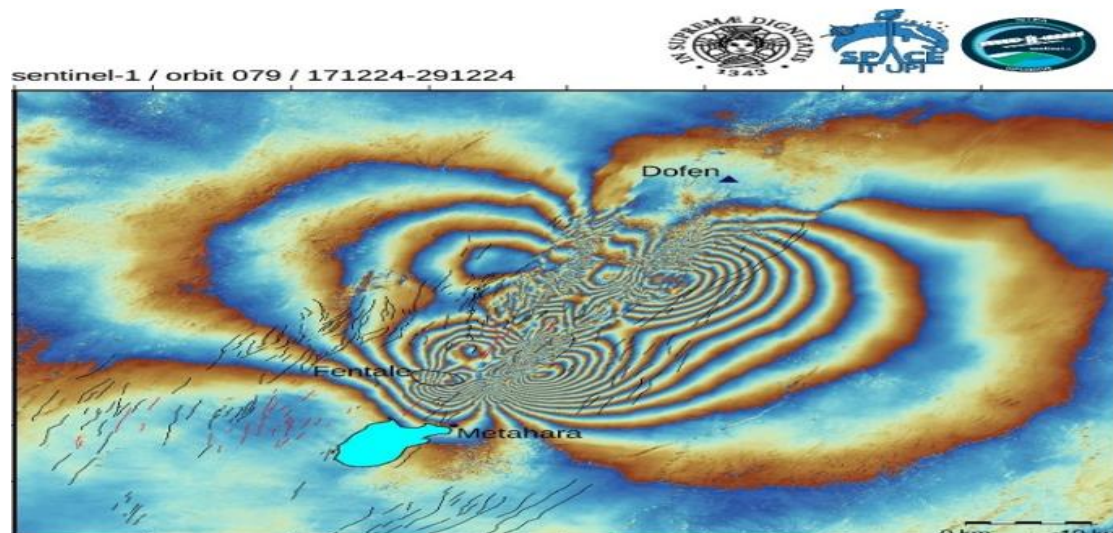


Fig 1.2 Magma Intrusions Under Ethiopia Near the Fanale Volcano [5]

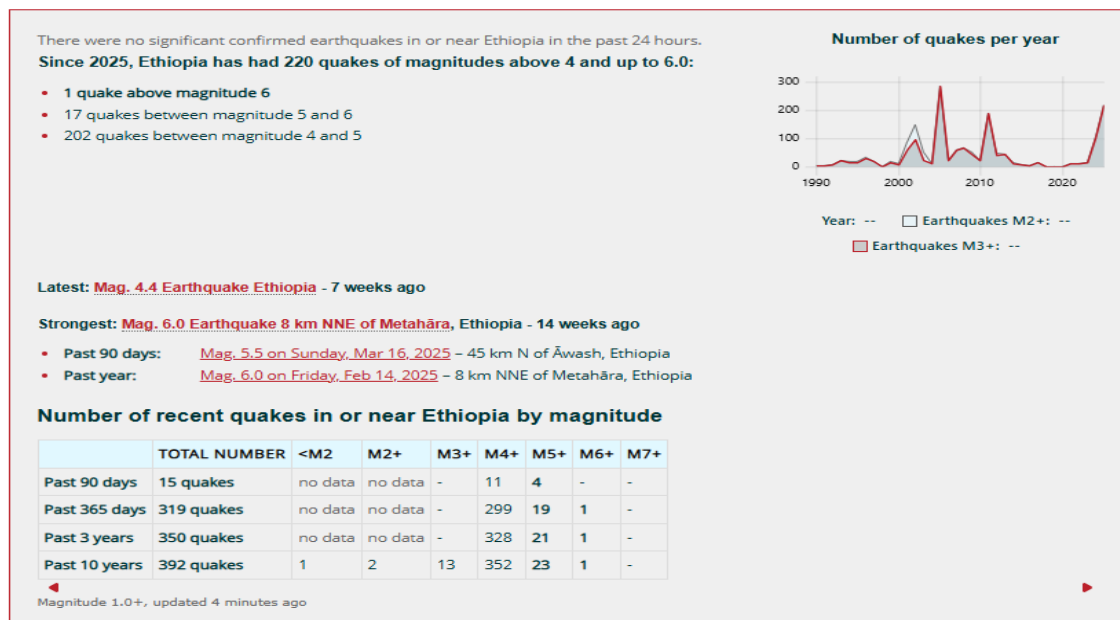


Fig1.3 Earth quick report Data of Ethiopia [5]

Condominium houses in Ethiopia building structures constructed for the purpose of Sheltering peoples in form of clustering.” The Housing Development Project Office (HDPO) was created to manage the three main parts of Addis Ababa’s IHDP: designing the homes, building them, and handling their transfer and administration. To help with the construction of the condominium units, ten HDPO branch offices were set up throughout the city. Back in 2003, GTZ reviewed the prefabricated building technologies needed for a low-cost housing program as part of its bilateral partnership. Their work wrapped up in March 2010, when the final project was handed over to

the government, officially ending their mandate. During their time as the implementing agency, they also appointed a project manager to oversee both the financial side of the program and the construction itself.” [6].

“The consulting and design firm first developed cost-efficient condominium designs for Addis Ababa while working with GTZ’s Low-Cost Housing Project. During this collaboration, they introduced the Cost-Efficient Methodology (the LOCH-MH system), which uses pre-cast concrete elements to make construction more affordable. ME also created a manual that openly shared their ideas, addressing the housing challenges faced by low-income communities and offering their conceptual design as a freely accessible solution for others to build on.” [6].

In this regard analysis and design of buildings is fundamentally concerned with ensuring that the components of buildings can adequately serve their intended functions, (Gravity load and lateral force-resisting system). “The main cause of lateral force in the analysis and design of building is seismic forces (earth quick). Seismic Analysis is one main component of structural analysis and is the calculation of the response of a building structure to earthquakes. It is a part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent. Seismic structural assessment could be done by the analytical methods of static analysis and dynamic analysis. These two main categories can be divided into two main types of analysis, the linear and non-linear analysis” [7].

The methods of analysis used in this study include both linear static analysis and nonlinear static analysis. The studied buildings in this paper are condominium residential buildings located in Addis Ababa. They are comprised of a reinforced concrete structural frame with infill shear walls. The structure members are made of in-situ reinforced concrete and ribbed slab. The other Engineering tools need to be sharpened for analyzing and design structures under the action those loads are design codes. The goal of the code is to make sure that during an earthquake, people stay safe, damage is kept to a minimum, and essential buildings needed for public safety continue to function. In this study we use design codes. The first seismic code for buildings in Ethiopia was introduced in 1980 and revised in 1984 and 1995. One thing all three codes introduced in the country over the past 40 years have in common is that they use a 100-year return period, unlike most building codes around the world, which are based on a 475-year return period. The main argument in favor of this choice has been relatively economical construction of structures designed for a less powerful earthquake [8].

This Standard Specification, which covers all types of building work, was first published in 1991 and then updated in 1995. This latest revision was created to reflect the Building Proclamation and to include improvements based on how the standard has been used, as well as feedback from the various organizations applying it. [8]

The Ministry of Construction introduced a new revision of the Ethiopian Standards aligned with Euro Norms, known as ES EN 1998:2015. This code is based on a 475-year return period and incorporates additional criteria. It establishes five seismic zones, each defined by a specific peak ground acceleration (PGA) value. Overall, the PGA values for a 475-year return period are about 33.3% to 50% higher than those for a 100-year return period. These adjustments to PGA values, along with changes in parameters and analysis requirements, increase the seismic demand on structures. As a result, it becomes necessary to evaluate the safety of existing buildings using the updated ES EN 1998:2015 seismic criteria. [9].

Building performance evaluation examines how well a structure meets its intended goals—such as serving its planned functions, satisfying its occupants, and carrying the loads it is expected to face throughout its lifespan. The main aim of this evaluation is to enhance design practices and support the ongoing improvement of design methods and building codes by providing designers with feedback on how effectively their design decisions perform.

1.2 Study Area

In 2004, the Addis Ababa city government, through EHC, launched the Addis Ababa Integrated Housing Development Program (AAHID) to help reduce the city's housing shortage (UN-HABITAT, 2011). The main goal of this program is to address public housing needs while working toward a future where the city is free of slums. The 40/60 housing scheme is one of its key projects, focused on constructing condominium buildings of up to 24 stories and positioning Addis Ababa among the world's leading cities [10]. The study try to covers all area of Addis Ababa which constructed 40/60 condominium site that constructed starting the program to now.

This study focuses on assessing the seismic performance of residential buildings within Addis Ababa's 40/60 condominium projects. The study covers fourteen 40/60 condominium sites that having 415 blocks with 9 topology types. Among the fourteen sites, we selected four topology types (B+G+7, B+G+9, 2B+G+12, and 2B+G+18) based on their typological density and diversity to ensure that the sampled structures accurately reflect the broader condominium housing site by evaluating site-specific topologies—ranging.

Addis Ababa 40/60 Condominium Sites – Official Topology & Block Distribution				
No.	Site Name	Topology Types	Number of Blocks	Source
1	Emil Nigde	2B+G+18	19	AHAD 2023
2	Tourist Site	2B+G+18	11	IHDE 2022
3	Ayat (Site 1-4)	B+G+7, B+G+8, B+G+9,2B+G+12	133	AHAD 2023
4	Bole Hayat 2	B+G+7, B+G+8, B+G+9,2B+G+13	50	IHDE 2022
5	Summit	B+G+9, B+G+10	10	AHAD 2023
6	Bole Be shale	B+G+10, 2B+G+12	53	IHDE 2022
7	Bole Bulbul	B+G+9, B+G+10	26	AHAD 2023
8	Senga Tera	B+G+7, B+G+8	5	IHDE 2022
9	Hinza Karabi /Imperial/	B+G+7, B+G+8, B+G+9	8	AHAD 2023
10	Jemo	B+G+9, B+G+10,2B+G+12	32	IHDE 2022
11	Bole Bulbul 2	B+G+7, B+G+8	13	AHAD 2023
12	Crown Hotel (Kaity)	B+G+7, B+G+8	14	IHDE 2022
13	Meri Loque	2B+G+12, 2B+G+15,2B+G+18	16	AHAD 2023
14	Kolbe Keranio/Asko	B+G+9, B+G+10	25	IHDE 2022
Total Blocks			415.00	

Table 1.1 Case Study Building Data to be Represent

Building Name	No. Story	Story Hight	Slab Type (Solid)	Slab Type (Ribbed)	Number of shear walls	No. of column per each floor
2B+G+18	22	3 m	B +GF & 10th-18th	1st-9th	4 (2in X & 2 in Y)	65.5
2B+G+12	15	3 m	Basement +GF	1st-12th	6 (4in X & 2 in Y)	45
B+G+9	12	3 m	GF	1st-9th	4 (4in X & 0 in Y)	35
B+G+7	10	4 m	GF	1st-7th	5 (4in X & 0 in Y)	27

Table 1.2 Selected Building Data

1.3 Scope of the Study

By reducing the hazards associated with earthquakes, more buildings will be built in the future. Therefore, the required precautions should be taken beforehand to lessen the loss of life, property, and economic impact caused by the earthquake's occurrence. This requires evaluating how well the built structures withstand particular types of ground shaking. As a result, the study on how earthquake-prone the current buildings are is essential. ***Equivalent Lateral Force Analysis and nonlinear analysis (push over analysis and procedures)***. Elastic response spectra, seismic hazard maps, pick ground acceleration, regularity criteria, analysis procedure selection criteria, and drift limitation are all covered on equivalent lateral force analysis and nonlinear modeling on Abashing property, nonlinear modeling plastic and fiber hinges and nonlinear seismic analysis procedures under ***nonlinear analysis***. This study solely focuses on high-rise condominium buildings for the assessment of seismic demand and performance. Only working drawings are used in the case study building's performance evaluation. Damage of any kind is not taken into account. The outcome will help retrofit mechanisms that were in place before the earthquake. Furthermore, it is advantageous to take the necessary precautions that need to be considered for future building design

1.4 Objectives

1.4.1 General objectives

The general objective of the study is

- Seismic analysis of 40/60 Condominium Building $B+G+7$, $2B+G+9$, $2B+G+12$ and $2B+G+18$ Structures by using international seismic evaluation standards such as EC8, FEMA 440 and ASCE41-17.
- Evaluate the seismic performance of a representative condominium building in Addis Ababa using the revised new code ES EN -2015.
- To show the linear and nonlinear performance evaluation mechanisms and their predication level and that difference using various type of buildings

1.4.2 Specific Objectives

- Performing Equivalent lateral and Response spectrum analysis of the given structures.
- To Quantify Seismic Response: Analyze key performance indicators such as inter-story drift, base shear, and energy dissipation capacity to quantify the seismic response of the structures and pushover analysis.

1.5 Statement of the problem

The Awash region of Ethiopia, particularly Addis Ababa, is increasingly exposed to seismic hazards due to a rise in the frequency of earthquakes. This alarming trend poses a significant risk to the safety and stability of residential buildings, particularly cost-effective housing designed to accommodate the rapidly growing urban population. Despite the urgent need for affordable housing solutions, many of these structures have been constructed without adequate consideration of seismic design principles, resulting in potential vulnerabilities that could lead to catastrophic failures during seismic events.

In our case study which is seismic evaluation of 40/60 condominium building B+G+7, 2B+G+9 & 2B+G+12 Structures is wanted to be studied because of the repeating earthquakes starting in awash region with epicenter around “Methara” city.

Many residential buildings, particularly condominiums, in Addis Ababa are designed and constructed using ECBS 1995 code which may not adequately address current seismic design standards and the Ethiopian building code has been updated to ES EN 1998-1:2015. The revised Ethiopian building code (EBCS-EN-2015) introduces significant changes to the country’s seismic hazard map, particularly regarding peak ground acceleration. For Addis Ababa, the PGA value has been revised from 0.05g to 0.1g, marking a notable increase in expected ground shaking. And also incorporate two types of spectra that depends the surface wave magnitude (Type 1 and Type 2) that happen recently earth quick fluctuate in those two ranges. The design and construction of many recent buildings have been based on outdated building codes. As a result, their structural performance is significantly impacted by the updated standards. It is therefore essential to evaluate these existing structures according to the provisions of the revised code.

1.6 Significance of the study

This study holds considerable significance for multiple stakeholders, including local communities, policymakers, architects, engineers, and the broader field of earthquake engineering, enhancing the seismic resilience of cost-effective housing in Addis Ababa and the broader awash region:

The main significance of the study is to know to what extent affects the building structures under study and to address the population for preventive measures and to give to the inhabitant

population to discover some structural member cracks which helps to create knowledge and keep the population in alert mood before some devastating situation (If total collapse of the building structures) occurs

1.7 General Over View

The study is completely integrated in to five chapters by doing so. Chapter one presents general description about the thesis including; introduction, study area, scope, objective, and thesis overview. The second chapter shows literature review on details: seismic hazard, seismic performance evaluation methods of a structure against earth quick, different methods of analysis to estimate Demand-capacity ratio (linear method). To evaluate a nonlinear behavior and performance level of buildings, nonlinear static method (pushover) was also included. The third chapter contains General methods (linear & nonlinear static analysis) & procedures that should be considered to evaluate the capacity ratio and performance level are present in the third chapter. Chapter Four This chapter included talks of the thesis along with Analyses and design results as well as discussions of the thesis were encompassed in this chapter. Chapter five covers summary, conclusion and recommendations derived from the study.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

Throughout the 20th and early 21st centuries, Ethiopia has been impacted by multiple earthquakes, resulting in loss of life and structural damage. Records from the EM-DAT database for the period 1900–2013 indicate that seismic events in the country led to 93 fatalities, 165 injuries, and left 420 people homeless. In total, approximately 11,000 individuals were affected, with economic losses surpassing US\$7 million. [2].

According to a report published in 1999, a 6.5-magnitude earthquake, which seismologists say could occur in areas near Addis Ababa, the country's major city, could cause as many as 4,000 to 5,000 deaths, 8,000 to 10,000 injuries, the displacement of up to 500,000 people, and total damages exceeding 12 billion Birr.

Addis Ababa is located only 75 to 100 kilometers from the western edge of the Main Ethiopian Rift Valley, a region known for frequent tremors and active volcanoes. Several major Ethiopian cities, including Addis Ababa, Nazret, Dire Dawa, and Awassa, lie close to significant fault lines such as the Wonji fault, the Nazret fault, the Addis-Ambo-Gedo fault, and the FilWoha fault lines along Fil Woha fault. These numerous earthquakes have experienced magnitude have occurred over magnitudes cities like Arba Minch, cities, such as Mekele are also located Mekele, some of situated most seismically active areas in the country. The of the for example, the Woha (hot springs) in the hot springs Addis Ababa center for example, is serves as a natural lie on fault lines rests have been slowly where strain has It gradually accumulating. The accumulated over this accumulated strain cause time causes earthquakes. [11].

2.2 Seismic performance Evaluation methods

The structure is considered safe depending on the performance objective of the structure in performance-based analysis and design. Two key elements of a performance-based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion and Capacity is a representation of the structure's ability to resist the seismic demand.

Two or multiple-stage performance targets can be made by choosing two or more distinct intended performances, each for a separate ground motion level, according to ATC-40 A limiting damage condition that might be deemed acceptable for a specific building and ground motion is described

by a performance level. The physical damage to the building, the risk to the life safety of the structure's residents that the damage creates, and the building's capacity to be serviced after an earthquake are the limiting conditions [12].

In other way “Seismic performance analysis or simply seismic analysis is a major intellectual tool of earthquake engineering which breaks the complex topic into smaller parts to gain a better understanding of seismic performance of building and non-building structures” [13].

Different analysis methods, based on linearity or Elasticity we have elastic (linear) and inelastic (nonlinear) analyses,

Linear analysis (Elastic) methods include

- ❖ Linear Static Analysis [static lateral force procedures]
- ❖ Linear Dynamic [dynamic lateral force procedures]
- ❖ Elastic procedures using demand capacity ratios.

Nonlinear or inelastic analyses

- ❖ Nonlinear static analysis (pushover)
- ❖ Nonlinear dynamic analysis (NLTHA)

Based on Dynamic Character, the above methods are also categorized. The static analysis method and dynamic analysis are available for the analysis of existing concrete buildings.

Static analysis methods include

- ❖ Linear static analysis [Equivalent static method]
- ❖ Nonlinear static method [push over analysis]

Dynamic analyses

- ❖ Linear dynamic (Response Spectrum)
- ❖ Nonlinear dynamic analysis (NLTHA)

When linear static or dynamic methods are applied in seismic evaluation, the design earthquake forces, the vertical distribution of loads along the building height, and the resulting displacements are calculated assuming linear elastic behavior. However, these linear approaches have limitations and often fail to provide reliable results for structures that experience nonlinear response during seismic events [7]. The primary method for inelastic structural analysis is full nonlinear time-history analysis. In addition, several simplified nonlinear approaches, commonly known as nonlinear static analysis or pushover analysis, are also available.

- ❖ Capacity spectrum method (CSM)
- ❖ Displacement coefficient (DC) method and
- ❖ Secant method (SM)

2.3 Linear methods of analysis

“The requirements for this method's applicability are outlined in ES EN 1998-1; 2015, with the addition of the following: $\rho_i = \frac{D_i}{C_i}$ represents the ratio between the demand D_i derived from the analysis under the seismic load combination and the corresponding capacity C_i for the i -th "ductile" primary element of the structure (bending moment in moment frames or shear walls, axial force in a bracing of a braced frame, etc.); $\rho_{\max}$ and $\rho_{\min}$ are the maximum and minimum values of ρ_i , respectively, over all "ductile" primary elements of the structure with $\rho_i > 1$, and the ratio $\rho_{\max}/\rho_{\min}$ does not exceed a maximum acceptable value in the range of 2 to 3”[15]. “On the basis of comparing the total flexural capabilities of the beams and columns, the ratio ρ_i around beam-column joints must only be assessed at the portions where plastic hinges are anticipated to form. When calculating the capacity C_i , the structural model will use the mean values of the material attributes. The value of the axial force may be assumed to be equal to that resulting just from the vertical loads in order to calculate the bending moment capacity C of vertical parts” [14].

The capacity of brittle elements or mechanisms (C_i) must exceed the corresponding demand (D_i), which is calculated from equilibrium conditions based on the action effects transferred from ductile components. In this assessment, all action effects from ductile components acting on the brittle element or mechanism are taken as unity.

- A. The analysis's value D (demand) fulfills $\rho = D/C \leq 1$ if the ductile component's capacity C , as determined by the mean values of its material properties.
- B. The structure's capacity indicates its ability to withstand seismic demands. The strength and deformation capabilities of a structure's different parts determine its overall capacity.

The ability of structural systems to withstand seismic actions in the non-linear range typically enables their design for resistance to seismic forces smaller than those corresponding to a linear elastic response, allowing for the determination of capacities beyond the elastic limits. We can account for a structure's ability to safely absorb and dissipate earthquake energy (through bending or other methods) by using a simplified, scaled-down model of the seismic forces. Instead of running a highly complex analysis of the structure's damaged state, engineers perform a standard elastic analysis using a reduced "design spectrum." This practical approach avoids the need for an explicit, complicated inelastic analysis during the design phase.

The behavior factor (q) is introduced in order to achieve this reduction. When using a traditional elastic analysis model, the behavior factor q approximates the ratio of the seismic forces that the

structure would experience if its response was fully elastic with 5% damping to the seismic forces that could be used in the design while still guaranteeing a satisfactory response. The q values, which also take into consideration the impact of viscous damping that is less than 5%, are provided for different materials and structural systems based on the pertinent ductility classes in the several ES EN 1998;2015 parts. Although the ductility classification will be the same in all directions, the behavior factor q value may vary in the structure's horizontal directions [1]. The following two limit states are examined for this Study utilizing the linear analysis method.

2.3.1 Ultimate limit state (Near collapse-NC)

The design seismic activity pertinent to this Limit State will serve as the basis for demands. Demands for brittle and ductile elements will be assessed in light of the analysis's findings. When employing a linear analysis approach, the demands placed on brittle elements must be adjusted to match the demand to capacity ratio, or the value D derived from the analysis, if the ductile component's capacity C , as determined by the mean values of its material properties, fulfills ($D/C=1$). For ductile elements, capacities must be based on properly characterized ultimate deformations; for brittle elements, they must be based on ultimate strengths.

"The Significant Damage Limit State is satisfied by the values of $q = 1.5$ and 2.0 for steel and reinforced concrete structures, respectively, as well as by the higher values of q that may be justified in light of the local and global available ductility in compliance with the pertinent provisions of EN 1998-1: 2004. Use a q -factor value that is roughly one-third higher than those in the above if the q -factor approach is selected to check the Near Collapse Limit State" [14].

2.3.2 Damage Limit State (DL)

The damage limitation (DL) limit state is one of the two primary seismic performance levels defined in EBCS 8 (the other being the "No Collapse" (NC) limit state). Its purpose is to ensure that a structure remains serviceable after an earthquake, minimizing damage to both structural and non-structural elements. The design seismic activity pertinent to this Limit State will serve as the basis for demands. Capabilities for all structural elements ductile and brittle must be determined by their yield strengths, with the exception of the q -factor approach. The mean inter story drift capacity for infills will be used to determine their capacities. The mean inter-story drift will be used to compare demands and capacity in the q -factor method [15]. The damage limitation requirement is considered satisfied if, for a seismic action with a higher likelihood of occurrence

than the design earthquake for the no-collapse condition, the inter story drifts remain within the specified drift limits.

Comparison with No Collapse (NC) Limit State

Aspect	Damage Limitation (DL)	No Collapse (NC)
Seismic Action	Reduced ($v \cdot a_g$)	Full design seismic action
Return Period	~50 years (frequent)	~475 years (rare)
Objective	Prevent non-structural damage & ensure serviceability	Prevent collapse, life safety
Drift Limits	0.5%–0.75%·h	Generally, not explicitly limited (but P-Δ effects must be checked)
Structural Damage	Minimal (elastic behavior)	Ductile inelastic behavior allowed

2.3.3 Inter story drift limit

In earthquake-resistant structural design, the inter story drift limit is specified to ensure sufficient building stiffness and to control excessive seismic deformations, thereby minimizing damage to structural and non-structural elements. The allowable inter story drift ratio (δ_{rv}) is defined according to the type of non-structural elements and their interaction with the structural system:

- ❖ *For buildings that have fragile non-structural components, such as masonry partitions or glass facades, the design must limit the movement between floors to a small, safe amount. The allowable drift is calculated as 0.5% of the story height (h), ensuring these brittle elements are protected from damage during a seismic event.*
- ❖ *When a building has non-structural elements that are designed to be flexible, such as certain partition walls or curtain systems, the allowable drift between floors can be more generous. The design can permit a movement of up to 0.75% of the story height (h), allowing the structure to sway a bit more while these components safely bend and flex*
- ❖ *For buildings where non-structural elements are either:*
 - ✓ Fixed in a manner that prevents interference with structural deformations, or
 - ✓ Entirely absent, the inter story drift ratio shall not exceed **0.010h**.

Reduction Factor (v) for Importance Classes

The reduction factor (v) accounts for the building's importance class and modifies the seismic action spectrum for damage limitation checks. The elastic response spectrum used for this assessment follows the same shape as the design spectrum for the no-collapse requirement under ES EN 1998-1:2015.

The national annex specifies the appropriate values of (v), which may vary by seismic zone based on hazard levels and property protection objectives.

The suggested default values are $v = 0.4$ for Importance Classes III and IV (lower priority) and $v = 0.5$ for Importance Classes I and II (higher priority). This method helps keep structural deformations within acceptable limits while reflecting seismic risk and the importance of the building [1].

2.4 Nonlinear static analysis

Static nonlinear/pushover analysis/, can provide idea of global behavior and possible failure with less computational effort. It works convert multiple degree of freedom models to single degree of freedom utilizing pushover analysis to determine capacity curve. Static nonlinear have two analyzing methods

- ❖ Capacity spectrum method (CSM)
- ❖ Displacement coefficient methods (DCM)

Push-over analysis pushes the structure laterally in a single direction while applying a specific lateral force or displacement distribution until a structural instability occurs or a predetermined drift is reached. A structure's seismic performance can be assessed and its weaknesses or weak points identified by using one of these analysis techniques. The lateral force analysis approach is utilized because the response of buildings whose contributions from vibration modes larger than the fundamental mode in each primary direction do not significantly affect [16].

A. Capacity spectrum method (CSM): - Uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement. The displacement coefficient method. Based on the equivalent linearization approach, convert the push-over curve to spectral acceleration and spectral displacement. This graph is called the capacity spectrum, and the response spectrum is converted to the acceleration-displacement response

spectrum. This graph is called the demand spectrum. Then the capacity spectrum and demand spectrum are plotted together. The point where they intersect is called the performance point, which is an estimate of maximum displacement.

- B. *Displacement coefficient methods (DCM)*:** -Applies modifying coefficient to pick elastic displacement to determine target displacement. The pushover curve is used to determine effective stiffness, period, and when used with a response spectrum, it gives spectral acceleration. Finally, spectral acceleration is converted into elastic displacement, to which modifying coefficients are applied to determine target displacement.

The capacity spectrum approach is applied in this study. This approach compares the response spectra representations of the earthquake demand with a graphical depiction of the structure's global force-displacement capacity curve, also known as the pushover curve. The graphical representation gives a quick and clear image of how different retrofit techniques, like adding stiffness or strength, would affect the building's response to seismic deformation. It also shows how a building reacts to ground motion during an earthquake. Using the Capacity Spectrum Method to assess a building's performance requires you to understand two fundamental things: what the building can handle, and what the earthquake demands of it. Essentially, you compare the structure's own strength and flexibility (its "capacity") against the expected seismic forces (the "demand"). The building's performance depends on this relationship. Let's look at each of these elements in more detail.

- A. *capacity*: A building's capability to withstand seismic forces is structural capacity. This capacity is determined by combining the strength and deformation abilities of all structural elements. When evaluating performance beyond elastic behavior (where permanent deformations occur), advanced nonlinear analysis methods like pushover analysis become necessary to accurately assess the structure's ultimate resistance.
- B. *Demand*: it is a displacement estimate of the maximum expected response of the building during the ground motion that is demanded displacement.
- C. *Performance*: To check a building's performance, we start by comparing two things: its capacity curve (showing how much force and movement it can handle) and the demand displacement (showing the forces and movement an earthquake would place on it). The key is the point where these two curves meet. If, at that intersection, the building's movement and the forces involved stay within acceptable limits, we know that both the main structure and the interior elements (like walls and fixtures) will likely remain safe

and not sustain excessive damage. The way these two curves intersect also tells a clear story about the building's seismic health:

- Good Performance (Figure 2.1 A): If the demand curve meets the capacity curve while the building is still mostly in its elastic, "spring-back" range, it means the structure has strong, resilient resistance to the earthquake.
- Poor Performance / Need for Retrofit (Figure 2.1 B): If they intersect only after the building has already used up most of its strength and ability to bend, it indicates the structure will perform poorly. In this case, it likely needs to be strengthened (retrofitted) to prevent serious damage or collapse in a future quake.
- Critical Failure (Figure 2.1 C): Sometimes, the two curves don't intersect at all. This usually happens when a key part of the structure would fail early on, preventing the building from developing enough overall strength to even meet the earthquake's demand. This signals a critical vulnerability

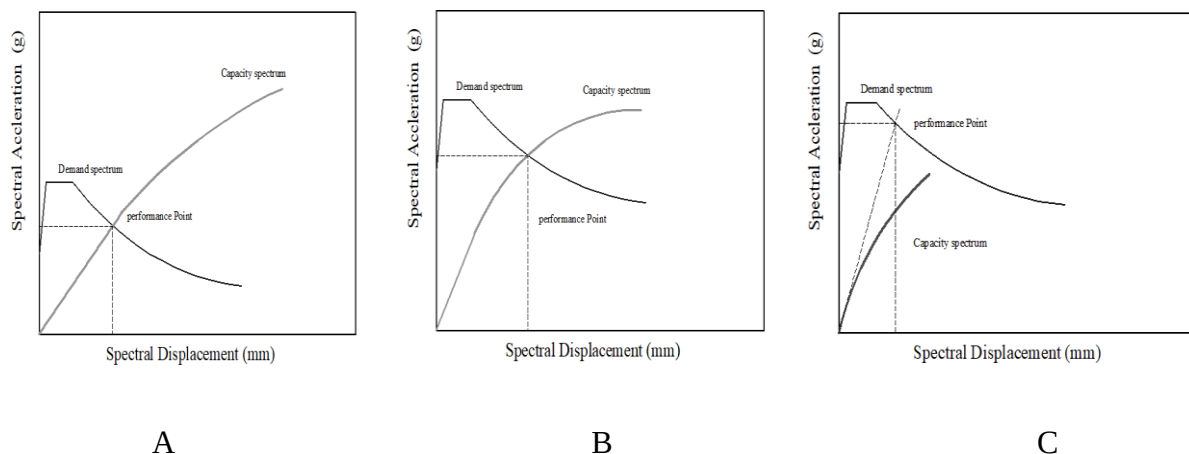


Fig 2.1 location of Performance points

2.4.1 Static nonlinear/pushover analysis using ETABS

Static nonlinear analysis allows for the consideration of various nonlinear effects in structural modeling. The approach takes into consideration geometric nonlinearity throughout all structural components, nonlinear material behavior in link elements, and material nonlinearity through predetermined plastic hinges in frame elements. In ETABS software, these static nonlinear analyses operate as standalone procedures, separate from and unaffected by other analysis types

performed on the model. The application of loads in static nonlinear analysis can be implemented through two distinct control methods - force control or displacement control - depending on the load characteristics and anticipated structural response. The following section will present the step-by-step methodology for conducting static nonlinear analysis using ETABS software.

To perform a nonlinear static pushover analysis in ETABS, here is the typical workflow you we follow:

1.Modeling: -Develop the structural model following standard modeling procedures

2.Hinge Property Definition: -In definition of frame hinge the main thing to be consider is hinge type

Hinge Type	Behavior	Where to Assign
Moment (M3)	Plastic bending failure	Beam ends, Column ends
Shear (V2)	Shear failure	Short columns, Deep beams
Axial (P)	Compression/Tension yielding	Braces, Truss members
P-M-M (PM2M3)	Combined axial + biaxial bending	Columns

In our model we define Moment (M3) and P-M-M (PM2M3)

3.Preliminary Analysis Setup: -Define required static and dynamic analysis cases and run preliminary analyses if needed for design verification

4.Concrete Design (if applicable): - Before performing pushover it requires Run Concrete Design and Verify reinforcement is realistic (no excessive rebars) and Adjust design preferences if needed:

5.Perform concrete design when using default hinge properties: - Ensure reinforcement is properly calculated

6.Lateral Load Setup: - Define lateral load patterns for pushover analysis and configure load distribution (modal, uniform, etc.)

7.Nonlinear Analysis Configuration: -Create nonlinear static analysis cases with: Gravity load application phase (load-controlled) and Pushover phase (displacement-controlled). Set monitoring point (typically roof displacement) and enable geometric nonlinearity if required

8.Analysis Execution: -Run the configured pushover analysis cases

9.Results Evaluation: -Review pushover curve (base shear vs. monitored displacement) Examine hinge formation sequence and states Analyze force/moment distribution sand Verify other relevant output parameters

10.Model Refinement: -Modify model based on analysis findings and Repeat analysis if necessary to achieve desired performance

The static nonlinear analysis in ETABS generates multiple valuable output formats for performance evaluation, including: A graphical representation of base shear versus roof displacement (pushover curve), tabular data showing the relationship between base reactions and monitored displacements at each analysis step, acceleration-displacement response spectrum (ADRS) formatted plots of the pushover results, numerical output of capacity spectrum data comparing structural capacity with seismic demand, visualizations of plastic hinge development sequences with color-coded hinge states (IO, LS, CP), and step-by-step graphical displays of member forces and stresses throughout the analysis

2.4.2 Structural modelling

Structural models are simplified, purpose-built versions of the actual building, designed to accurately simulate its core physical behavior. For earthquake response analysis, three modeling layers are typically utilized: detailed, stick, and substitution models. “**Substitution models** are idealized as an equivalent single degree of freedom system or substitute system. Four parameters are needed to define the substitute system: *effective mass*, *effective height*, *effective stiffness* and *effective damping*. The height defines the location of the equivalent or effective mass of the

substitute system” [3]. Stick models are systems with multiple degrees of freedom, where each component idealizes several components of the prototype structure. Each story of a multi-story building structure is represented by a finite element that captures the deformational properties of every column and how it interacts with the beams by a single line. The stick element for three-dimensional models connects the narrative torque to the relevant inter-story translations and rotations, and the shear forces in two horizontal orthogonal directions. **Detailed models** include general finite element idealizations and detailed fiber modeling of each section of the frame system in which structures are discerned into a large number of elements with section analysis or spatial elements in 2D or 3D. Such a modelling approach allows representation of details of the geometry of the members and enables the description of the history of stresses and strain station fibers along the length or across the section dimensions. In the detailed modelling approach, beams and columns of frames are represented by flexural elements, braces by truss elements, and shear and core walls by 2D elements, such as plates and shells [3].

All new and existing components that significantly influence a structure’s mass, strength, stiffness, and deformability near the target performance level should be included in the analytical model. Elements shown to have negligible impact on the structural assessment may be excluded. Components expected to continue contributing to lateral resistance and stiffness through repeated seismic loading must be modeled. Allowing the omission of certain elements simplifies the analysis while still representing the structure at or near the expected performance point. For example, stiff but weak components, such as coupling beams or specific wall segments that are likely to yield or degrade well before the overall lateral system reaches its capacity, do not need to be explicitly modeled [13].

An element or section of a structure that resists vertical and/or lateral loads. The vertical components of shear walls, frames, and combined frame walls are frequently used in reinforced concrete construction. Typically, horizontal components are diaphragms made of reinforced concrete. Foundations made of reinforced concrete have both horizontal and vertical components. Component modeling is described by elements, which include beams, slabs, columns, joints, wall segments, and others.

2.4.3. Finite Element Non-linear modelling of beams and columns

Model must also accurately depict the strength distribution in non-linear analysis. The model should also take into consideration how joint regions—such as the end zones in beams or columns of frame-type structures—contribute to the building's deformability. It's also important to consider nonstructural factors that could affect how the main seismic structure responds. In general, the structure may be considered to consist of a number of vertical and lateral load-resisting systems, connected by horizontal diaphragms. When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the center of gravity [7].

Two planar models, one for each main direction, may be used for the examination of buildings that meet the requirements for regularity in plan with the constraints outlined in ES EN 1998.1;2015. The influence of cracking should generally be taken into consideration when evaluating the stiffness of the load-bearing parts in concrete, composite steel-concrete, and masonry constructions. The beginning of the reinforcement's yielding should be correlated with this stiffness. Unless a more detailed evaluation of cracked behavior is carried out, the elastic flexural and shear stiffness of concrete and masonry elements may be assumed as 50% of their uncracked stiffness. Structural masses should be derived from the gravity loads specified in the action combinations of ES EN 1998-1:2015. For buildings with lateral-resisting systems in two orthogonal directions, both directions must be taken as the principal directions of analysis [1].

Nonlinear modeling and analysis of complex structures (e.g., high-rise buildings with RC shear walls) is generally considered a difficult area in structural engineering practice due to many reasons. Firstly, it requires a detailed understanding of various complex interactions and phenomena (associated with individual inelastic components). Secondly, nonlinear analysis also demands significant computational effort and the use of specialized computer software. In some cases, the obtained results can be significantly sensitive to nonlinear modeling assumptions and inelastic properties of components, which may not always be well-defined. An ordinary design office may not have necessary resources to undergo this process for each project. For most practical cases, the linear elastic analysis may serve the purpose of estimating design demands within their required degree of accuracy. However, with the advent of the latest “Performance-based Design (referred to onwards as PBD)” methodology, the need for nonlinear modeling and analysis is growing rapidly as the structural engineers are constantly trying to equip themselves

with the latest technological advancements. The above-mentioned limitations are also diminishing with the development of the latest seismic analysis solvers, software tools, and guidelines (e.g., ASCE/SEI 41-06/13), which provide significant help in understanding and implementing the nonlinear modeling of structural components.

CHAPTER THREE

3.RESEARCH METHODOLOGY

3.1 Assessment Of seismic specifications

The methodology for performing this study is checking the structural design of 40/60 Condominium Building B+G+7, B+G+9, 2B+G+12 and 2B+G+18 Structures using structural design software ETABS 22.0.0 incorporates **linear static analysis** and **nonlinear static (pushover) analysis** to assess structural behavior under seismic loads, following international codes such as **ES EN 1998 (2015)**, **EN 1998-1:2004**, and guidelines from **FEMA 440** and **ATC-40** and Second order elastic global analysis, imperfection with appropriate gravity loads, seismic loads and design combination also considered based on the new ES EN1998; 2015 code. The analysis and design were generated according to the requirements of EN1998-2004 in this software.

3.3 Performance assessment

1. Linear Static Analysis

- The **Capacity Ratio Method** compares the **demand force** (load on each member) with its **capacity** (strength).
- If **Demand/Capacity** ≤ 1 , the member is safe; if >1 , it fails.

2. Nonlinear static analysis (pushover)

- ❖ A **Demand-Capacity Curve** plots the building's strength vs. deformation to assess seismic performance (e.g., Life Safety, Collapse Prevention).
- ❖ **FEMA 440** and **ATC-40** guide this analysis for nonlinear behavior.

3. Limit State Checks

- **Ultimate Limit State:** Ensures safety against collapse (checked via Capacity Ratio).
- **Serviceability Limit State:** Controls damage (checked via **Inter-Story Drift**, per **ES EN 1998:2015**).

Generally, in this section, the process for conducting a seismic performance assessment of the case study buildings is explained in a sequential manner. The evaluation combines **linear static analysis** (for code compliance) and **nonlinear static (pushover) analysis** (for performance-based assessment) following **ES EN 1998:2015** and **FEMA 440** guidelines.

3.3 General analysis strategy

A. Collection of Building Information from Drawings: -Gather structural details for accurate modeling.

- ❖ Extract dimensions, material properties, reinforcement details, and member sizes from architectural and structural drawings.
- ❖ Verify data completeness (e.g., missing slab thickness or beam sections may require assumptions).

B. Data Sufficiency Evaluation: - Ensure the data is adequate for seismic analysis.

- ❖ Are all structural elements (beams, columns, slabs, walls) documented?
- ❖ Are material strengths (concrete grade, steel yield strength) specified?
- ❖ Are seismic design parameters (soil type, seismic zone) available?
If insufficient:
 - ❖ Use conservative assumptions (e.g., minimum code-specified reinforcement)
 - ❖ Refer to similar buildings for missing details.

C. Modeling the Case Study Buildings: -Create a computational model for analysis.

1.**Software Used:** ETABS 2022.0.0 (finite element-based) (cracked).

2.**Modeling Approach:**

- ❖ **Frame Elements:** Beams and columns.
- ❖ **Shell Elements:** Slabs and shear walls.
- ❖ **Material Properties:** Assign concrete C25/30, C30/37) and steel (S300, S400).

3.**Load Application:**

- Dead loads (self-weight, finishes).
- Live loads (per ES EN 1998:2015 and EBCS).
- Seismic loads (per ES EN 1998:2015 and EBCS).

4.**Boundary Conditions:**

- Fixed base (no soil-structure interaction).

D. Linear Static Analysis (Equivalent Lateral Force Method): -Check elastic response under seismic loads.

1. Combinations of loads:

- ❖ The factored gravity load combination used in the design is $1.35 \text{ DL} + 1.5 \text{ LL}$.
- ❖ The seismic load combination in the X-direction is represented by $1.0 \text{ DL} + 0.3 \text{ LL} \pm 1.0 \text{ EQX} \pm \text{IMPX}$, which combines earthquake forces and impact effects with dead and some live load.
- ❖ $1.0 \text{ DL} + 0.3 \text{ LL} \pm 1.0 \text{ EQY} \pm \text{IMPY}$: This load combination, which combines gravity loads with earthquake and impact loads, also applies to seismic effects in the Y-direction.

2. Analysis Outputs:

- Member forces (M, V, N).
- Story drifts.

E. Inter-Story Drift Check (Damage Limitation): -Ensure serviceability under seismic loads.

1. Calculate **inter-story drift** (lateral displacement difference between two consecutive floors).

2. Compare with code limits (ES EN 1998:2015):

- **Damage Limitation (Serviceability):** Drift $\leq 0.5\%$.
- **Collapse Prevention (Ultimate):** Drift $\leq 2.0\%$. **If exceeded:**
- Redesign stiffening elements (shear walls, bracings).

F. Demand-Capacity Ratio (DCR) Check (Ultimate Limit State): -Verify member strength against seismic demand.

1. **Calculate Demand (D):** The internal forces (moment, shear, axial) induced in a structural member due to applied loads (gravity, seismic, wind). Extract forces from ETABS (e.g., moment (Moment Demand (M_d)), shear (Shear Force Demand (V_d)), Axial Force Demand (N_d))

2. **Calculate Capacity (C):** The maximum resistance a member can sustain before failure. Calculated based on material strength (concrete, steel) and cross-section properties.

- ❖ **Beams:** Moment capacity = $A_s f_y (d - a/2)$
- ❖ **Columns:** Axial-moment interaction (PMM) per Eurocode 2.

3. **Compute DCR:** $\text{DCR} = \text{Demand Force (D)} / \text{Capacity (C)}$

- ❖ Safe if **DCR** ≤ 1.0 (per FEMA 356).
- ❖ Unsafe if **DCR** > 1.0 (requires redesign).

G. Presentation of Results (Linear Analysis)

1. **DCR Values:** Tabulate critical members (e.g., beams, columns).
 2. **Drift Ratios:** Highlight floors exceeding limits.
 3. **Code Compliance Summary:**
- Pass/Fail status for serviceability and ultimate limits.

H. Nonlinear Static (Pushover) Analysis: - Assess post-elastic behavior and failure mechanisms and Determines performance levels (Immediate Occupancy, Life Safety, Collapse Prevention). To perform this step, apply gravity loads first then Push the structure laterally until failure (using displacement control) and Plot capacity curve (Base Shear vs. Roof Displacement) finally Compare with demand spectrum (per ATC-40 & FEMA 440).

1. Define Hinge Properties:

- Use FEMA 356 Performance criteria (IO (Immediate Occupancy): Minor damage, fully operational, LS (Life Safety): Repairable damage, safe for evacuation and CP (Collapse Prevention): Near collapse, severe damage.)

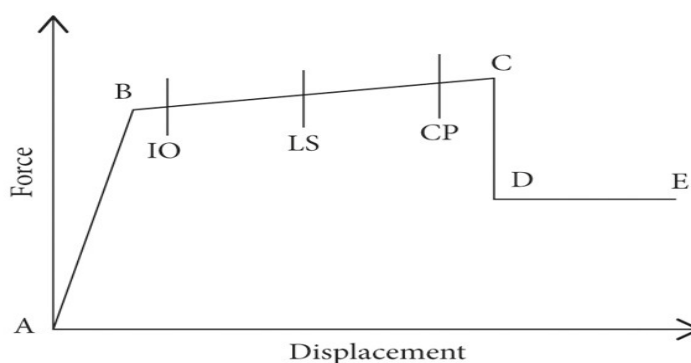


Fig 3.1 performance level of Hinges

2. Apply Lateral Load Pattern:

- ❖ Triangular or uniform distribution.

3. **Push to Failure:** Monitor hinge progression.

I. Nonlinear Analysis Results:

1. **Capacity Curve:** Base shear vs. roof displacement.
2. **Performance Point:** Intersection of capacity and demand spectra (per ATC-40).
3. **Hinge Formation Sequence:**

- ❖ **IO (Green):** Minor cracking.
- ❖ **LS (Yellow):** Yielding, repairable damage.
- ❖ **CP (Red):** Collapse imminent.

“The operational and life safety building performances for this level of ground motion is selected as per the recommendation of ATC40 sufficient enough to carry out seismic evaluation of the buildings” [12]

J. Conclusions and Recommendations

- ❖ **Linear Analysis:** Identified weak members (high DCR) and excessive drifts.
- ❖ **Nonlinear Analysis:** Revealed ductility and collapse mechanisms.

Recommendations:

1. **Retrofit Solutions:**
 - a. Add shear walls or bracings for drift control.
 - b. Strengthen columns with high DCR.
2. **Design Improvements:**
 - a. Increase beam reinforcement in critical zones.

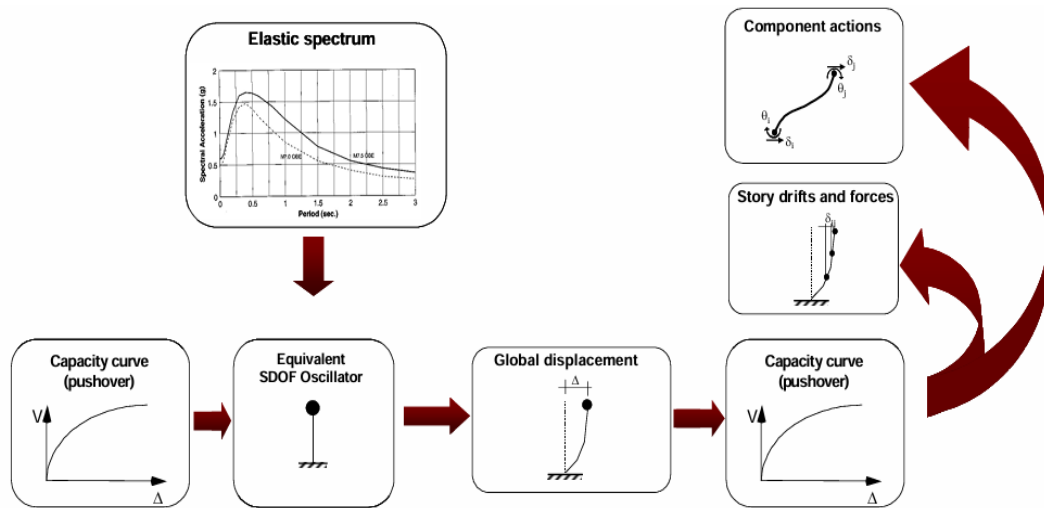


Fig 3.2 Flow chart depicting the process followed in nonlinear static procedures [13].

Note that component actions are based on global displacement demand and a pushover/capacity curve [13].

Summary Table: Linear vs. Nonlinear Analysis

Aspect	Linear Static Analysis	Nonlinear Pushover Analysis
Purpose	Code compliance (elastic checks)	Performance-based assessment (ductility)
Outputs	DCR, story drifts	Hinge formation, capacity curve
Governed by	ES EN 1998:2015	FEMA 440, ATC-40
Limitations	Ignores plastic behavior	Computationally intensive

Table 3.1 Summary Table: Linear vs. Nonlinear Analysis

This structured approach ensures a **comprehensive seismic evaluation**, combining **code-based checks** with **advanced nonlinear assessment** for safety and performance.

CHAPTER FOUR

4. ANALYSIS AND RESULTS OF CASE STUDY BUILDINGS

4.1 Case Study Building

In this chapter discussed the observed results from equivalent linear static & nonlinear static analysis. In the case of linear static analysis [Response spectrum and Equivalent static Analysis] to compare seismic demand of artificially generated ground motion, EBCS 8 1995 and ES EN 1998:2015 using base shear comparison, story displacement comparison, inter story drift comparison & capacity ration (PMM) have been obtained for each frame elements. As indicated graphically, the seismic performance level result has been obtained from nonlinear static analysis. Each result obtained from linear and analysis is discussed here under.

Based on the available design drawings and specifications, the structural properties of the reinforced concrete (RC) members in these buildings are known. This is because the buildings were originally designed and constructed in compliance with the EBCS-1995 code. Using the integrated building design software ETABS 2022(cracked), a three-dimensional finite element model of the buildings was developed for analysis. Figure 4.1(Sample from 4 topology 2B+G+18)

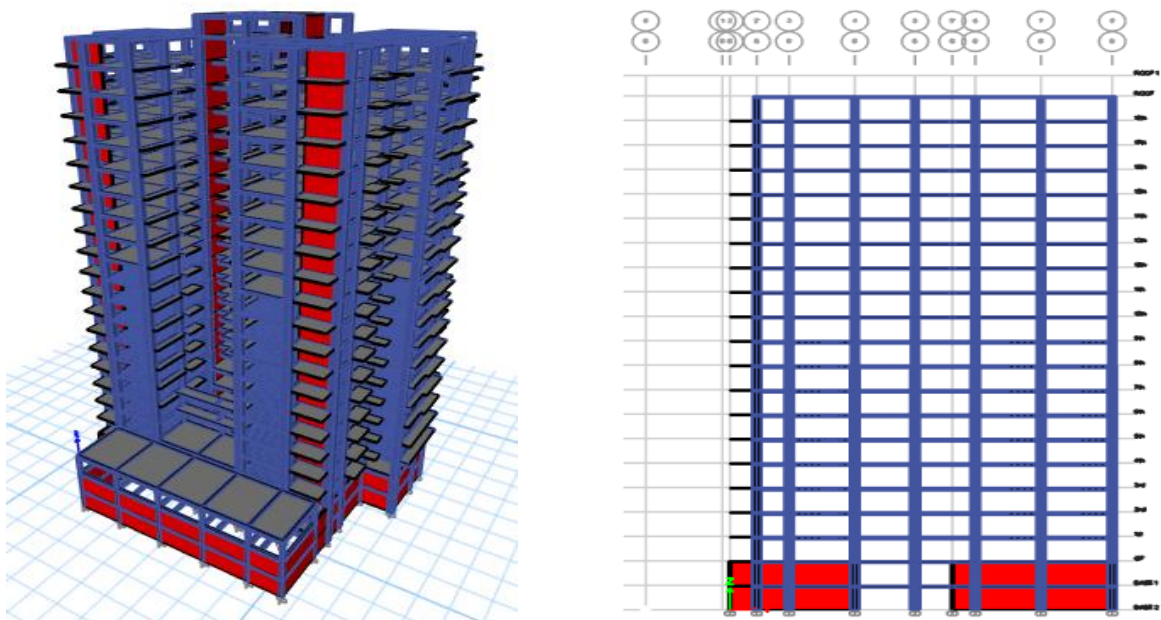


Figure 4.1 Sample Modeling -Three-dimensional and Elevation of Final element model of the 2B+G+18 building

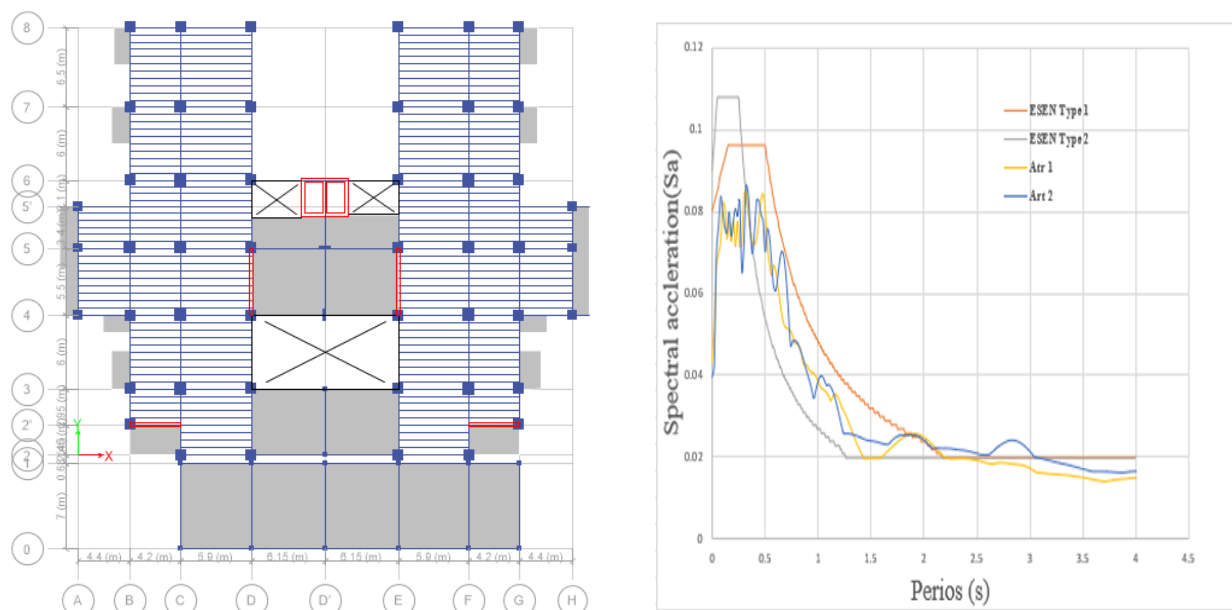


Figure 4.2 Sample Typical floor plan view of 2B+G+18 building and Response spectrum comparison

4.1.1 Criteria for Regularity in plan and Elevation.

In general, the regularity in plan can be checked when the structure model is defined. The criteria for regularity in plan are described in ES EN 1998-1(4.2.3.2)

- ❖ The slenderness of the building shall not be higher than 4 ($\lambda = L_{max} / L_{min}$).
- ❖ The structural eccentricity shall not be smaller than 30% of the torsional radius ($e_{ox} \leq 0.30r_x$, $e_{oy} \leq 0.30r_y$ and
- ❖ The torsional radius shall be larger than the radius of the gyration of the floor mass in plan ($r_x \geq l_s$, $r_y \geq l_s$). Sample for B+G+9 calculated and summarized as follow.

L_{max}	42.4	1.02663438	≤ 4	OK!	ESEN 1998-8:2015-----Page #35
L_{min}	41.3		For the case of typical floors from 1st to roof		
L_{max}	10.9	0.88617886	≤ 4	OK!	ESEN 1998-8:2015-----Page #35
L_{min}	12.3		For the case of water tanker		

Table 4.1 Regularity in Plan of B+G+9 checking for slenderness of the building

Story	Diaphragm	UX	UY	RZ	K _{Fx}	K _{Fy}	K _{MT}	r _x	r _y	<0.3r _x	<0.3r _y	e _{ox}	e _{oy}	CHECK PLAN REGULARITY
		mm	mm	rad	(KN/m)	(KN/m)	(KNm/rad)	m	m	m	m	m	m	
ROOF	10 th	152.982	89.702	0.002868	6.537	11.148	348675.03	176.85	230.96	53.056	69.287	0.231	3.154	regular
9th	9 th	146.626	95.286	0.003246	6.820	10.495	308071.47	171.33	212.54	51.400	63.761	0.163	3.813	regular
8th	8 th	142.536	87.133	0.002931	7.016	11.477	341180.48	172.42	220.52	51.725	66.157	0.128	3.864	regular
7th	7 th	135.101	77.499	0.002596	7.402	12.903	385208.01	172.78	228.13	51.834	68.438	0.105	3.941	regular
6th	6 th	123.968	67.252	0.002245	8.067	14.869	445434.30	173.08	234.99	51.924	70.497	0.072	3.917	regular
5th	5 th	109.479	56.428	0.00188	9.134	17.722	531914.89	173.25	241.32	51.974	72.395	0.045	3.888	regular
4th	4 th	91.973	45.223	0.001507	10.873	22.113	663570.01	173.23	247.04	51.969	74.113	0.02	3.841	regular
3rd	3 rd	71.683	33.954	0.001133	13.950	29.452	882612.53	173.11	251.53	51.934	75.460	0.026	3.636	regular
2nd	2 ND	48.865	23.133	0.000772	20.465	43.228	1295336.79	173.10	251.59	51.931	75.476	0.072	3.388	regular
1st	1 st	26.839	13.619	0.000446	37.259	73.427	2242152.47	174.75	245.31	52.424	73.593	0.018	2.341	regular
GF	GF	11.612	7.063	0.000184	86.118	141.583	5434782.61	195.92	251.21	58.777	75.364	0.39	0.466	regular
BASE 1	BASEMENT	0.831	1.232	0.000023	1203.369	811.688	43478260.87	231.44	190.08	69.432	57.024	0.109	0.172	regular

Table 4.2 Regularity in Plan of B+G+9 checking for structural eccentricity

The structure evidently fulfils all requirements for regularity in elevation stated in ES EN 1998:2015 in 4.2.3.3 provided that only the upper part of the structure (above the ground) is considered.

- All lateral resisting system (core, structural wall or frame) run without interruption
- Both lateral stiffness and the mass of individual story shall remain constant or reduce gradually without abrupt change.
- When set back present full fill setback requirement

For the case of studied topology all regularity in elevation criteria is checked and all criteria is fulfilled by all topology except 2B+G+18 that not fulfill regularity in mass there is abrupt change in mass when slab is changed from ribbed to solid.

4.2 Inter story drift consideration at damage limitation

4.2.1. Damage limitation requirement nonlinear static

The Ethiopian Seismic Code EBCS-8 defines two seismic ground motion levels. The first corresponds to a 10 % chance of being exceeded in 50 years (a 475-year return period), and for this level structures must be designed so that neither local nor overall collapse occurs, addressing the ultimate limit state. The second level corresponds to a 10 % chance of exceedance in 10 years (a 95-year return period), for which structures are required to have enough strength and stiffness to preserve critical functions without damage, with associated usage limits tied to the damage limit state. The damage limitation requirement is considered to have been satisfied if, under a

seismic action having a larger probability of occurrence than the design seismic action, it corresponds to the no collapse requirement. The inter-story drifts are limited in accordance with ES EN 1998; 2015. For this project work, use the value of reduction factor $v=0.5$ (for structure importance class II) for damage limitation requirement. The limitation of inter-story drift is $\Delta r \cdot v \leq 0.005 \cdot h$ for this limit state. Story drift (Δe) finds from the analysis result used to calculate the lateral displacement (Δs) with the behavior factor. i.e., $\Delta s = q \cdot \Delta e$, use $q = 3.0 \cdot a_u / a_1$, and (Δr) design inter-story drift.

Damage limit evaluated as the difference of the average lateral displacement's Δs at the top and bottom of the story under consideration. For all case study buildings, the story drift from analysis results and inter story drift computation using behavior factor are done. B+G+9 and 2B+G+18 presented below as a sample. lateral and rotational stability of the structure checking proses summaries in the table.

$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0.1$		Øx or Øy		< 0.1	ok!									
		Øx or Øy		> 0.1	p-delta effects should be consider									
		Øx or Øy		> 0.2	a rigorous second-order analysis is									
TABLE: Story Forces														
Story	Output Cas	Location	Drift X	Drift y	P kN	VX kN	VY kN	Inter-story	Inter-story	check in x	check in y	Damage Limitation v(x)<0.00v(x)<0.00		check in x
ROOF.F	COMB_ RSX_ 4+IMPX	Bottom	0.001065	0.000626	697.71	107.48	77.28	0.00230	0.00188	ok!	ok!	0.00053	0.00031	ok!
ROOF	COMB_ RSX_ 4+IMPX	Bottom	0.000851	0.000644	5,545.02	403.90	332.29	0.00389	0.00358	ok!	ok!	0.00043	0.00032	ok!
9TH	COMB_ RSX_ 4+IMPX	Bottom	0.00083	0.000705	15,567.39	843.40	828.50	0.00511	0.00442	ok!	ok!	0.00042	0.00035	ok!
8TH	COMB_ RSX_ 4+IMPX	Bottom	0.000933	0.000758	25,589.76	400.80	407.29	0.01986	0.01587	ok!	ok!	0.00047	0.00038	ok!
7TH	COMB_ RSX_ 4+IMPX	Bottom	0.001105	0.000802	35,612.12	1,488.95	1,147.46	0.00881	0.00830	ok!	ok!	0.00055	0.00040	ok!
6TH	COMB_ RSX_ 4+IMPX	Bottom	0.001242	0.000827	45,634.49	1,731.86	1,509.79	0.01091	0.00833	ok!	ok!	0.00062	0.00041	ok!
5TH	COMB_ RSX_ 4+IMPX	Bottom	0.001358	0.000827	55,656.86	1,961.73	1,638.49	0.01284	0.00936	ok!	ok!	0.00068	0.00041	ok!
4TH	COMB_ RSX_ 4+IMPX	Bottom	0.00146	0.000797	65,679.23	2,168.15	1,939.50	0.01474	0.00900	ok!	ok!	0.00073	0.00040	ok!
3RD	COMB_ RSX_ 4+IMPX	Bottom	0.001381	0.000797	75,701.57	2,334.39	2,182.28	0.01493	0.00922	ok!	ok!	0.00069	0.00040	ok!
2ND	COMB_ RSX_ 4+IMPX	Bottom	0.001006	0.000558	86,322.43	2,512.38	2,249.94	0.01152	0.00714	ok!	ok!	0.00050	0.00028	ok!
1ST	COMB_ RSX_ 4+IMPX	Bottom	0.00119	0.000492	100,126.38	2,779.58	3,040.69	0.01429	0.00540	ok!	ok!	0.00060	0.00025	ok!
GF	COMB_ RSX_ 4+IMPX	Bottom	0.000994	0.000875	118,651.22	4,479.30	5,511.35	0.00878	0.00628	ok!	ok!	0.00050	0.00044	ok!

Table 4.3 Analysis of B+G+9 story drift result used to compute inter story drift

TABLE: Damage Limitation Check									Check for p-delta		Damage Limitation			
Story	Input Ca	Drift X	Drift y	P kN	VX kN	VY kN	Inter- story	Inter- story	check in x	check in y	dr.v(x)<0.005h	dr.v(x)<0.005h	check in x	check in y
ROOF 1	C_RSY _6_IMP Y	0.004204	0.0225	53,710.07	4,103.72	83,072.71	0.01834	0.00485	ok!	ok!	0.00210	0.01125	ok!	ok!
ROOF	C_RSY _5_IMP Y	0.005592	0.024228	50,824.77	4,043.00	81,601.33	0.02343	0.00503	ok!	ok!	0.00280	0.01211	ok!	ok!
18th	C_RSY _5_IMP Y	0.005388	0.023406	54,260.26	3,734.39	76,414.45	0.02610	0.00554	ok!	ok!	0.00269	0.01170	ok!	ok!
17th	C_RSY _5_IMP Y	0.005023	0.021733	65,705.14	3,348.98	68,680.37	0.03285	0.00693	ok!	ok!	0.00251	0.01087	ok!	ok!
16th	C_RSY _5_IMP Y	0.004563	0.019304	77,130.18	2,910.20	59,491.14	0.04031	0.00834	ok!	ok!	0.00228	0.00965	ok!	ok!
15th	C_RSY _5_IMP Y	0.003996	0.016187	88,958.94	2,411.41	48,407.17	0.04914	0.00992	ok!	ok!	0.00200	0.00809	ok!	ok!
14th	C_RSY _5_IMP Y	0.003294	0.012462	100,794.10	1,855.80	35,889.63	0.05964	0.01167	ok!	ok!	0.00165	0.00623	ok!	ok!
13th	C_RSY _5_IMP Y	0.002555	0.008403	112,599.78	1,450.18	22,878.50	0.06613	0.01379	ok!	ok!	0.00128	0.00420	ok!	ok!
12th	C_RSY _5_IMP Y	0.00175	0.00411	126,899.70	1,117.74	12,587.03	0.06623	0.01381	ok!	ok!	0.00088	0.00206	ok!	ok!
11th	C_RSY _6_IMP Y	0.000975	0.002151	146,063.84	1,344.23	17,311.91	0.03531	0.00605	ok!	ok!	0.00049	0.00108	ok!	ok!
10th	C_RSY _5_IMP Y	0.001329	0.005605	165,227.97	1,929.75	31,662.78	0.03793	0.00975	ok!	ok!	0.00066	0.00280	ok!	ok!
9th	C_RSY _5_IMP Y	0.00223	0.009367	186,856.92	2,704.78	49,993.03	0.05135	0.01167	ok!	ok!	0.00112	0.00468	ok!	ok!
8th	C_RSY _5_IMP Y	0.003045	0.012921	209,159.47	3,417.36	67,239.18	0.06212	0.01340	ok!	ok!	0.00152	0.00646	ok!	ok!
7th	C_RSY _5_IMP Y	0.003837	0.015769	231,781.05	4,033.74	82,620.64	0.07349	0.01475	ok!	ok!	0.00192	0.00788	ok!	ok!
6th	C_RSY _5_IMP Y	0.004541	0.017988	254,205.87	4,540.35	95,592.80	0.08475	0.01594	ok!	ok!	0.00227	0.00899	ok!	ok!
5th	C_RSY _5_IMP Y	0.005013	0.019303	276,484.52	4,955.93	106,297.24	0.09322	0.01674	ok!	ok!	0.00251	0.00965	ok!	ok!
4th	C_RSY _5_IMP Y	0.005169	0.019514	293,712.08	5,306.69	114,934.64	0.09536	0.01662	ok!	ok!	0.00258	0.00976	ok!	ok!
3rd	C_RSY _5_IMP Y	0.00497	0.018563	321,871.49	5,612.55	121,680.94	0.09501	0.01637	ok!	ok!	0.00249	0.00928	ok!	ok!
2nd	C_RSY _5_IMP Y	0.004283	0.016203	346,474.73	5,872.24	126,715.66	0.08424	0.01477	ok!	ok!	0.00214	0.00810	ok!	ok!
1st	C_RSY _5_IMP Y	0.003287	0.010918	376,291.95	6,097.40	130,178.76	0.06762	0.01052	ok!	ok!	0.00164	0.00546	ok!	ok!
GF	C_RSX _1_IMP X	0.000131	0.002033	407,659.45	6,239.55	131,512.40	0.00285	0.00210	ok!	ok!	0.00007	0.00102	ok!	ok!
BASE 1	C_RSX _1_IMP X	0.000059	0.000922	435,429.35	6,347.50	132,411.83	0.00135	0.00101	ok!	ok!	0.00003	0.00046	ok!	ok!

Table 4.4 Analysis of 2B+G+18story drift result used to compute inter story drift

As you see in the above table, the damage limit should be less than or equal to 0.005H so that the required design inter-story drift limit is fulfilled. Hence, Tables 4.3 and 4.4 results indicate that the design inter-story drift would be less than the limit for all stories. The limit of inter-story drift for this case study building is 0.015 in all stories. This indicated that case study

buildings had been designed & constructed to withstand a seismic action having a larger probability of occurrence without damage or limitation of use. Similar results are found for the remaining two topology types.

4.3 Equivalent lateral forces analysis

Seismic analysis for many structures is still commonly performed using an equivalent lateral force approach, in which the complex dynamic earthquake effects are represented by simplified horizontal forces. In this method, the base shear—defined as the total horizontal force acting on the structure—is determined from the structural mass, fundamental vibration period, and the corresponding modal response. This base shear is then allocated along the height of the building as lateral forces in accordance with code-specified formulas. The approach is generally conservative and suitable for low- to mid-rise buildings with regular configurations.

4.3.1 Base shear comparison

Tables 4.5 and 4.6 highlight a significant increase in calculated base shear when using ES EN 1998:2015 compared to EBCS 8 1995, with values rising by 67% to 74%. This increase results from key updates in the new code, including changes to the ground type classification, peak ground acceleration, and the base shear formula, reflecting a stricter requirement for seismic resistance in the equivalent force method.

Base Shear Using EBCS 8			
INPUTS			
Type of the building	reinforced concrete moment resisting frames		
Structural Type	Frame system		
Nature of Structure	regular		
Height =	45	m	
ZONE	2		
SUB SOIL CLASS	B		
Importance category	II		
Ductility Class	DL"III"		
$C_1 = 0.075$	$\gamma_0 = 0.2$		
$\alpha_0 = 0.05$	$K_D = 1.5$		
$I = 1.2$	$K_W = 1$		
$S = 1.2$	$K_R = 1$		
$T_1 = 1.303$	$\gamma = 0.3$		
$\alpha = 0.06$	$\beta = 1.207022575$		
$S_d(T_1) = 0.0217$			
TOTAL WEIGHT (w) =	10,108,690.03	99,166,249.19	kg/N
$F_b = 2,154.53$	KN		
$F_t = 196.53$			

Base shear using ENSN			
Inputs			
Seismic Weigt of Building	W	21,107,400.76	Kg
Ground Types	B	>250	Cu(kpa)
Value of response spectra(Tpe 1)	S	1.20	
Value of response spectra(Tpe 1)	Tb(S)	0.15	
Value of response spectra(Tpe 1)	Tc(S)	0.50	
Value of response spectra(Tpe 1)	Td(S)	2.00	
Bade Rocke accleration Zone (4)	$\alpha_c = \alpha_g / \beta$	0.10	
Importance Factor	II	1.00	
Hight of building	H	65.50	m
	l_{vis}	5.50	m
	l_{visy}	8.40	m
Effective Ania of Shire wall	A_i	3.48	m ²
Total effective Area of shire wall	A_c	0.00	m ²
	C_t	0.11	km
Fundamental period	T_1	1.57	Sec
Outputs			
$F_b = S_d(T_1) \cdot m \cdot \lambda$			
Ordinate of Design Spectrum	$S_d(T_1)$	0.383168	sec
Behaviour Factor	q	3.12	m
Lower bound Factor	β	0.20	
Correction Factor	λ	1.00	
Base shire Force	F_b	12,689,586.79	N
	F_b	-	N

Table 4.5 Linear static analysis of base shear calculation procedures.

Topology Type	PRIOD USED	Linear static case	Base sher (KN)		Hight Of Building	Percent increased
	Building Mass(Kg)		EBCS 8	ESEN 1998:2015		
B+G+7	1.75	ELF	678.97	2,615.27	27.00	74%
	2,467,573.58	RS	432.06	1,333.86		68%
B+G+9	1.829	ELF	1,832.25	6,621.68	35.00	72%
	7,581,517.52	RS	1,288.99	3,910.01		67%
2B+G+12	1.944	ELF	2,154.53	7,334.8	45.00	71%
	10,108,690.03	RS	1,650.19	4,904.95		66%
2B+G+18	1.569	ELF	3,728.87	11,529.97	65.00	68%
	21,107,400.76	RS	3,974.87	12,689.59		69%

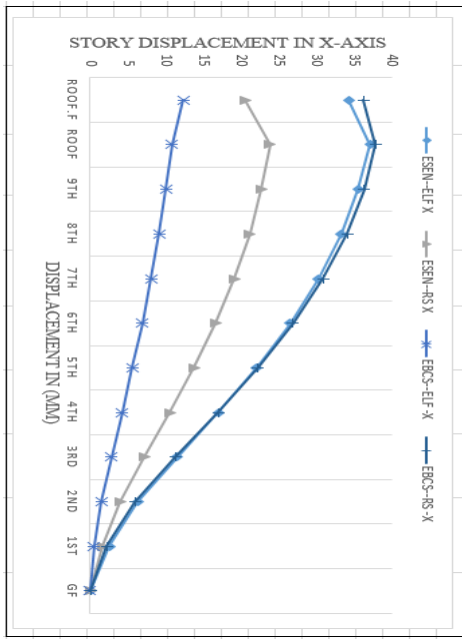
Table 4.6 Linear static analysis base shear result of the two codes

4.3.2 Story Displacement Comparison

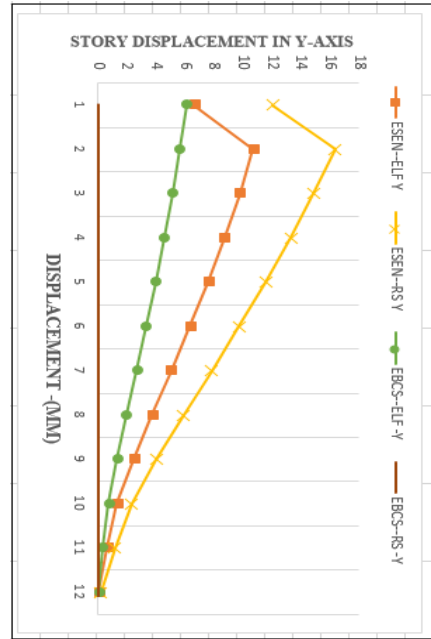
A comparison of the story displacements resulting from the two codes is provided in Figure 4.3, with the top story displacement detailed in Table 4.7. As a consequence of the differing base shear values, the story displacements calculated by each code also show significant variation. As shown in the Table 4.7 and Figure 4.3 below the maximum displacement found in top of the story but in 2B+G+18 maximum displacement not found at the top. From the result we cannot decide the governing methodology for story displacement we must check all cases to get the maximum story displacement.

Topology Type	Linear static case	Top Story Displacement (mm)				Percent increased		Hight Of Building
		EBCS 8		ESEN 1998:2015				
		X	Y	X	Y	X	Y	
B+G+7	ELF	10.52	31.11	29.46	36.29	64%	17%	27.00
	RS	34.10	25.35	18.99	23.20	-80%	-8%	
B+G+9	ELF	12.424	6.086	34.27	6.64	64%	9%	35.00
	RS	36.285	29.747	20.578	12.048	-76%	-59%	
2B+G+12	ELF	18.76	24.08	58.4	27.3	68%	14%	45.00
	RS	40.01	32.49	25.99	19.53	-54%	-40%	
2B+G+18	ELF	5.111	3.409	4.843	3.621	6%	-6%	65.00
	RS	10.447	10.002	23.994	70.954	86%	56%	
		26.994	15.092	26.348	18.643	19%	-2%	
		62.284	46.195	108.34	447.095	90%	43%	

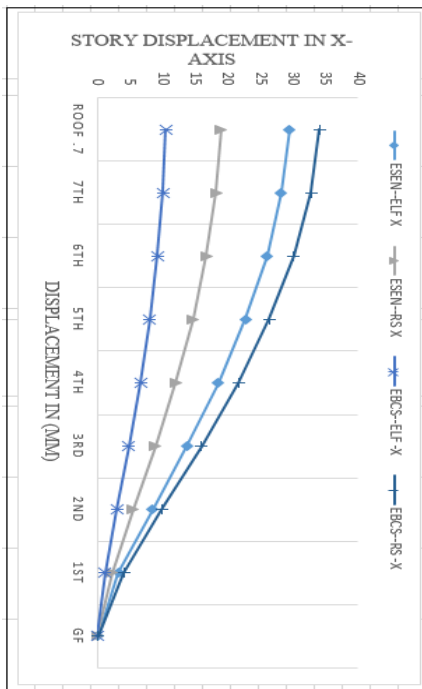
Table 4.7 Linear static analysis Top story displacement result of the two codes



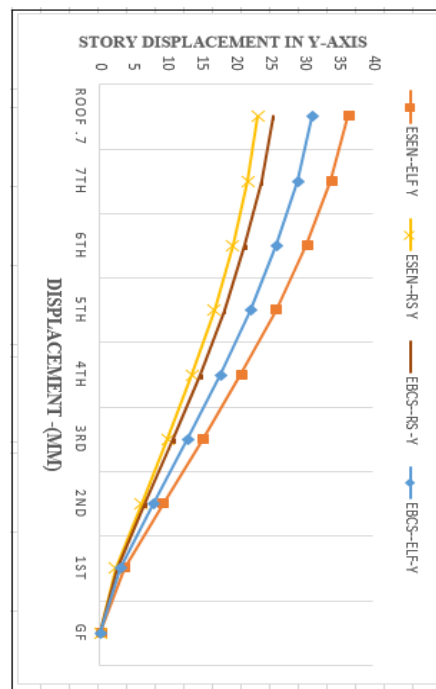
A



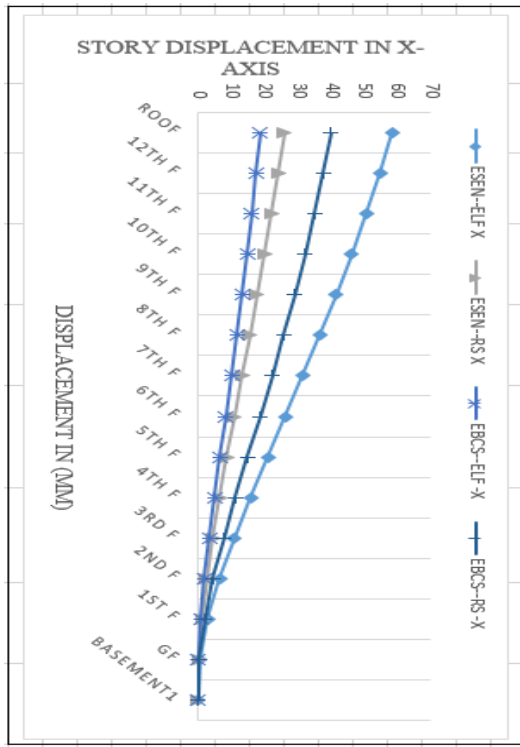
B



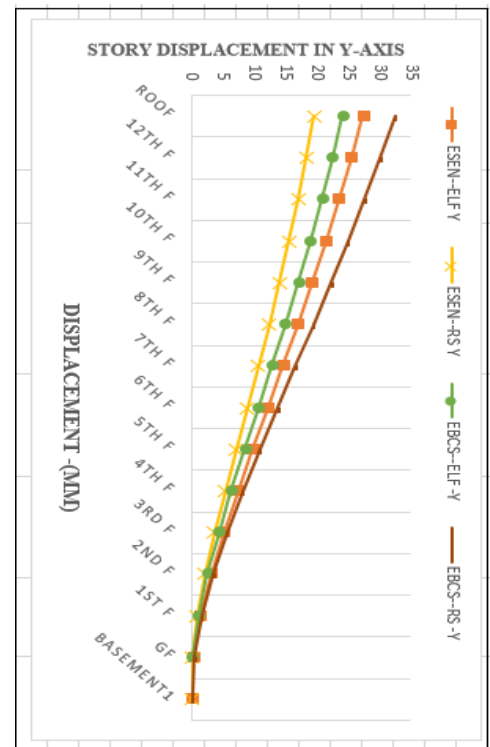
C



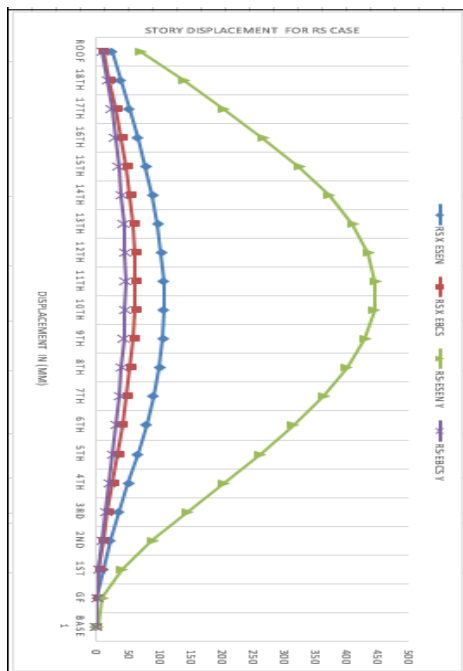
D



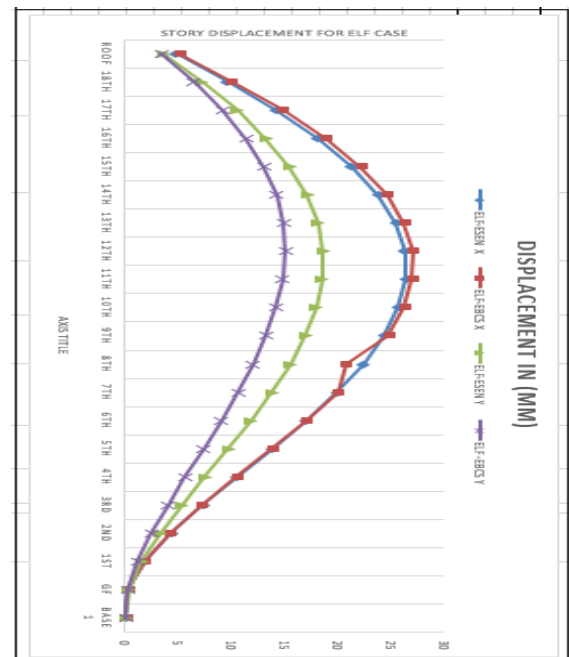
E



F



G



H

Figure 4.3 Equivalent lateral force and response spectrum analysis of story displacement of all topologies

Similarly, we can assess story drift like base shear and story displacement as you can see below in figure 4.4. The graph for sample of topology B+G+7, but the result is similar pattern of the above two cases, Notable difference in base shear, story displacement, and story drift are often found through equivalent lateral force analysis.

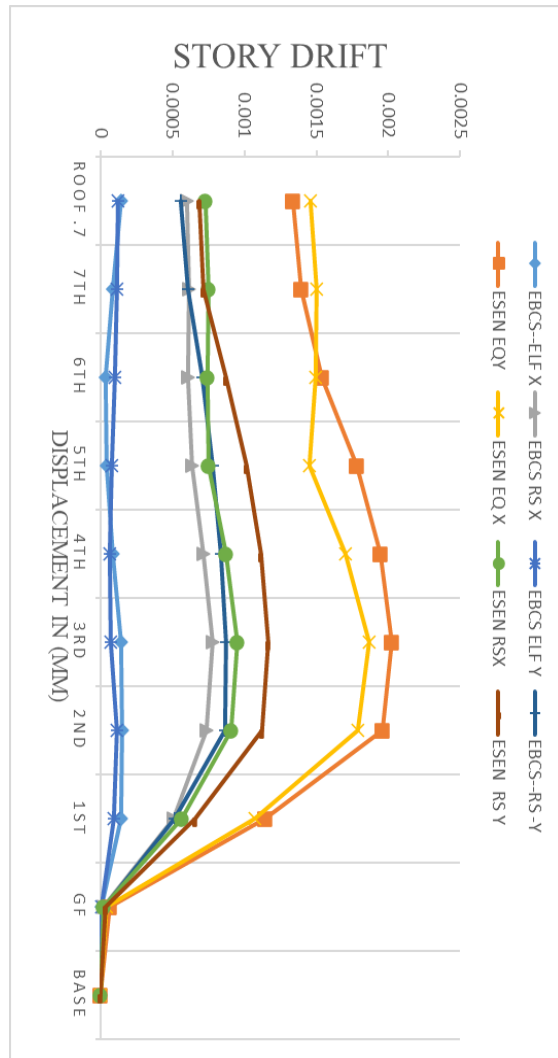


Figure 4.4 Equivalent lateral force and response spectrum analysis of story drift

4.4 Safety compromising based on equivalent lateral force analysis (ELFA)

The existing building evaluation according to equivalent static analysis method using EBCS 8 1995 and we use ES EN 1998:2015 to check the capacity ratio of columns and beams. The column capacity ratio (PMM) becomes an indication of the existing columns capacity to resistor or not the applied internal forces which have been created by external loads.

4.4.1 Demand to capacity Ratio of columns based on ELFA

Once the geometry and layout of the reinforcement in a concrete column section are specified, the software is able to evaluate the column's capacity. Alternatively, it can determine the required amount of reinforcement for column design based on an assumed bar arrangement. Capacity checks or reinforcement calculations are performed at a user-defined number of sections along the column height. The reinforced concrete column design process includes generating axial force–biaxial moment interaction surfaces for all concrete section types in the model, and then computing either the capacity ratio or the required reinforcement area corresponding to the factored axial loads and biaxial (or uniaxial) bending moments from each load combination at each output station. The procedure for designing column shear reinforcement is largely similar to that used for beams, with the additional consideration of the influence of axial force on the concrete's shear resistance.

In short as to evaluate the load condition of a column, a capacity ratio is determined. This factor provides an indication of the column's loading relative to its full capacity, calculated by comparing the amount of steel reinforcement needed to the amount actually provided. From 27 column and 12 columns type we take 4 representative sample design result from columns presented as follow

Story	Column x-section (mm) C1	Area of Reinforcement of column As(mm)	Demand from analysis result			As(mm)	Demand Capacity Ratio (PMM Ratio)	Column x-section (mm) of C2	Area of Reinforcement of column	Demand from analysis result				Demand Capacity Ratio (PMM)
			NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)					NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)		
1	700x350	2512.00	2021.65	40.43	-43.48	3000	1.19	450x450	3768.00	1821.11	-64.33	58.83	4900.00	1.30
2	700x350	2512.00	1798.09	-35.96	35.96	3000	1.19	450x450	3768.00	1698.04	-68.71	-52.34	4900.00	1.30
3	700x350	2512.00	1500.99	-36.29	30.19	3000	1.19	450x450	3768.00	1440.76	-60.05	47.37	4900.00	1.30
4	700x350	2512.00	1263.71	-37.88	25.27	3000	1.19	450x450	3768.00	1363.34	57.87	38.44	4900.00	1.30
5	700x350	2512.00	1027.05	-37.84	20.27	3000	1.19	450x450	3768.00	1372.05	-50.21	-39.21	4900.00	1.30
6	700x350	2512.00	790.93	-36.99	-15.81	3000	1.19	450x450	3768.00	1222.13	-46.91	31.65	4900.00	1.30
7	700x350	2512.00	555.25	-34.72	-11.85	2970	1.18	450x450	3768.00	1143.23	-44.02	-29.31	3600.00	0.96
8	700x350	2512.00	319.85	-31.38	14.44	2970	1.18	450x450	3768.00	867.75	-39.89	27.21	3600.00	0.96
9	700x350	2512.00	83.47	-28.01	-34.99	2720	1.08	450x450	3768.00	198.76	-38.01	-34.09	3600.00	0.96
Story	Column x-section (mm) of C3	Area of Reinforcement of column As(mm)	Demand from analysis result				Demand Capacity Ratio (PMM Ratio)	Column x-section (mm) of C4	Area of Reinforcement of column	Demand from analysis result				Demand Capacity Ratio (PMM)
			NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)					NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)		
1	800x800	3768.00	2284.93	-74.49	66.949	5700.00	1.513	800x800	3768.00	2433.5	-82.76	72.24	5700.00	1.51
2	800x800	3768.00	2005.87	-79.57	-59.563	5700.00	1.513	800x800	3768.00	2136.3	-88.40	-64.27	5700.00	1.51
3	600x600	3768.00	1680.21	-69.54	53.907	5700.00	1.513	800x800	3768.00	1789.4	-77.26	58.17	5700.00	1.51
4	600x600	3768.00	1418.08	67.01	43.745	5700.00	1.513	800x800	3768.00	1510.3	74.45	47.20	5700.00	1.51
5	500x500	3768.00	1157.84	-58.14	-44.621	4900.00	1.300	600x600	3768.00	1233.1	-64.60	-48.15	4900.00	1.30
6	450x450	3768.00	897.26	-54.32	36.018	4900.00	1.300	600x600	3768.00	955.6	-60.35	38.86	4900.00	1.30
7	450x450	3768.00	637.37	-50.98	-33.355	4900.00	1.300	500x500	3768.00	678.8	-56.63	-35.99	4900.00	1.30
8	400x400	2512.00	378.22	-46.19	30.965	3600.00	1.433	450x450	3768.00	402.8	-51.32	33.41	4900.00	1.30
9	400x400	1607.68	118.4	-44.02	-38.794	3600.00	2.239	450x450	2512.00	126.1	-48.90	-41.86	4900.00	1.95

Table 4.8 Column demand to capacity ratio for B+G+7 topology type

In the case study building B+G+7& B+G+9, 4(C1, C2, C3 and C4) columns were selected to assess the performance of seismic load. The selected columns are internal and on the axis of long span, assumed to represent the others. The linear static analysis result shown in Table 4.8, except at top story of C2, the other floor columns capacity was found to be inadequate to the earth quake load. Similarly, Table 4.9 tells us C1 and C2 adequate up to 5th story and in adequate above but C3 and C4 fully inadequate. In the case study building 2B+G+12& 2B+G+18, in the design procedure most column section is in adequate so no need of detail capacity demand ratio calculation. Adequate column capacity would mean that the column demand interaction diagram is within the column capacity interaction diagram. In addition, it also signifies that column capacity can hold the demand loads and vice versa. This means the column forms hinge or shear failure due to this new lateral earthquake load effect. The degree and mechanism of failure will be further investigated next with nonlinear analysis of performance evaluation.

Story	Column x-section (mm) C1(32pic)	Area of Longitudinal Reinforcement of	Demand from analysis result			As(mm)	Demand Capacity Ratio (PMM Ratio)	Column x-section (mm) of C2 (16 pic)	Area of Longitudinal Reinforcement of	Demand from analysis result			As(mm)	Demand Capacity Ratio
			NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)					NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)		
1	600x500	3768.00	2895.54	57.91	-57.91	3000	0.80	600x500	5024.00	3151.21	63.02	63.02	3000.00	0.60
2	600x500	3768.00	2590.24	51.80	-51.80	3000	0.80	600x500	5024.00	2726.43	-54.52	54.52	3000.00	0.60
3	600x500	3768.00	2245.61	44.91	44.91	3000	0.80	600x500	5024.00	295.42	47.91	-47.91	3000.00	0.60
4	600x500	3768.00	1969.87	-39.39	39.39	3000	0.80	600x500	3768.00	2096.07	41.92	41.92	3000.00	0.80
5	600x500	3768.00	1694.88	33.89	-33.89	3000	0.80	600x500	3768.00	1799.41	35.99	-35.99	3000.00	0.80
6	500x500	2009.60	1420.49	-28.41	-28.41	2500	1.24	600x500	2009.60	1504.61	30.09	-30.09	3000.00	1.49
7	500x500	2009.60	1146.73	22.93	22.93	2500	1.24	600x500	2009.60	1211.34	-26.67	-24.23	3000.00	1.49
8	500x500	2009.60	873.55	-17.47	19.03	2500	1.24	600x500	2009.60	919.32	-27.11	-18.38	3000.00	1.49
9	500x500	2009.60	600.91	-12.02	16.87	2500	1.24	600x500	2009.60	628.45	-26.01	12.57	3000.00	1.49
10	500x500	2009.60	328.6	-10.44	16.72	2500	1.24	600x500	2009.60	338.62	21.36	14.69	3000.00	1.49
11	500x500	2009.60	54.36	-16.58	-10.33	2500	1.24	600x500	2009.60	46.89	-19.44	-11.57	3000.00	1.49

Story	Column x-section (mm) of C3(4 pic)	Area of Reinforcement of column As(mm)	Demand from analysis result			As(mm)	Demand Capacity Ratio (PMM Ratio)	Column x-section (mm) of C4 (6 pic)	Area of Reinforcement of column	Demand from analysis result			As(mm)	Demand Capacity Ratio
			NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)					NEd (kN)	MEd3 (kN.m)	MEd2 (kN.m)		
1	600x500	2009.60	1683.19	47.43	33.66	3000.00	1.493	500x500	2009.60	531.0	72.95	10.62	2500.00	1.24
2	600x500	2009.60	1498.94	50.45	-29.98	3000.00	1.493	500x500	2009.60	299.9	20.04	30.36	2500.00	1.24
3	600x500	2009.60	1350.36	46.06	-27.01	3000.00	1.493	500x500	2009.60	160.8	-30.75	39.15	2500.00	1.24
4	600x500	2009.60	1202.00	52.59	-24.04	3000.00	1.493							
5	600x500	2009.60	1044.34	49.74	-20.88	3000.00	1.493							
6	600x500	2009.60	891.71	47.48	-17.83	3000.00	1.493							
7	600x500	2009.60	743.07	45.77	-15.07	3000.00	1.493							
8	600x500	2009.60	597.80	44.14	-14.64	3000.00	1.493							
9	600x500	2009.60	456.76	41.21	-14.64	3000.00	1.493							
10	600x500	2009.60	325.45	44.78	-19.49	3000.00	1.493							
11	600x500	2009.60	172.66	24.74	28.41	3000.00	1.493							

Table 4.9 Column demand to capacity ratio for B+G+9 topology type

In the case study building 2B+G+12& 2B+G+18, follow similar procedure but to simplify using section designer of ETABS simply check the design and get similar result in the above to topology.

Story	Column x-section (mm) C5(2pic)	Area of Longitudnal Reinforcemet	Demand from analysis result			As(mm)	Demand Capacity Ratio	Column x-section (mm) of C6 (8 pic)	Area of Longitudnal Reinforceme	Demand from analysis result			As(mm)	Demand Capacity Ratio
			N _{Ed} (kN)	M _{Ed3} (kN.m)	M _{Ed2} (kN.m)					N _{Ed} (kN)	M _{Ed3} (kN.m)	M _{Ed2} (kN.m)		
1	600x500	7234.56	3686.25	110.58	110.58	8100	1.12	700x700	6832.64	4984.06	166.13	166.13	10000.00	1.46
2	600x500	7234.56	3430.34	-97.19	-135.04	7225	1.00	700x700	6832.64	4658.84	-147.53	-184.97	9025.00	1.32
3	600x500	5024.00	3194.18	-85.18	85.18	6400	1.27	600x600	6832.64	4355.33	137.91	137.91	9025.00	1.32
4	600x500	5024.00	2939.64	-73.49	-73.49	5625	1.12	600x600	6832.64	3933.35	-1180	-1180	8100.00	1.19
5	600x500	5024.00	2685.36	-67.13	67.13	5625	1.12	600x600	6280.00	3514.71	99.5	-99.5	7115.00	1.13
6	600x400	3215.36	2413.2	56.31	-56.31	4900	1.52	600x500	6280.00	3144.54	83.83	-83.85	6400.00	1.02
7	600x400	3215.36	2139.91	-49.93	67.02	4900	1.52	600x500	6280.00	2778.2	-69.45	-69.45	5625.00	0.90
8	600x400	3215.36	1875.10	-37.5	58.14	3600	1.12	600x400	6280.00	2415.06	56.35	-65.68	4900.00	0.78
9	600x400	3215.36	1610.69	32.21	48.86	3600	1.12	600x400	3215.36	2060.86	-48.08	4.08	4900.00	1.52
10	600x400	3215.36	1346.98	41.18	74.95	3000	0.93	600x400	3215.36	1711.15	-39.25	-67.81	3600.00	1.12
11	600x400	3215.36	1083.41	29.2	34.34	2400	0.75	600x400	3215.36	1365.56	-27.27	-49.16	3000.00	0.93
12	600x400	3215.36	823.76	28.61	43.09	2400	0.75	600x400	3215.36	1022.76	-28.31	-41.99	2400.00	0.75
13	600x400	3215.36	546.73	25.71	40.4	2400	0.75	600x400	3215.36	686.58	-24.08	-41.29	2400.00	0.75
14	600x400	3215.36	306.46	24.41	36.21	2400	0.75	600x400	3215.36	353.99	-25.91	-37.75	2400.00	0.75
15	600x400	3215.36	455.52	20.26	-52.86	2400	0.75	600x400	3215.36	13.06	26.85	-51.07	2400.00	0.75

Story	Column x-section (mm) of C7(2 pic)	Area of Reinforcemet of column	Demand from analysis result			As(mm)	Demand Capacity Ratio	Column x-section (mm) of C9(8 pic)	Area of Reinforceme t of column	Demand from analysis result			As(mm)	Demand Capacity Ratio
			N _{Ed} (kN)	M _{Ed3} (kN.m)	M _{Ed2} (kN.m)					N _{Ed} (kN)	M _{Ed3} (kN.m)	M _{Ed2} (kN.m)		
1	700x700	5024.00	3862.25	-109.43	-109.43	7225.00	1.438	600x500	5024.00	4009.1	120.27	120.27	8100.00	1.61
2	700x700	5024.00	3600.52	96.01	-169.01	6400.00	1.274	600x500	5024.00	3794.9	-107.52	111.27	7225.00	1.44
3	600x600	5024.00	3331.85	88.85	-88.85	6400.00	1.274	600x500	3215.36	3587.7	-101.65	101.65	7225.00	2.25
4	600x600	5024.00	3050.16	-82.61	-76.25	5625.00	1.120	600x500	3215.36	3297.5	87.93	-87.93	6400.00	1.99
5	600x500	5024.00	2764.02	-78.20	64.49	4900.00	0.975	600x500	3215.36	3009.5	-75.24	-75.24	5625.00	1.75
6	600x400	3215.36	2448.9	-78.36	-57.14	4900.00	1.524	600x500	3215.36	2704.7	-67.62	67.62	5625.00	1.75
7	600x400	6280.00	2135.37	-94.88	61.85	4900.00	0.780	600x500	3215.36	2402.7	56.06	-56.06	4900.00	1.52
8	600x400	3215.36	131.80	-91.60	55.47	3600.00	1.120	600x500	3215.36	2099.9	-48.99	51.77	4900.00	1.52
9	600x400	3215.36	1539.73	91.43	56.44	3000.00	0.933	600x500	3215.36	1801.2	36.02	46.93	3600.00	1.12
10	600x400	3215.36	163.14	-69.19	49.95	2400.00	0.746	600x500	3215.36	1508.6	30.17	42.48	3000.00	0.93
11	600x400	3215.36	998.12	-71.96	48.74	2400.00	0.746	600x500	3215.36	1217.9	-24.36	41.48	3000.00	0.93
12	600x400	3215.36	742.46	-71.09	46.47	2400.00	0.746	600x500	3215.36	927.8	-18.55	40.97	3000.00	0.93
13	600x400	3215.36	386.91	-68.97	44.01	2400.00	0.746	600x500	3215.36	63.8	-16.03	39.52	3000.00	0.93
14	600x400	3215.36	191.46	-68.11	39.25	2400.00	0.746	600x500	3215.36	348.7	-18.60	35.33	3000.00	0.93
15	600x400	3215.36	11.83	-121.34	14.04	2400.00	0.746	600x500	3215.36	30.2	22.31	47.12	3000.00	0.93

Table 4.10 Column demand to capacity ratio for B+G+12 topology type

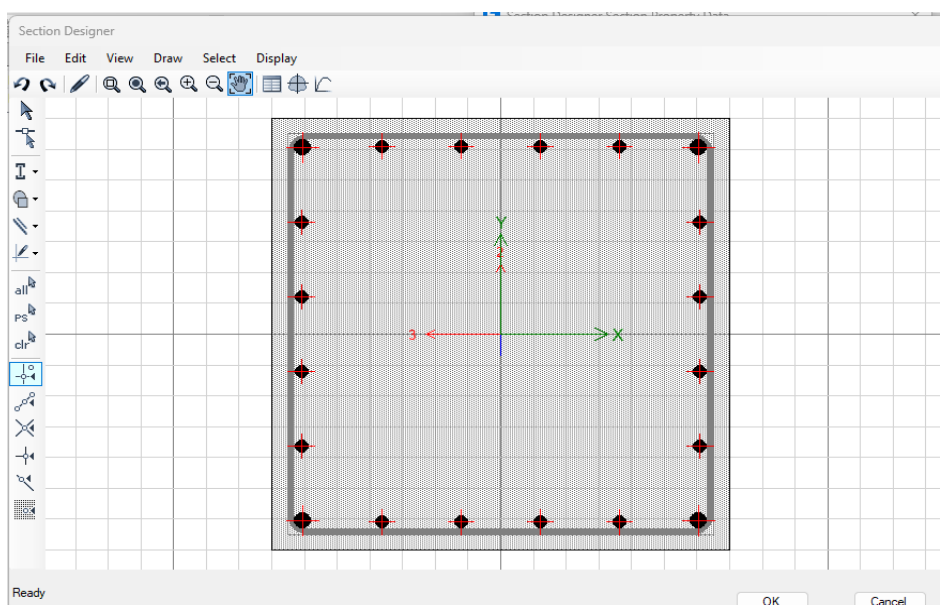


Figure 4.5 Section Designer Modeling of Existing Structure

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the beam section. Bottom steel is the result of positive occasions. In these situations, the beam can be shaped like a T-beam or a rectangle. Top steel is the result of negative situations. The beam is always designed as a rectangular portion in these situations [1]. For this project work, the major direction flexure moment was used to check the cross-sectional capacity of the beams. It also compared required reinforcements from the design result and from provided reinforcements to the existing buildings for each support: negative (top), positive (bottom), and span reinforcements. The compared results of the beams are presented in the table below.

Axis	Story	Beam section	Location	Provided Rainforcement		Reuired Rainforcement		Required/Provided	
				As Top (mm2)	As Bottom (mm2)	As Top (mm2)	As Bottom (mm2)	Top	Bottom
"@B-(2-3)"	"3-7"	500X300	END I	602.88	803.84	1215	944	2.015	1.174
			MIDDLE	401.92	803.84	87	517	0.216	0.643
			END J	602.88	803.84	677	942	1.123	1.172
"@B-(3-4)"	"3-7"	500X300	END I	602.88	803.84	1307	808	2.168	1.005
			MIDDLE	401.92	803.84	2	916	0.005	1.140
			END J	602.88	803.84	959	1001	1.591	1.245
"@1-(C-D)"	"2-4"	500X300	END I	1256	1256	1002	1141	0.798	0.908
			MIDDLE	628	1256	644	1478	1.025	1.177
			END J	942	1256	644	1301	0.684	1.036
"@1-(D-E)"	"2-4"	500X300	END I	942	1256	1033	1283	1.097	1.021
			MIDDLE	628	1256	644	1465	1.025	1.166
			END J	1256	1256	877	907	0.698	0.722
"@A-(4-3)"	"9"	400X250	END I	602.88	602.88	1298	702	2.153	1.164
			MIDDLE	602.88	602.88	368	668	0.610	1.108
			END J	602.88	602.88	1051	801	1.743	1.329
"@B-(3-2)"	"9"	400X250	END I	602.88	602.88	1230	653	2.040	1.083
			MIDDLE	602.88	602.88	86	688	0.143	1.141
			END J	602.88	602.88	693	641	1.149	1.063

Table 4.11 Beam demand to capacity ratio for B+G+7 topology type

Axis	Story	Beam section	Location	Provided Reinforcement		Reuired Reinforcement		Required/Provided	
				As Top (mm2)	As Bottom (mm2)	As Top (mm2)	As Bottom (mm2)	Top	Bottom
"@C&F-(6-7)"	"1-12"	500X250	END I	1570	942	2353	924	1.499	0.981
			MIDDLE	628	942	737	1149	1.174	1.220
			END J	1884	942	2029	924	1.077	0.981
"@C&F -(7-8)"	"1-12"	500X250	END I	1884	1570	2426	1346	1.288	0.857
			MIDDLE	628	1570	1000	1636	1.592	1.042
			END J	2512	1570	2700	1295	1.075	0.825
"@5-(A-B)"	"1-12"	500X250	END I	942	602.88	1003	1306	1.065	2.166
			MIDDLE	628	602.88	787	515	1.253	0.854
			END J	942	602.88	1297	1018	1.377	1.689
"@5-(C-D)"	"1-12"	500X250	END I	942	942	2243	1487	2.381	1.579
			MIDDLE	628	942	747	1030	1.189	1.093
			END J	1884	942	1003	1391	0.532	1.477
"@7-(C-D)"	"1-12"	500X250	END I	1004.8	602.88	1472	936	1.465	1.553
			MIDDLE	401.92	602.88	468	594	1.164	0.985
			END J	803.84	602.88	989	1081	1.230	1.793
"@D-(7-8)"	"1-12"	500X250	END I	45216	1004.8	1503	1028	0.033	1.023
			MIDDLE	628	1004.8	647	978	1.030	0.973
			END J	1256	1004.8	1805	819	1.437	0.815

Table 4.12 Beam demand to capacity ratio for 2B+G+12 topology type

The case study building of beam design results of building B+G+7 is shown in Table 4.11 the beam has been selected from axis B,1 and A and building 2B+G+12 is shown in Table 4.12 the beam has been selected from axis C&F,5,7 and D. The axis's is selected because it is an interior beam which carries large span loads and riptide in many stories. The ratio of the required to provided reinforcement shows whether the capacity of the existing reinforcement in the beam is adequate or not. If the ratio is less than one, it is possible to say that the beam has adequate reinforcement. On the contrary, if the ratio is greater than one, it is possible to say that the beam has less reinforcement. Thus, the sample beam result showed that the ratio of required to provided reinforcement is greater than one in most of the beam. This implies that the beam has inadequate reinforcement in most typical floor levels, which underlies the high vulnerability of the beam to form hinges followed by failure at all supports due to lateral loads. In the case study building 2B+G+18, in the design procedure most beam section is in adequate so no need of detail capacity demand ratio calculation. In general, the beam demand to capacity (required/provided) ratio is higher, which might be a cause for future premature failure.

4.5. Nonlinear static pushover analysis

Nonlinear analysis has been performed using structural analysis program ETABS 22.0 (cracked). The nonlinear behavior occurs in discrete user-defined hinges. Hinges can be introduced into both frame and vertical wall objects. Hinges may be assigned at any location along the frame object

but are restricted to mid-height in the wall object. Uncoupled moment, torsion, axial force, and shear hinges are available. There is also a coupled P-M2-M3 hinge whose yield is based on the interaction of axial force and bending moments at the hinge location. More than one type of hinge can exist at the same location; for example, both an M 3 (moment) and a V 2 (shear) hinge may be assigned to the same end of a frame object [12].

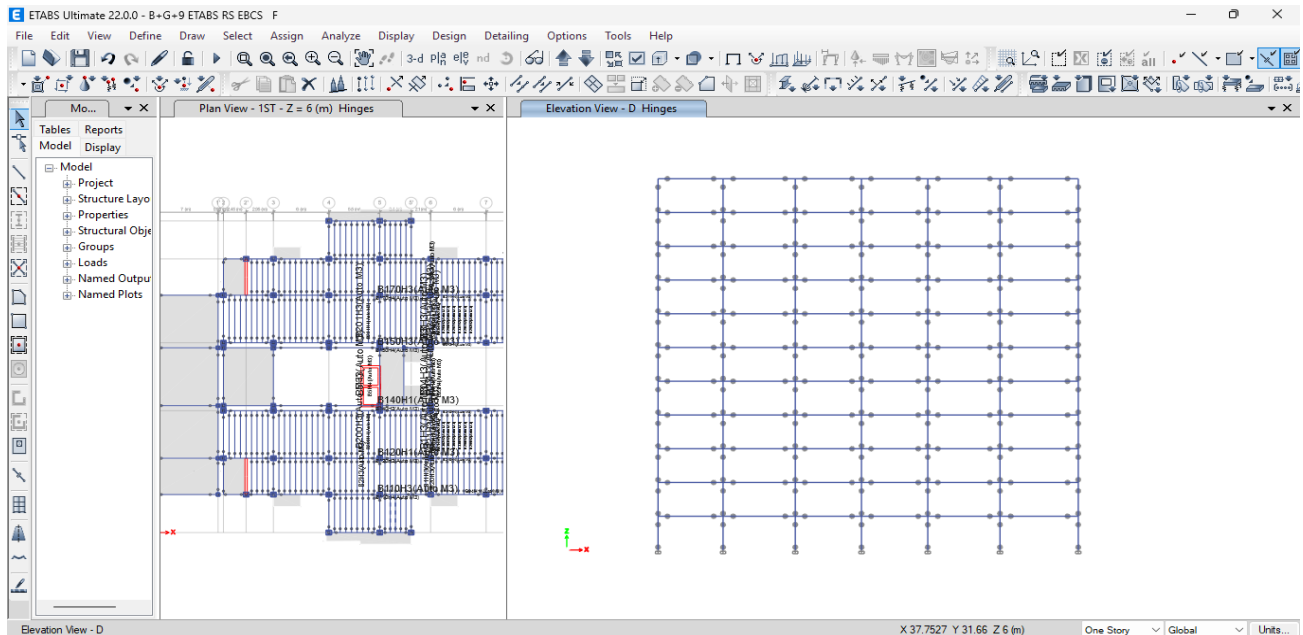


Figure 4.7 Hinge assignment in the beams

Multiple pushover load cases may be included in a pushover analysis. The structure may experience a varied load distribution in each pushover load scenario. Three pushover load situations, for instance, could be included in a standard pushover study. The structure would be subjected to gravity load in the first, one distribution of lateral load over its height in the second, and another distribution of lateral load over its height in the third. For a pushover load scenario, the distribution of load on the structure can be described using four different techniques [16].

- 1) Uniform Acceleration: A lateral force proportional to the nodal mass is applied at every node.
- 2) Modal Distribution: Lateral forces are based on a user-specified mode shape, calculated as the product of the mode shape, its circular frequency squared (ω^2), and the nodal mass.

- 3) User-Defined Pattern: Any arbitrary static load distribution can be specified. These methods can be used individually or combined. The analysis then provides various results on the structure's nonlinear behavior.

A plot of the base shear versus displacement at a control node.

- I. The same relationship plotted in ADRS format (spectral acceleration vs. spectral displacement), with the option to overlay demand spectra.
- II. A graphical, step-by-step visualization of both the sequence of plastic hinge formation (with color-coded states) and the evolution of member forces.
- III. Tabular data that can be viewed, printed, or exported. This includes the pushover curve coordinates (base shear vs. displacement), hinge progression statistics, and capacity spectrum parameters (ADRS curves, effective period, and damping).

4.6 Analysis results and discussion for nonlinear static pushover analysis

4.6.1 The general form of those case study buildings

Case study buildings B+G+7 and B+G+9 were selected for farther study by nonlinear static pushover analysis method. Because case study B+G+7 and B+G+9 is selected to simplify the discussion part explicitly and there is a similarity result in demand to capacity ratio evaluation. Nonlinear static pushover analysis shows the results through graphical and tabular form. This gives a better chance to compare the results and to evaluates the vulnerability of buildings at different performance level. For this project capacity curve, demands work-capacity curve (performance point) and plastic hinge formation through color points with graphical display have been evaluated below.

4.6.2 Capacity curve

The capacity (pushover) curve is one of the graphical displays of building capacity through base shear force in the vertical axis to target displacement (roof displacement) in the horizontal axis. It is a global response of a structural behavior. For each pushover step, the value of force and displacement increases up to the maximum capacity of the structure. After this point the base shear force decreases with increasing displacement if the structure has a ductile nature. But the structure has a brittle nature, the base shear force decreasing with decreasing displacement.

4.6.2.1. Capacity curve of case study buildings for push-x and push-y

The two resulting capacity curves for case study buildings B+G+7 and B+G+9 reinforced concrete buildings for Push X and for Push Y, the nonlinear static pushover analysis result, were plotted in Figures 4.8, 4.9, 4.10, and 4.11, respectively. The capacity curves of each case study building for push x and y show some difference. The capacity curve for case B+G+7 is initially linear but starts to deviate from linearity as the frames undergo inelastic deformation during the analysis in both directions. From figures 4.8 and 4.9, it is observed that the capacity curve of the case B+G+7 building in push-X and push-Y shows that the structure does demonstrate low deformation after the yield point, which means that the building's structural response is not similar to that of the behavior of earthquake-resistant buildings. From Figures 4.8 and 4.9, it is observed that the maximum values of the base shear and the monitor displacement capacity of the selected case B+G+7 building in the x and y directions were (11434.18 KN, -241.41 mm) and (10970.28 KN, -350.17 mm), respectively.

For the case of the case study building B+G+9, it is perfectly initially linear and starts to slightly deviate from linearity as the frames undergo inelastic deformation during the analysis in both directions. From figures 4.10 and 4.11, it is observed that in the case of the B+G+9 building during the pushover analysis, the frame undergoes inelastic deformation. When comparing with the B+G+7 case, B+G+9 has better performance for earth quick, from figures 4.10 and 4.11, it is observed that the maximum values of the base shear and the monitor displacement capacity of the selected case B+G+9 building in the x and y directions are (24108.60 KN, 98.08 mm) and (14160.21 KN, 29.39 mm), which have low displacement by very big base shear.

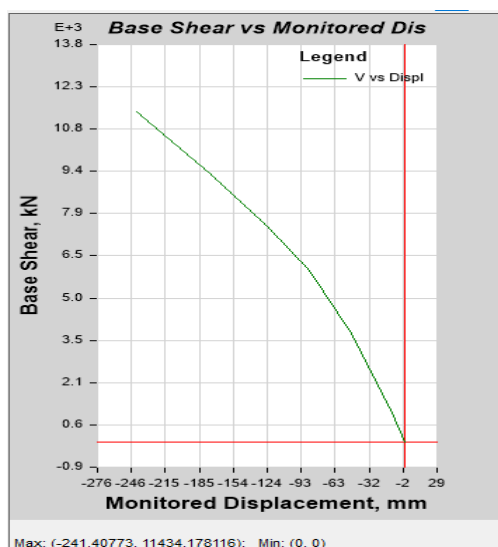


Figure 4.8 Capacity curve of B+G+7 for push-x

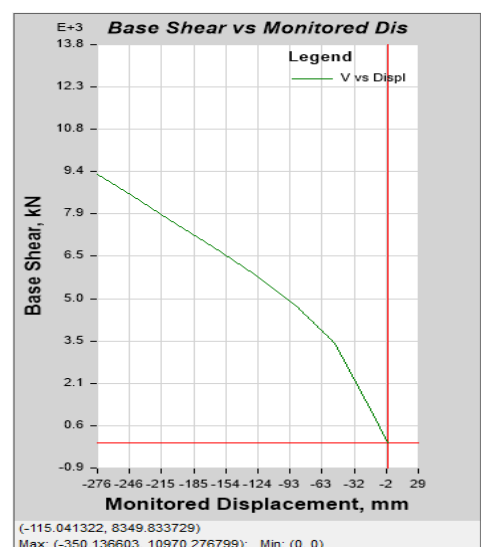


Figure 4.9 Capacity curve of B+G+7 for push-y

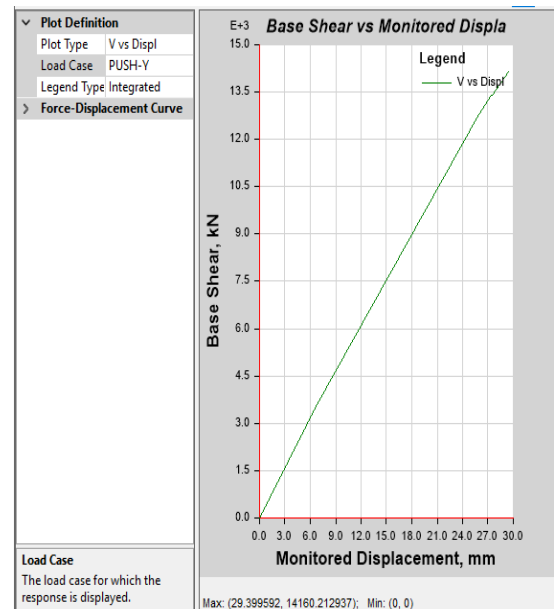
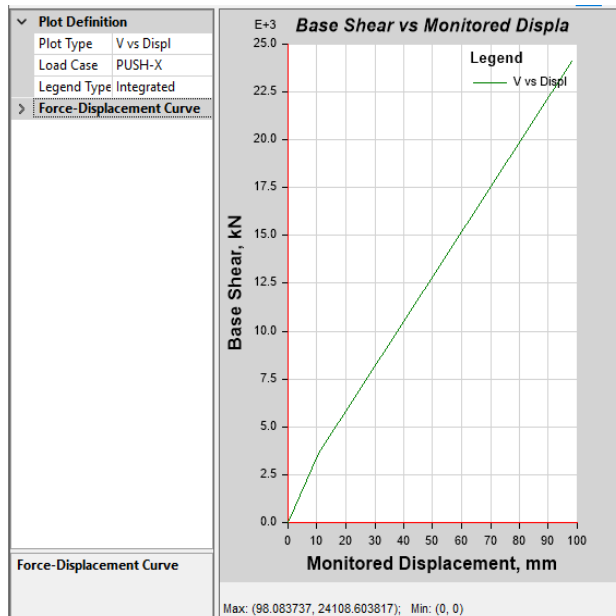


Figure 4.10 Capacity curve of B+G+9 for push-x Figure 4.11 Capacity curve of B+G+9 for push-y

4.7. Performance Point Analysis.

A graphical representation of the performance point that shows the structural damage from elastic range to near collapse is the demand to capacity interaction curve. In terms of response demand versus capacity, it is the primary result of a pushover analysis. The structure has good resistance if the demand curve crosses the capacity envelope close to the elastic range. The structure will behave poorly in response to the imposed seismic excitation and must be retrofitted to prevent future significant damage or collapse if the demand curve intersects the capacity curve with little reserve of strength and deformation capacity. The structure may be unstable for this earthquake lateral load or collapse completely if the demand curve does not intersect the capacity curve or if the intersection is outside the collapse limit.

“FEMA 440 defines the performance point as the maximum structural displacement expected for the demand ground motion due to an earth quake. The seismic demand is given by the amount of displacement at the performance point in the capacity spectrum method of FEMA 440. Member displacement and forces are computed at this displacement and are used as the inelastic demand and checked against the available capacities of members. The performance point is one of the fundamental goals of this study to evaluate the seismic performance of buildings designed” [13]. The performance points are found from capacity demand curve for case study building B+G+7 for Push

X and for Push Y were plotted in Figure 4.12 and performance points are found from capacity demand curve for case study building B+G+9 for Push X and for Push Y were plotted in Figure 4.13 but there is a performance point only in the x direction but not in y direction because the structure is expected with stand the design earth quake without significant damage.

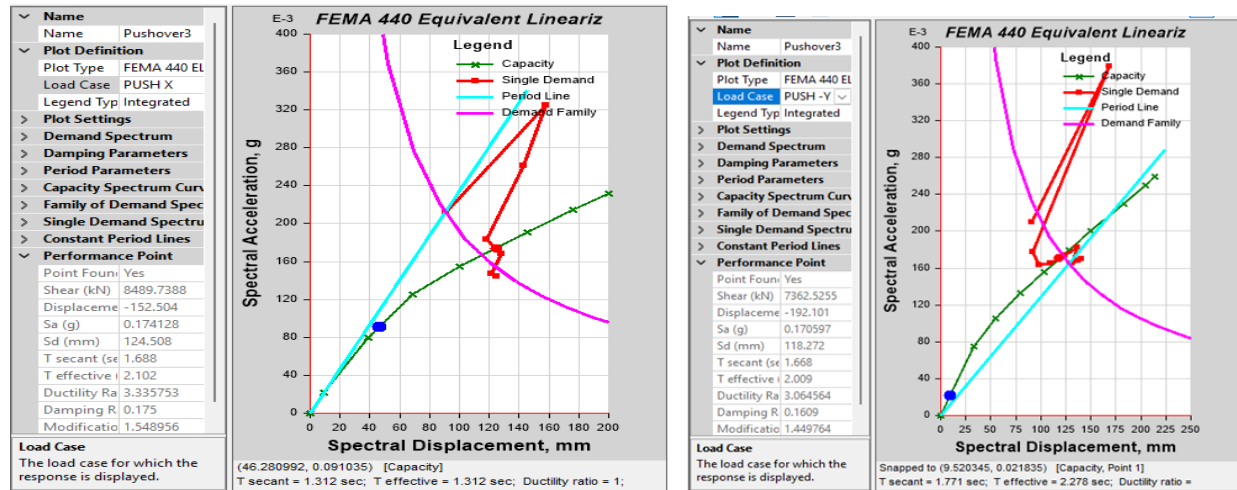


Figure 4.12 The Performance point, Demand curve capacity curve of structure.

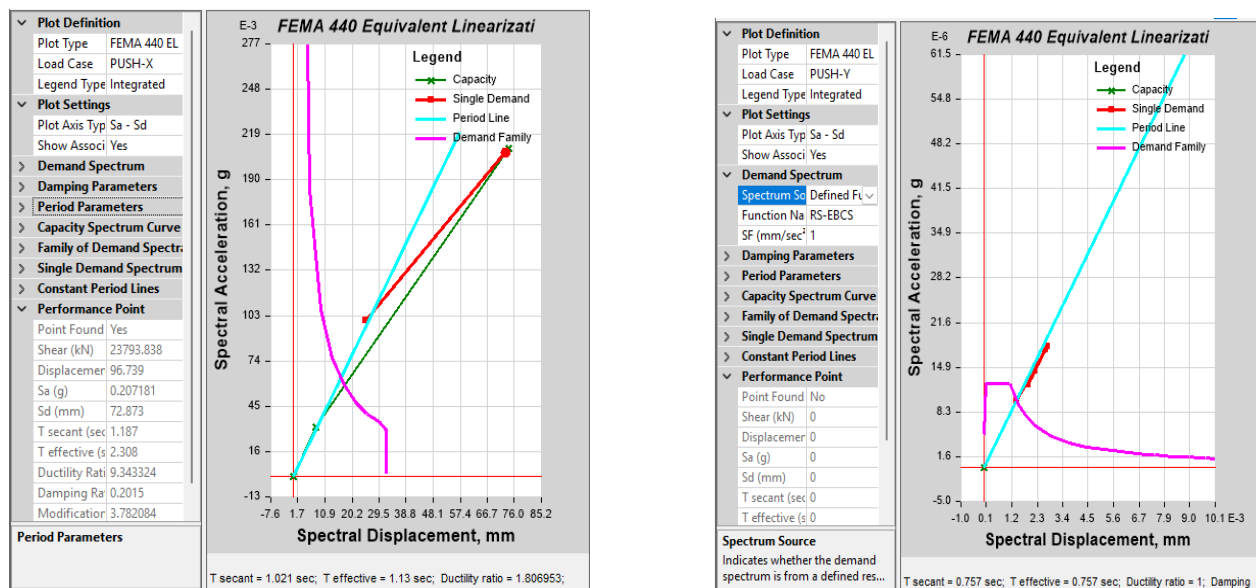


Figure 4.13 The performance point, demand curve, and capacity curve of the structure

Base Shear vs Monitored Displacement													
	Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	0	0	0	2937	1	0	0	0	2938	0	0	0	2938
	1	-14.14	994.1601	2937	1	0	0	0	2938	0	0	0	2938
	2	-50.439	3433.8296	2874	64	0	0	0	2938	0	0	0	2938
	3	-87.197	4733.1805	2734	204	0	0	0	2938	0	0	0	2938
	4	-128.056	5831.8145	2674	264	0	0	0	2938	0	0	0	2938
	5	-166.95	6773.2803	2642	296	0	0	0	2932	6	0	0	2938
	6	-207.909	7732.8849	2612	326	0	0	0	2881	57	0	0	2938
	7	-243.909	8565.1021	2577	361	0	0	0	2836	102	0	0	2938
	8	-297.909	9792.5831	2539	399	0	0	0	2759	179	0	0	2938
	9	-333.909	10605.01...	2524	414	0	0	0	2735	203	0	0	2938
	10	-350.137	10970.27...	2514	421	3	0	0	2717	221	0	0	2938

Table 4.13: Hinge status for various pushover steps in the y direction

Base Shear vs Monitored Displacement													
	Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	0	0	0	2937	1	0	0	0	2938	0	0	0	2938
	1	-11.884	1029.8116	2935	3	0	0	0	2938	0	0	0	2938
	2	-48.836	3749.1694	2823	115	0	0	0	2938	0	0	0	2938
	3	-86.006	5943.3966	2726	212	0	0	0	2938	0	0	0	2938
	4	-123.403	7480.8727	2670	268	0	0	0	2899	39	0	0	2938
	5	-177.403	9352.922	2639	299	0	0	0	2825	113	0	0	2938
	6	-213.403	10553.65...	2626	312	0	0	0	2790	148	0	0	2938
	7	-241.408	11434.17...	2616	318	3	1	0	2777	158	3	0	2938

Table 4.14: Hinge status for different Steps for pushover in the X direction

Base Shear vs Monitored Displacement													
	Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	0	0	0	7200	0	0	0	0	7200	0	0	0	7200
	1	10.591	3635.4012	7199	1	0	0	0	7200	0	0	0	7200
	2	98.084	24108.60...	6348	852	0	0	0	7074	126	0	0	7200

Table 4.15: Hinge status for different steps for pushover in the x direction

E

Base Shear vs Monitored Displacement

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VIEW

	Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A+IO	IO+LS	LS+CP	>CP	Total
▶	0	0	0	7200	0	0	0	0	7200	0	0	0	7200
	1	6.844	3613.4537	7199	1	0	0	0	7200	0	0	0	7200
	2	25.913	12787.21...	6658	542	0	0	0	7200	0	0	0	7200
	3	25.927	12789.77...	6657	543	0	0	0	7200	0	0	0	7200
	4	27.448	13402.19...	6625	575	0	0	0	7200	0	0	0	7200
	5	27.462	13413.59...	6624	576	0	0	0	7200	0	0	0	7200
	6	29.4	14160.21...	6597	603	0	0	0	7200	0	0	0	7200

Table 4.16: Hinge status for different steps for pushover in the y direction

The seismic performance point for the B+G+7 structure was reached between Step 5 and Step 6 in the Y-direction, and between Step 4 and Step 5 in the X-direction. The resulting hinge levels in the members are shown in Tables 4.13 and 4.14. Out of 2,928 hinges formed under pushover loading, 57 in the Y-direction and 113 in the X-direction reached a state between Immediate Occupancy and Life Safety. This indicates that the structure is not locally safe under seismic forces and requires retrofiting.

For the B+G+9 structure, the seismic performance point was reached at Step 2 in the X-direction. In the Y-direction, however, the demand curve approached but did not intersect the capacity curve, and no hinges formed in the push Y case. The hinge levels for push X are presented in Tables 4.15 and 4.16. Of the 7,200 hinges generated, 126 in the X-direction reached a state between Immediate Occupancy and Life Safety. Since these hinges are located in the beams, the structure can be considered locally safe for the given seismic forces.

4.7.1 Level of the structure's hinge formation at the performance point

One of the graphical representations of nonlinear static analysis is plastic hinge formation, which is a good indicator of the strong column weak beam analogy or capacity-based beam and column design. displays each hinge formation's performance level at each pushover and point.

4.7.1.1 Plastic hinge formation of case study buildings

From the analysis result of case study building B+G+7, starting from 2nd step of the pushover analysis, the beams and columns showed post-yield deformation. So, both beams and columns were severely damaged during the pushover analysis as illustrated in figure 4.14,4.15 and 4.16. The plastic hinge formation first occurred at the columns of Ground Floor level then up to top.

Afterwards, it propagated to upper stories especially along the beam for First, Second and Third floors. Due to this, more concentration of stress existed around here and continue the hinge formation mechanism gradually from yielding point to collapse up to the last steps. In push y-at step 2, the plastic hinge formation first occurred at the columns of ground floor and propagated up to roof floor to maximum step 10. The hinge formation mechanisms of the selected case study for building Typology- B+G+7 did not follow the principles of a strong column-weak beam analogy. The yielding occurs at event point IO, LS and CP respectively. The amount of damage occurrence in this building in X direction (push x) higher than y direction (push y)

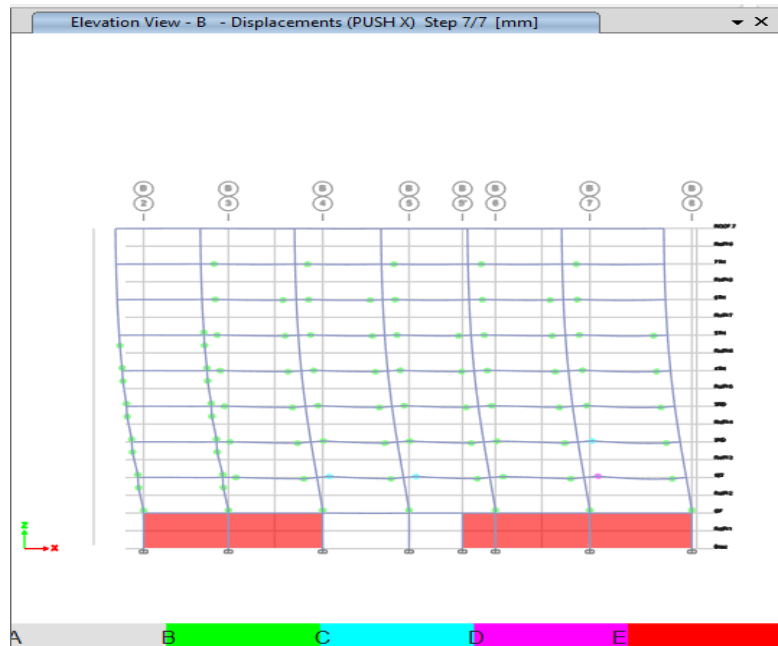


Figure 4.14 Plastic hinge distribution of last step for B+G+7 building in X-direction on axis B.

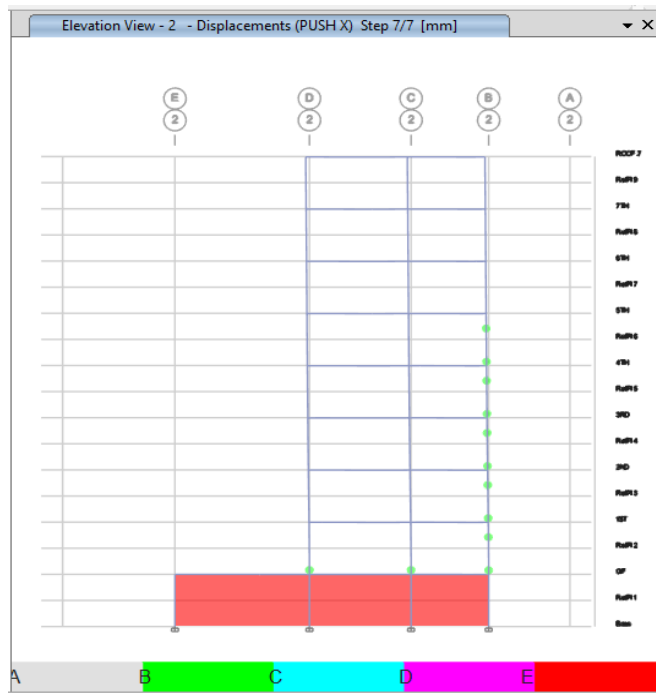


Figure 4.15 Plastic hinge distribution of last step for B+G+7 building in X-direction on axis 2.

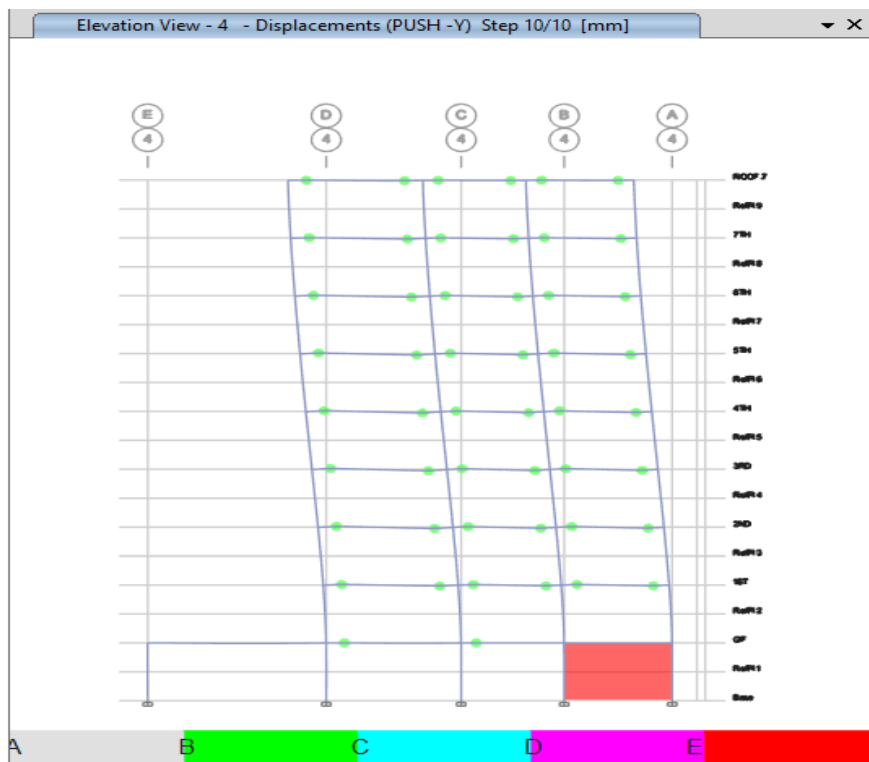


Figure 4.16 Plastic hinge distribution of last step for B+G+7 building in Y-direction on axis 4.

From the analysis results of the case study building B+G+9, it is observed that for all steps during the pushover analysis, most of the beams require post-yield deformation during the pushover analysis, as illustrated in figure 4.17 and figure 4.18. The plastic hinge formation occurred at only one beam at a step in the x-direction. But in the y-direction there is no plastic hinge formation. From the hinge formation mechanisms, it was observed that the principles of a strong column-weak beam analogy.

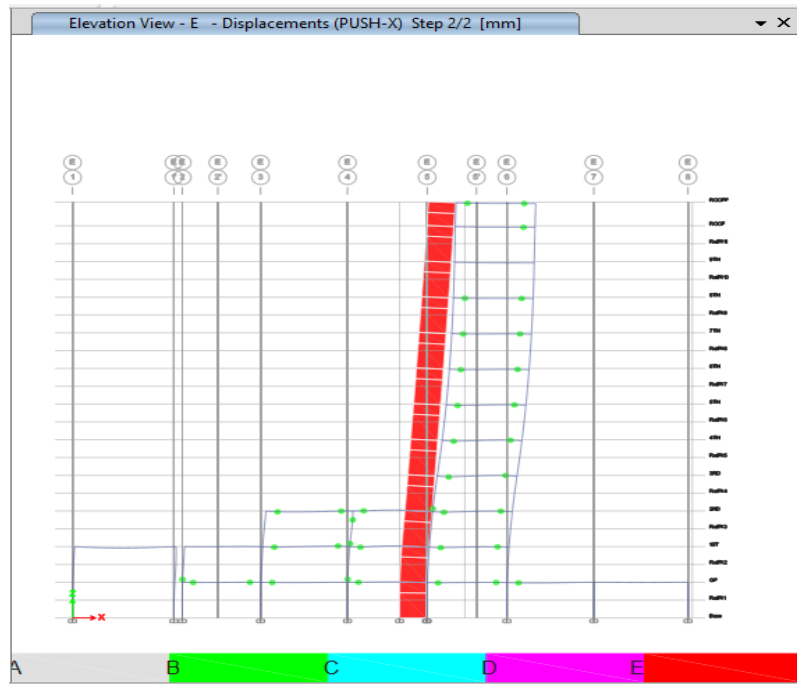


Figure 4.17 Plastic hinge distribution of last step for B+G+9 building in x-direction on axis E.

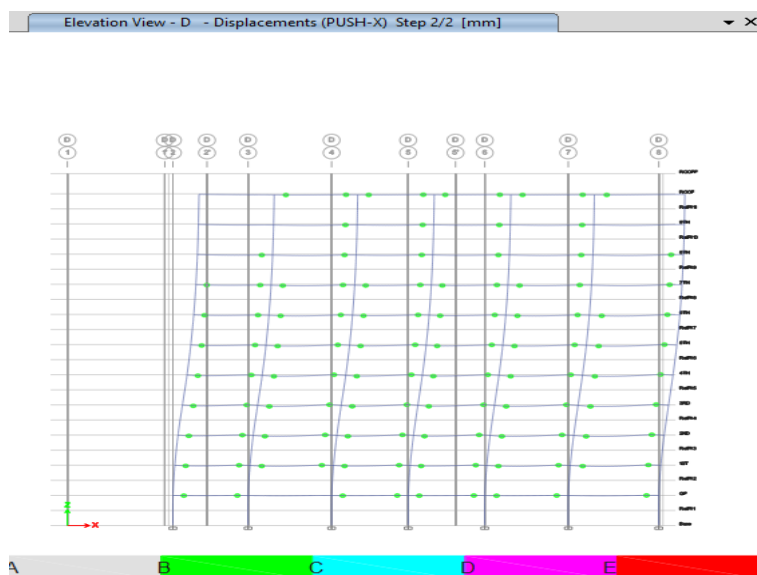


Figure 4.18 Plastic hinge distribution of last step for B+G+9 building in x-direction on axis D

4.8 Structural global performance based on story drift

A 3D elastic response spectrum analysis is carried out in addition to modal analysis to evaluate the building's drift and stability under a 0.1 g PGA demand.

The requirements for the response spectrum analysis are established using the elastic seismic spectrum curve, and the building's seismic mass is specified similarly to the modal analysis.

Story	Elevation	Push Y	IO <0.01	LS [0.01-0.02]	CP>0.02
ROOF	27	0.012729	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
7TH	24	0.013134	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
6TH	21	0.014888	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
5TH	18	0.017849	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
4TH	15	0.019837	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
3RD	12	0.020313	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
2ND	9	0.018219	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
1ST	6	0.008855	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
GF	3	0.000445	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
Base	0	0	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>

Table 4.17 Global performance of the structure for B+G+7 building

Story	Elevation	Push Y	IO <0.01	LS [0.01-0.02]	CP>0.02
ROOF.F	35	0	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
ROOF	33	0.002055	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
9TH	30	0.003417	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
8TH	27	0.005239	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
7TH	24	0.007509	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
6TH	21	0.01002	<i>Not Ok!</i>	<i>ok!</i>	<i>ok!</i>
5TH	18	0.012568	<i>Not Ok!</i>	<i>ok!</i>	<i>ok!</i>
4TH	15	0.013952	<i>Not Ok!</i>	<i>ok!</i>	<i>ok!</i>
3RD	12	0.014355	<i>Not Ok!</i>	<i>ok!</i>	<i>ok!</i>
2ND	9	0.013059	<i>Not Ok!</i>	<i>ok!</i>	<i>ok!</i>
1ST	6	0.007875	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
GF	3	0.000276	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>
Base	0	0	<i>ok!</i>	<i>ok!</i>	<i>ok!</i>

Table 4.18 Global performance of the structure for B+G+9 building

Story	Elevation	Push Y	IO <0.01	LS [0.01-0.02]	CP>0.2
ROOF	45	0.0282	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
12TH F	42	0.0283	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
11TH F	39	0.0282	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
10TH F	36	0.0279	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
9TH F	33	0.0272	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
8TH F	30	0.0262	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
7TH F	27	0.0246	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
6TH F	24	0.0226	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
5TH F	21	0.0201	<i>Not Ok</i>	<i>Not Ok</i>	<i>Ok</i>
4TH F	18	0.0172	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
3RD F	15	0.0139	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
2ND F	12	0.0121	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
1ST F	9	0.0108	<i>Not Ok</i>	<i>Ok</i>	<i>Ok</i>
GF	6	0.0036	<i>ok!</i>	<i>Ok</i>	<i>Ok</i>
BASEMENT1	3	0.0012	<i>ok!</i>	<i>Ok</i>	<i>Ok</i>
BASE	0	0.0000	<i>ok!</i>	<i>Ok</i>	<i>Ok</i>

Table 4.19 Global performance of the structure for 2B+G+12 building

Story	Elevation	Push Y	IO <0.01	LS [0.01-0.02]	CP>0.2
ROOF 1	65.5	0.0016	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
ROOF	63	0.0039	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
18th	60	0.0042	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
17th	57	0.0047	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
16th	54	0.0051	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
15th	51	0.0056	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
14th	48	0.006	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
13th	45	0.0065	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
12th	42	0.007	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
11th	39	0.0074	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
10th	36	0.0078	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
9th	33	0.0081	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
8th	30	0.0083	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
7th	27	0.0084	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
6th	24	0.0083	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
5th	21	0.008	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
4th	18	0.0076	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
3rd	15	0.0069	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
2nd	12	0.0061	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
1st	9	0.0051	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
GF	6	0.0029	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
BASE 1	3	0.0009	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>
BASE 2	0	0	<i>Ok</i>	<i>Ok</i>	<i>Ok</i>

Table 4.20 Global performance of the structure for 2B+G+18 building

The overall performance level of the building is assessed in accordance with the provisions of FEMA 356, which stipulate roof drift limits of 0.5%, 1%, and 2% for the Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) levels, respectively.

Inter-story drift is commonly used to evaluate ***floor-level*** performance. But when it comes to determining the ***overall seismic performance*** of the entire structure, you need to aggregate or interpret those floor-level results using performance objectives and structural response parameters. The ***worst-performing story*** governs the performance of the entire structure.

CHAPTER FIVE: CONNCULSION AND RECOMENDATION

6.1 Summary

This project was done to evaluate seismic performance condominium buildings constructed at Addis Ababa implemented by government project in from 2005 E.C up to now. After choosing representative samples to capture the variety of these housing projects, Building Typologies B+G+7, B+G+9, 2B+G+12, and 2B+G+18 were utilized to carry out linear static analysis and nonlinear static pushover analysis. Data such as structural design, some structural reports and existing built-up plans drawings were collected from plans Housing Development and Constructions Agency found. Such data were used to model the buildings in order to mock real-time scenarios. The modeling was carried out using ETABS 2022 Engineering Software. All load patterns and load combinations were modeled according to ES EN1992 &1998: 2015; which included gravity load, imperfection and earthquake loadings. Earthquake or lateral loads were designed in accordance with EN1998; 2004.National annexes for the ground acceleration were considered to be 0.1g, which is stipulated in the revised national code.

6.2 Conclusion

The parameter, analysis technique, and seismic demand of the structure vary as a result of seismic code revisions. The demand capacity ratio and pushover analysis were used to examine the capacity and performance level of reinforced concrete frames, respectively. As a result, the analysis of building typologies led to the following conclusions.

- The demand-to-capacity ratio method didn't show the level of performance capacity of the buildings explicitly; the mechanisms and mode of failure only tell the adequacy of the member or structure collectively. Whereas nonlinear static pushover analysis showed all these important attributes. Hence, nonlinear static pushover analysis is preferable to capture how the building behaves under lateral loadings.
- By using the demand to capacity ratio method, the PMM interaction became the main indictor for the ultimate limit state, while inter story drift result showed Damage Limitation state. By demand to capacity ratio method, all buildings considered in the case study buildings have different inadequate level standing from number of element inadequacy for the design earthquake load at ultimate limit state. This is a significant finding as it calls for an appropriate measure to be taken to avoid prospective catastrophes.

- Using the nonlinear static pushover analysis method, the case-study building in the B+G+7 typology exceeds the collapse-prevention performance level when subjected to earthquake loading in the X direction. This building type showed premature failure when such a type of earthquake happened. However, in the case of building B+G+9, the performance is better relative to B+G+7; there is a hinge in the x direction on the beam but no hinge in the y direction. This indicates that the building is expected to withstand the design earthquake without significant damage.
- In this project, the results of demand to capacity ratio (linear static analysis) and nonlinear static pushover analysis were found to be almost similar. However, detail information was obtained in nonlinear static pushover analysis.
- In the case study for building in Typology B+G+7, the hinges are developed in both beams and columns, but most of the hinges are developed in the beam. This shows that the designers did not use the strong column-weak beam analogy. This is detrimental in the aftermath of an earthquake in that it does not ensure safe evacuation of occupants. This design problem was not found in building Typology B+G+12. Building Typology B+G+9 was also observed to perform better in comparison with B+G+7, but there is hinge formation, and it needs retrofitting in beams in the other building types.
- Generally, all four typologies used for the study—Building Typology B+G+7, B+G+9, B+G+12, and B+G+18—had sufficient capacity to withstand damage-limiting earthquakes, according to the results of linear static analysis. And additional investigation on nonlinear static analysis showed only two structures (structure types B+G+9 and B+G+18) had sufficient capacity to ensure life safety. Additionally, 2B+G+18 has good performance in the Y direction between A and Io, while the other building (building types B+G+7 and 2B+G+12) was inadequate for IO (immediate occupancy); it is in the range of LS (life safety) but near Cp (collapse preventive). But the most important discovery was that every type of building did not pass the ultimate limit state evaluation, which is a crucial criterion outlined in the code.

6.3 Recommendation

As the majority of the buildings were found not to comply with a performance level in one way or the other, there is a need to use retrofitting schemes to ensure that these buildings can somehow withstand the possibility of future earthquake loads. The structure as a whole should be checked to ensure both overturning & sliding stability responses using soil investigation techniques. The effect of soil-structure interaction was not studied in this project.

Because of a lack of ground acceleration data and time, buildings do not cross-check using other nonlinear analysis methods like nonlinear time history analysis (NLTH). This factor should be studied in the future to get a more elaborate picture of how these buildings actually perform during earthquakes.

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APPENDIX A: CASESTUDY BUILDINGS DATA USED, MODELING AND ANALYSIS OUT DRAWI NGS

DATE AND TIME	↑↓	MAG DEPTH	LOCATION	DETAILS	MAP
Feb 14, 11:28 pm (Addis Ababa) 10 days ago		6.0 10 km	 Harari, 8.5 km northeast of Metehara, Oromiya, Ethiopia 24 reports	Info	
Jan 22, 12:44 am (Addis Ababa) 5 weeks ago		5.1 10 km	 19 km NW of Āwash, Ethiopia 63 reports	Info	
Jan 11, 06:19 pm (Addis Ababa) 6 weeks ago		5.2 10 km	 21 km E of Āwash, Ethiopia 100 reports	Info	
Jan 11, 03:19 am (Addis Ababa) 6 weeks ago		5.2 10 km	 30 km N of Āwash, Ethiopia 83 reports	Info	
Jan 8, 03:53 am (Addis Ababa) 7 weeks ago		5.3 10 km	 10 km W of Āwash, Ethiopia 22 reports	Info	
Jan 4, 03:52 am (Addis Ababa) 7 weeks ago		5.7 8 km	 52 km N of Āwash, Ethiopia 588 reports	Info	
Jan 3, 08:01 pm (Addis Ababa) 7 weeks ago		5.5 10 km	 44 km NE of Āwash, Ethiopia 62 reports	Info	
Jan 3, 05:27 pm (Addis Ababa) 7 weeks ago		5.2 10 km	 37 km SSE of Abomsa, Ethiopia 51 reports	Info	
Jan 2, 10:41 am (Addis Ababa) 8 weeks ago		5.1 10 km	 33 km N of Āwash, Ethiopia 64 reports	Info	

Figure A.1 Resent Ethiopian Earth quicks (image from Volcano Discovery)

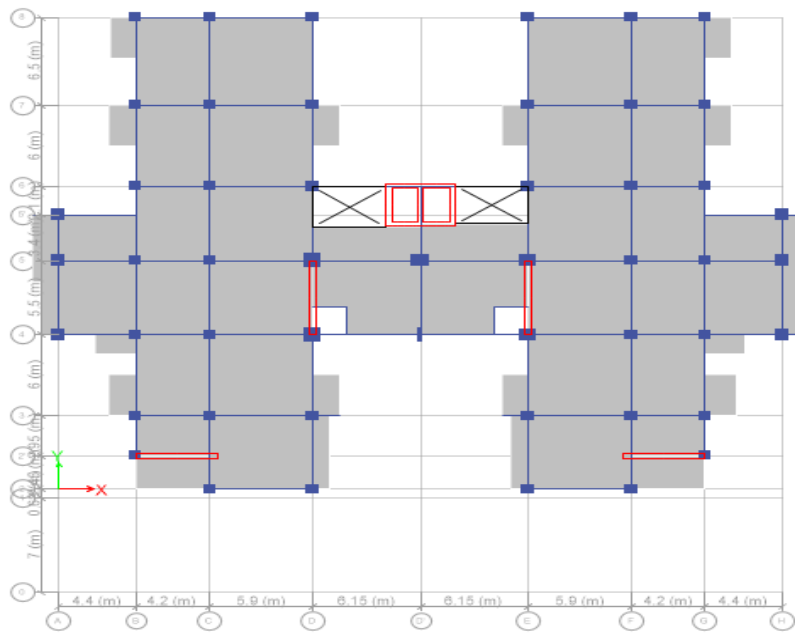


Figure A.2 Sample Floor Plan 2B+G+18

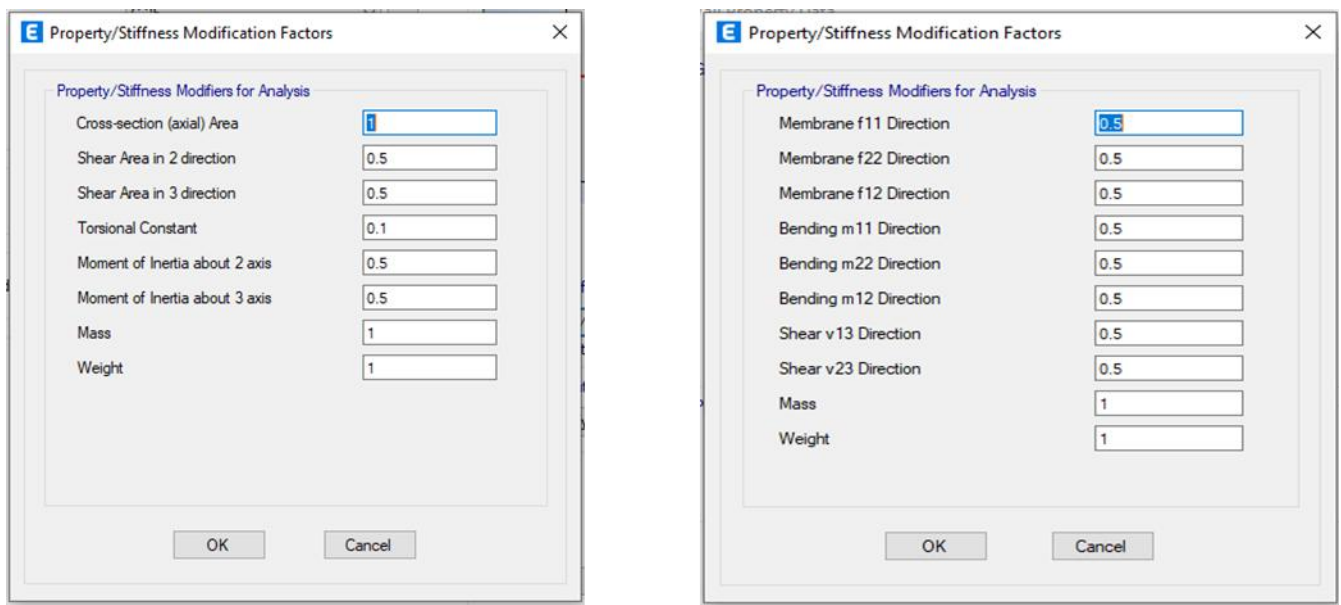


Figure A.3 Stiffness Modifier Factor Assigning Sample for Beam and Wall.

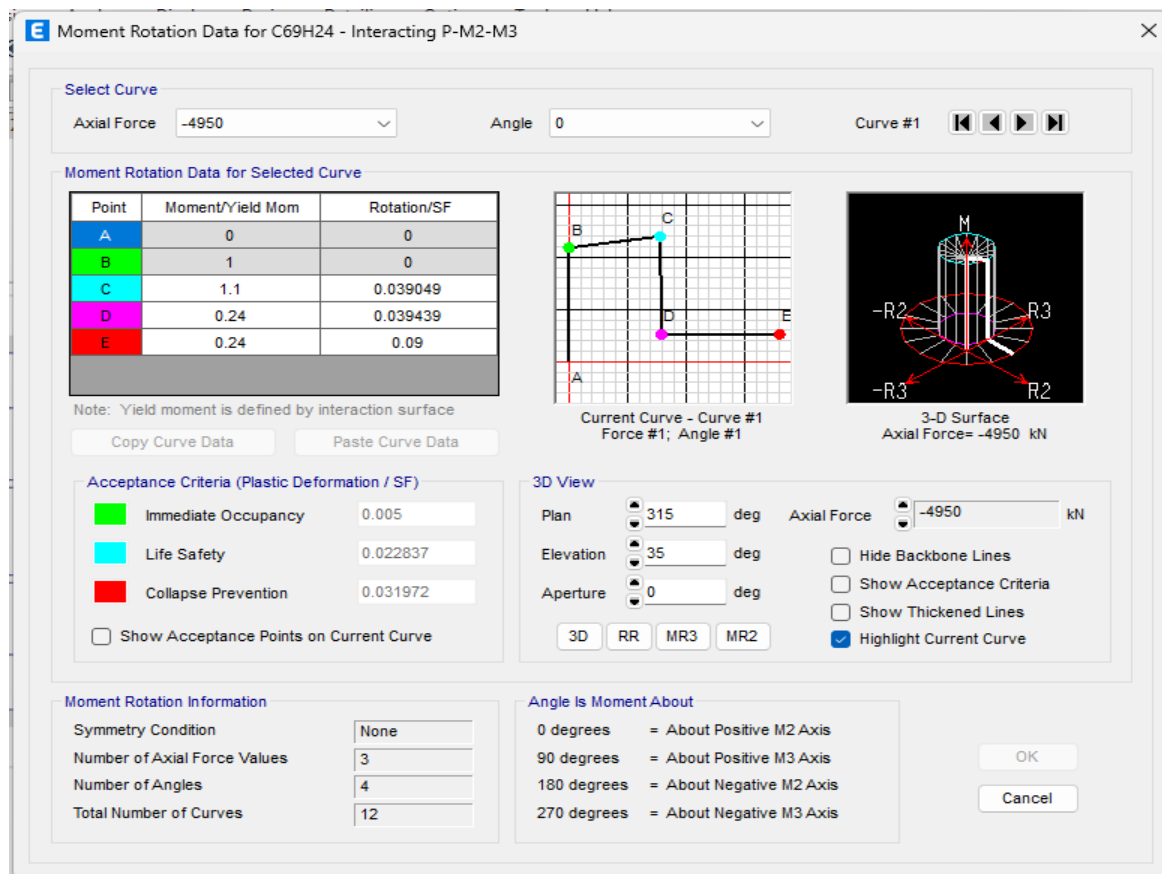


Figure A.4 Plastic Hinge Assigning Sample for column.

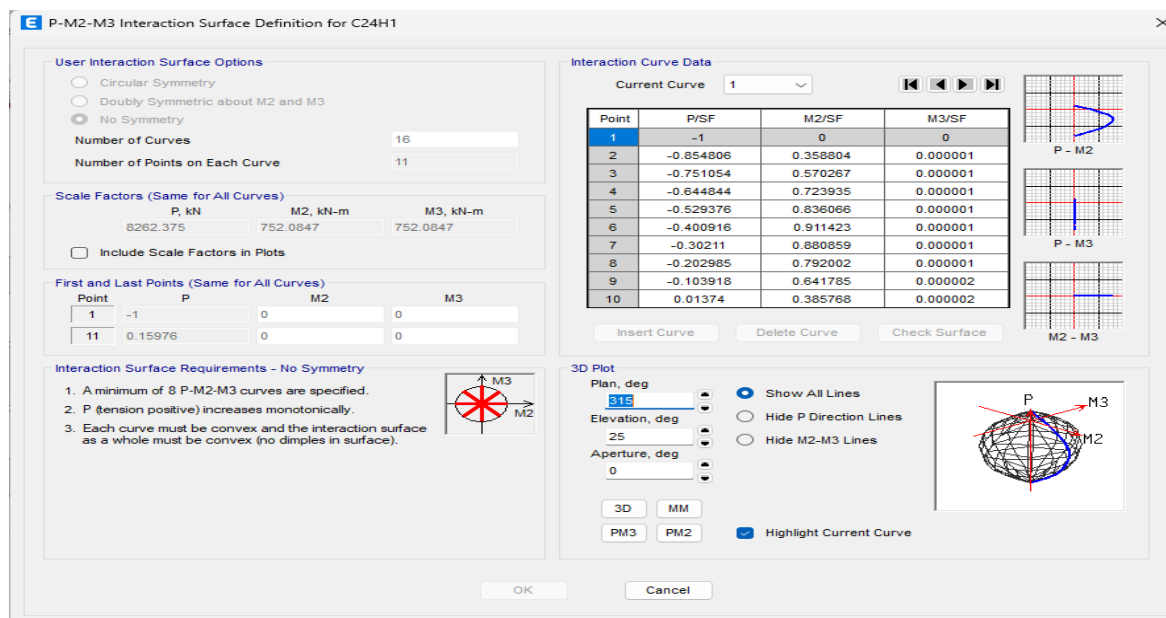


Figure A.5 Axial force moment interaction Diagram for Plastic Hinge Assigning Sample in column.

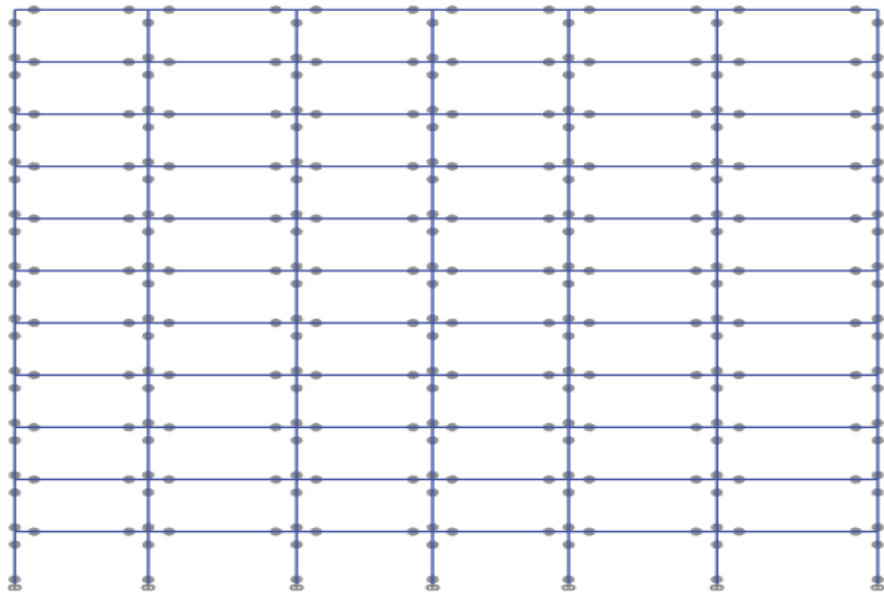
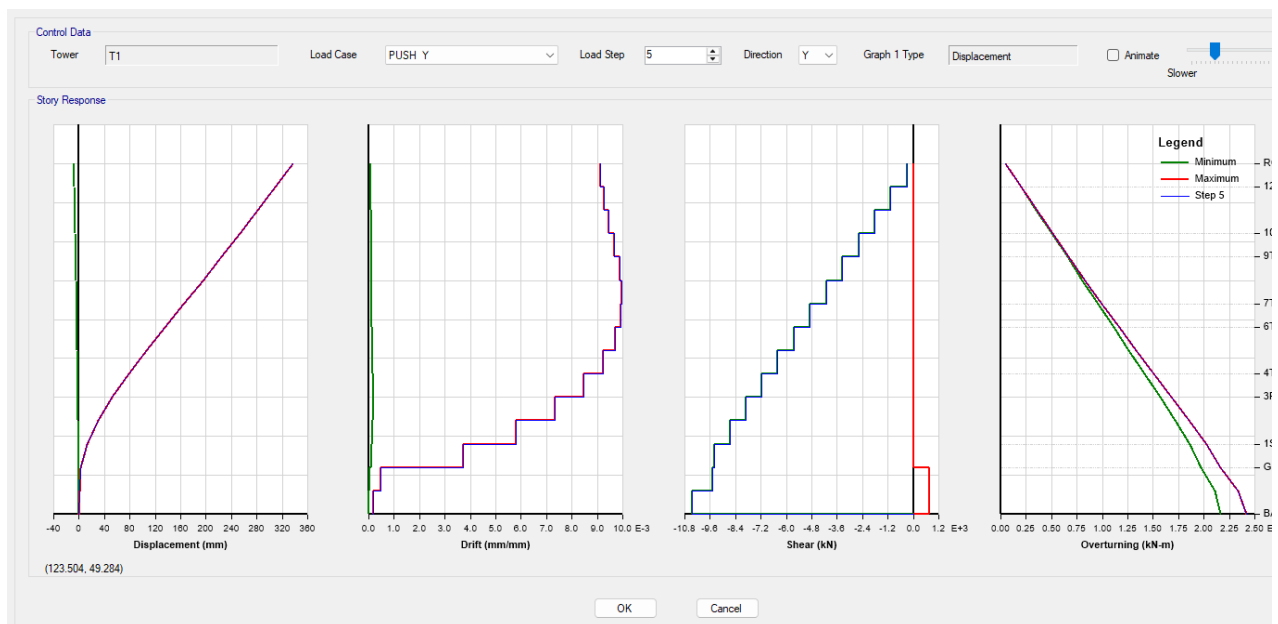


Figure A.6 The Assigned Plastic B+G+9



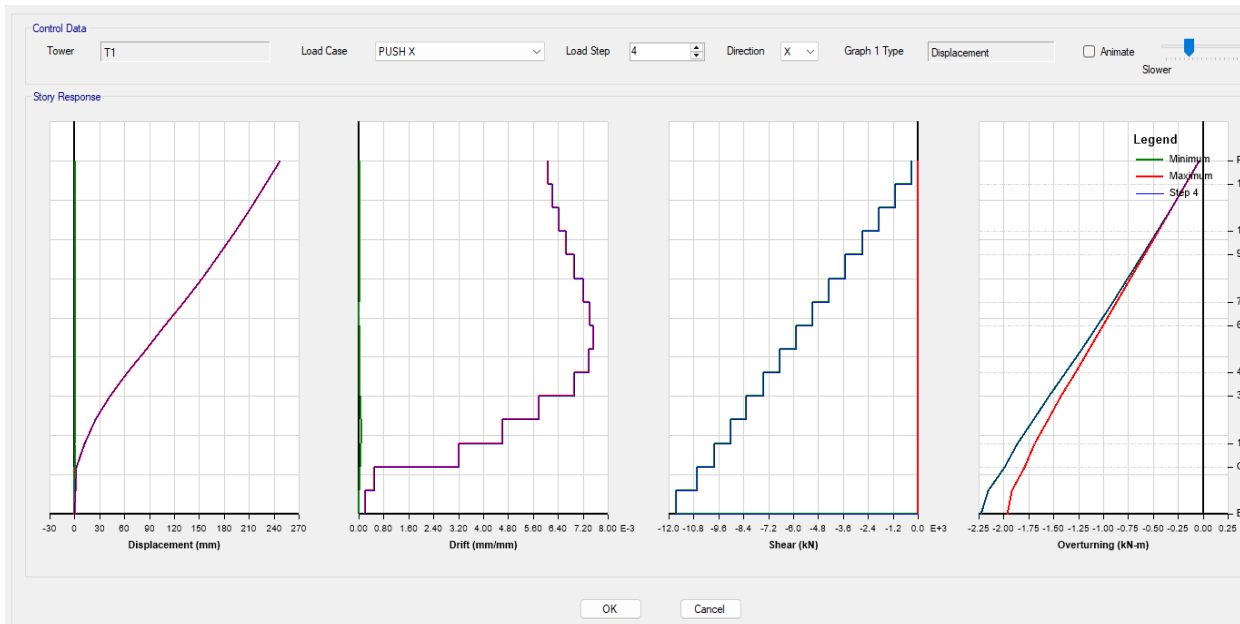


Figure A.7 General Story Response of B+G+12.

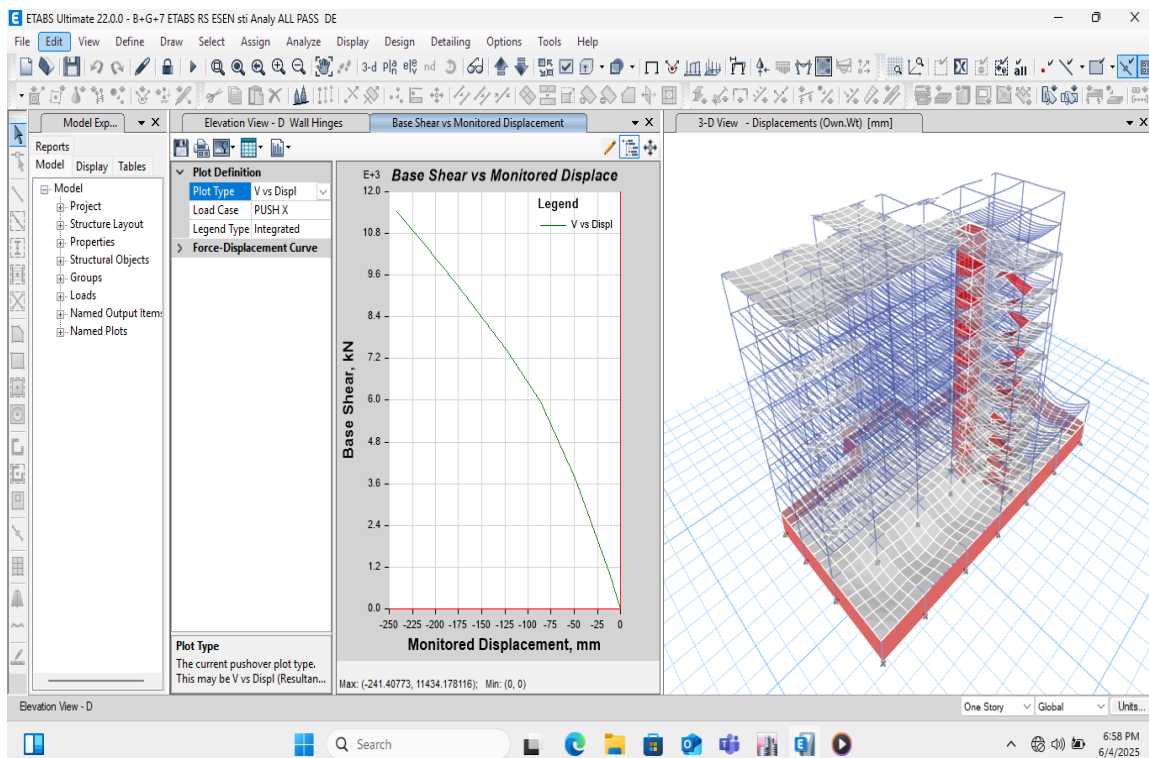


Figure A.8 Pushover curve in X direction B+G+7.

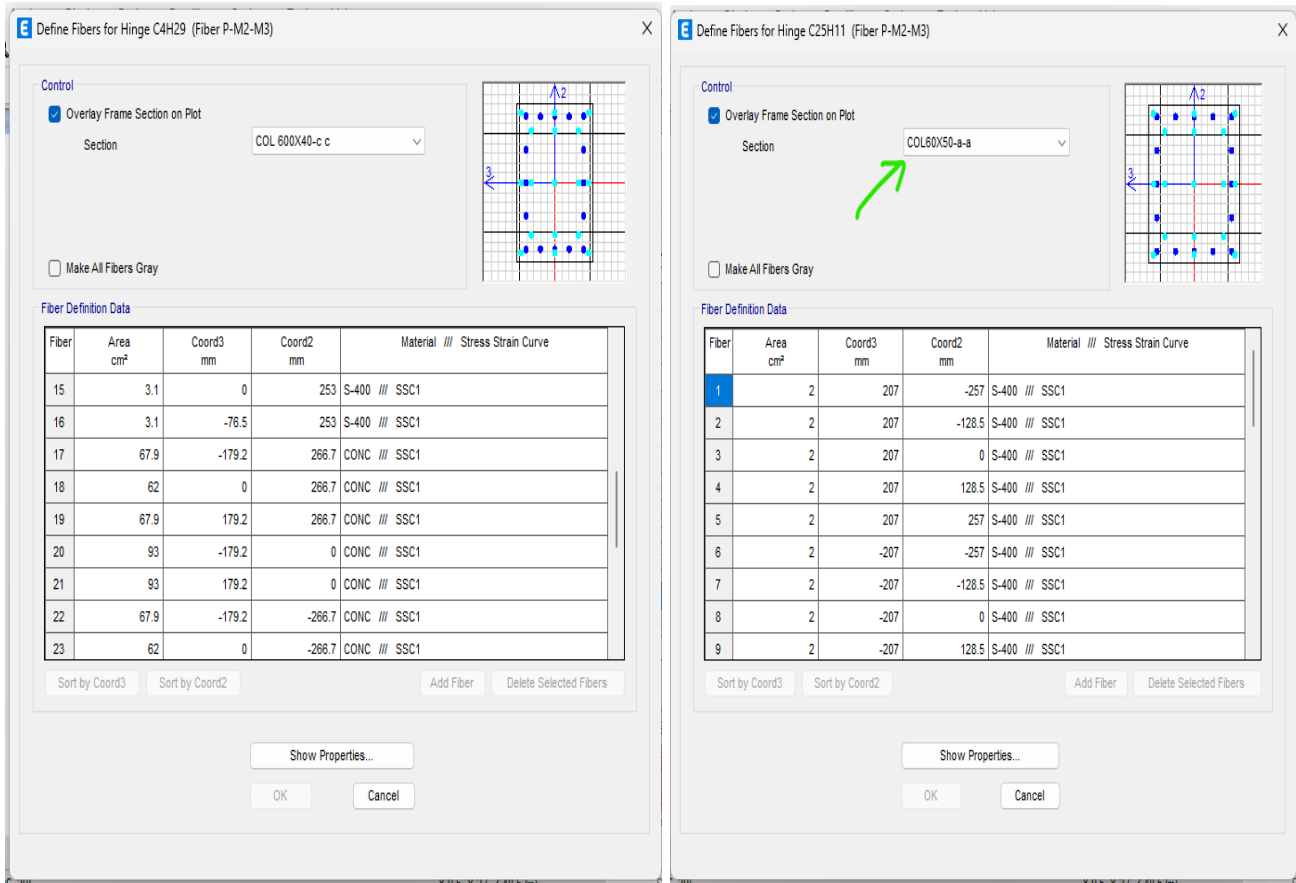


Figure A.9 Definition of Fiber hinge for column in 2B+G+12.

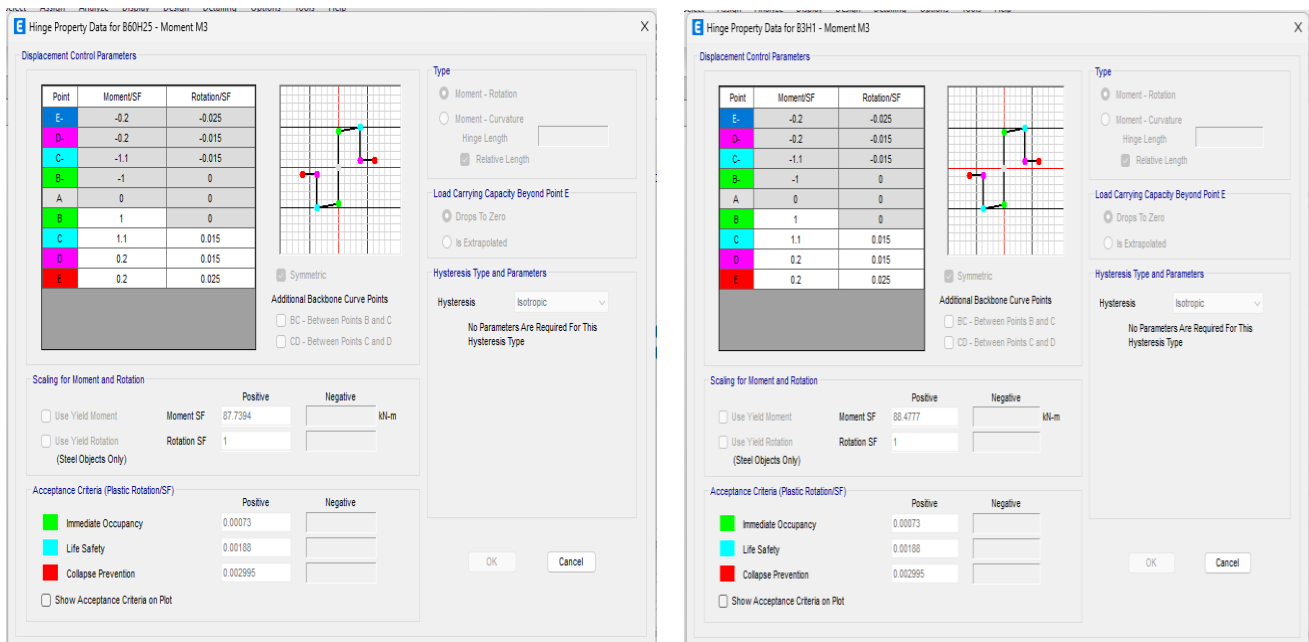


Figure A.10 Definition of Fiber hinge for Beam in 2B+G+12.

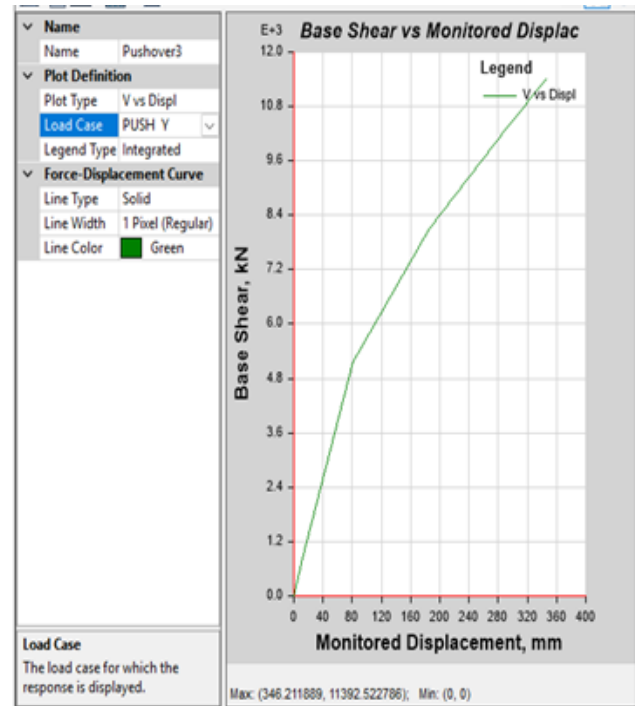
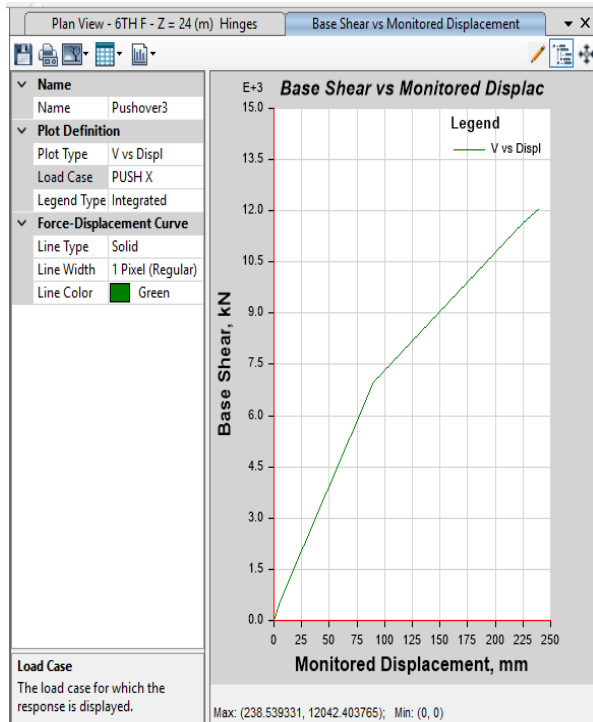


Figure A.11 Push over curve in X and Y direction for 2B+G+12.

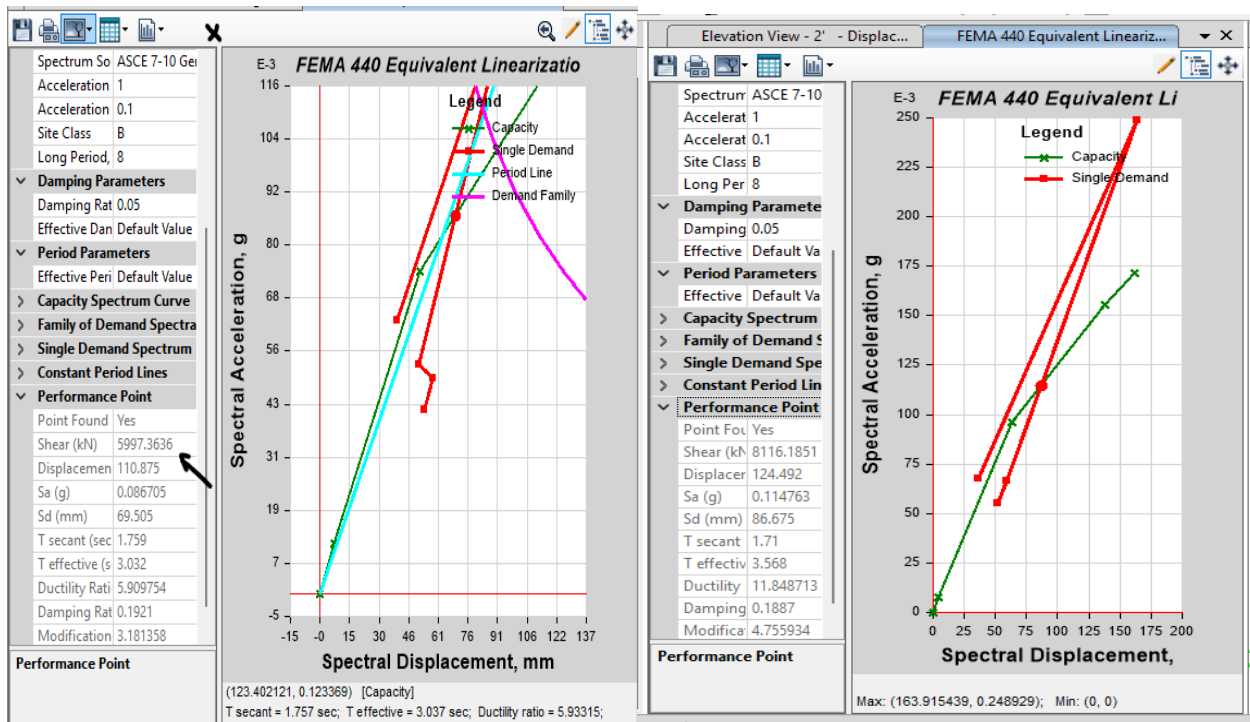


Figure A.12 Demand to capacity curve (performance point) for 2B+G+12.

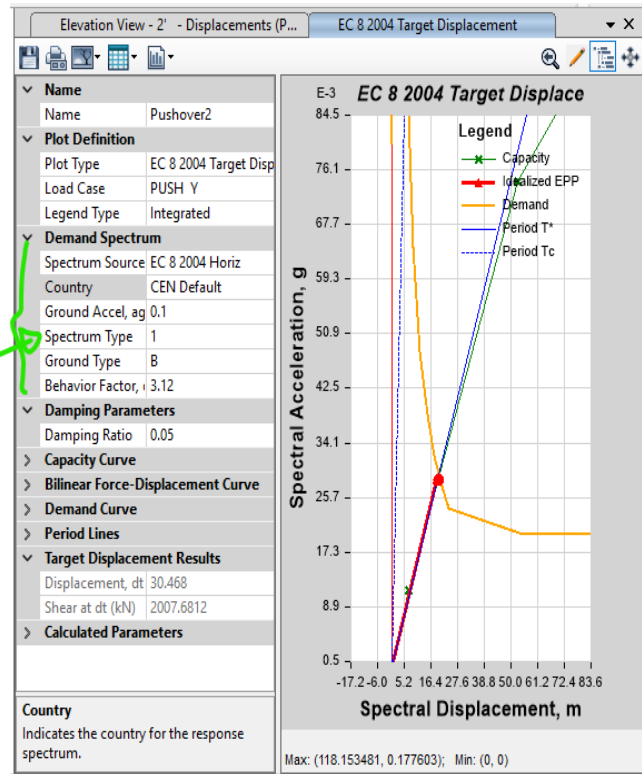
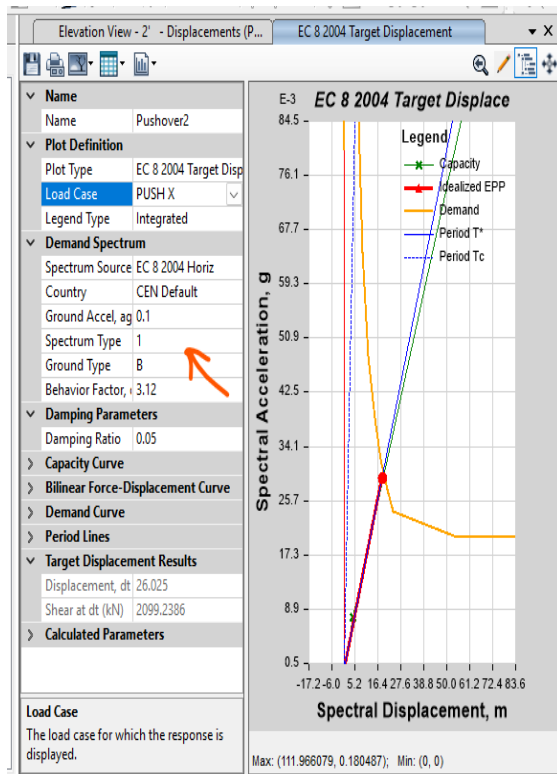


Figure A.13 Demand to capacity curve (performance point) for 2B+G+12.

Base Shear vs Monitored Displacement													
Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-I/O	I/O-LS	LS-CP	>CP	Total	
0	0	0	4816	26	0	0	0	4842	0	0	0	4842	
1	5.625	554.3765	4814	28	0	0	0	4837	5	0	0	4842	
2	90.334	6995.7026	4449	393	0	0	0	4614	107	93	28	4842	
3	224.991	11654.40...	4242	589	1	0	10	4267	161	31	383	4842	
4	238.539	12042.40...	4236	592	4	0	10	4255	162	31	394	4842	

Base Shear vs Monitored Displacement													
Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-I/O	I/O-LS	LS-CP	>CP	Total	
0	0	0	4816	26	0	0	0	4842	0	0	0	4842	
1	11.25	816.0534	4811	31	0	0	0	4841	1	0	0	4842	
2	81.172	5161.2257	4565	277	0	0	0	4726	105	11	0	4842	
3	185.105	8086.8714	4374	468	0	0	0	4400	85	102	255	4842	
4	291.977	10344.756	4314	528	0	0	0	4330	85	24	403	4842	
5	346.212	11392.52...	4290	545	6	1	0	4303	97	18	424	4842	

Figure A.14 Hinge formation and number of hinges for 2B+G+12.

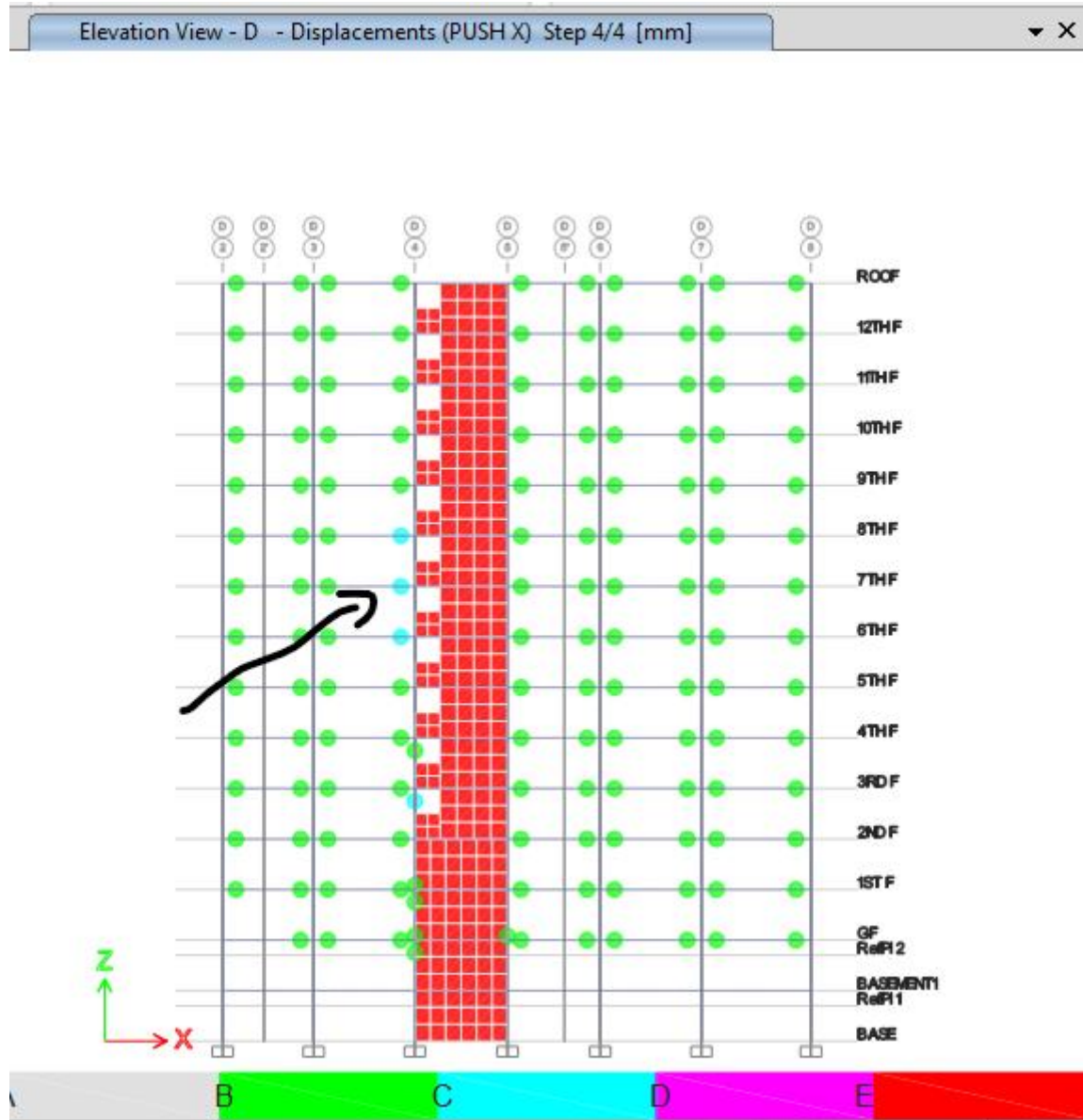


Figure A.15 The formation of Plastic Hinge Sample in columns and Beams B+G+12.

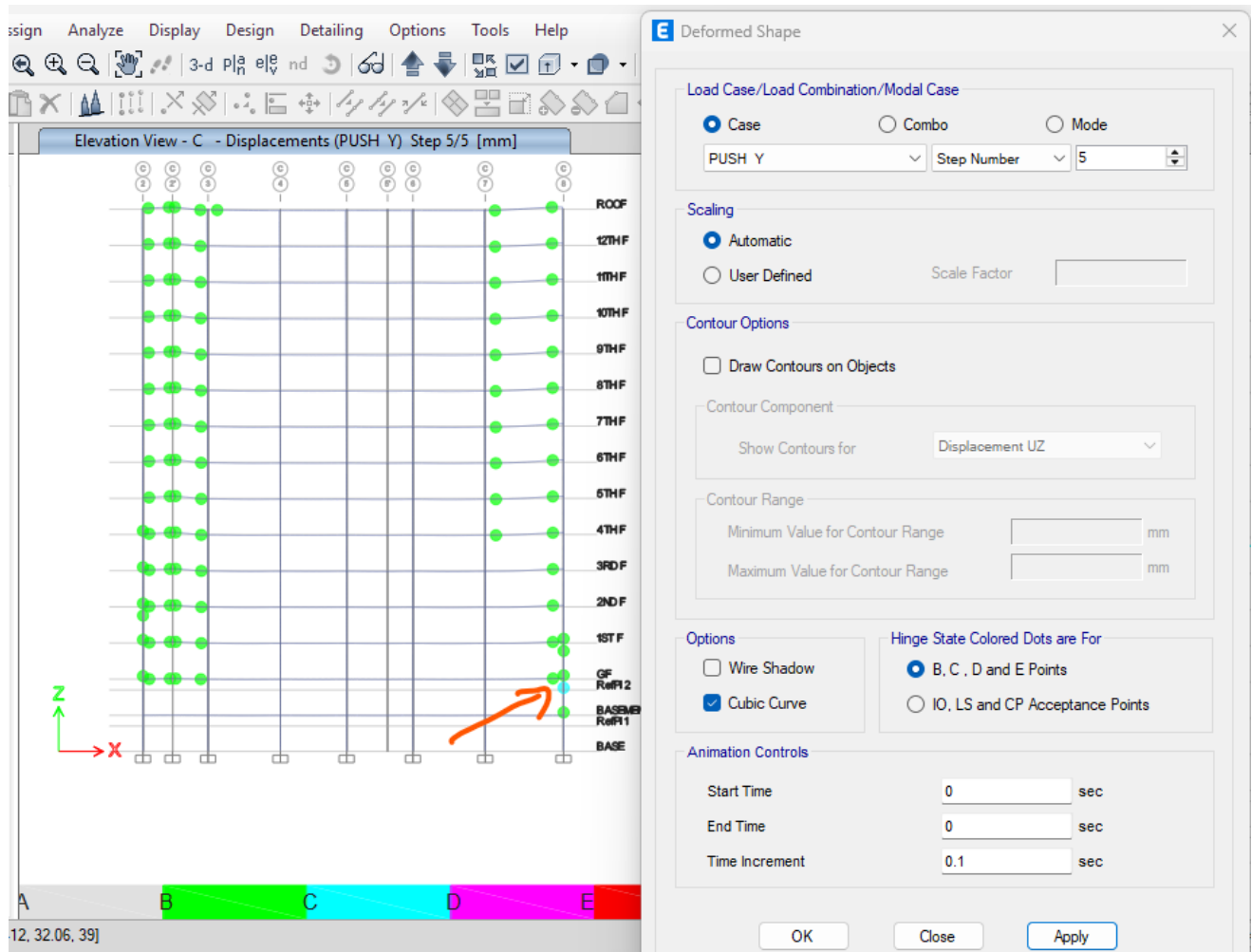


Figure A.16 The Plastic Hinge Formation Sample in columns and Beams B+G+12.

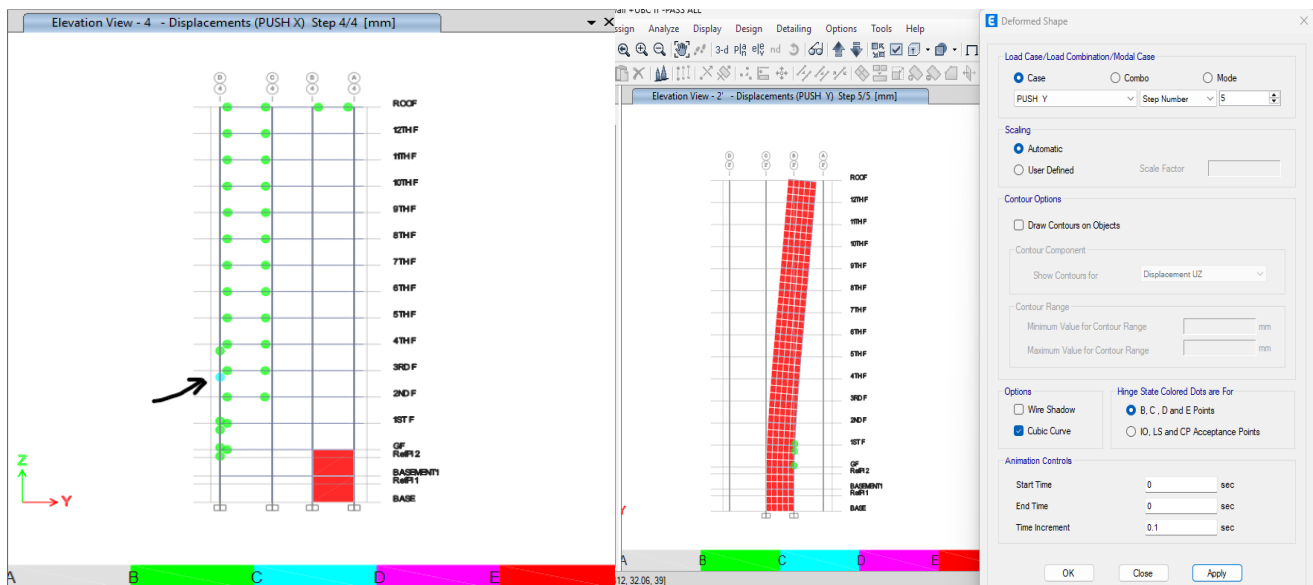


Figure A.17 The Plastic Hinge Formation Sample in columns and Beams B+G+12.

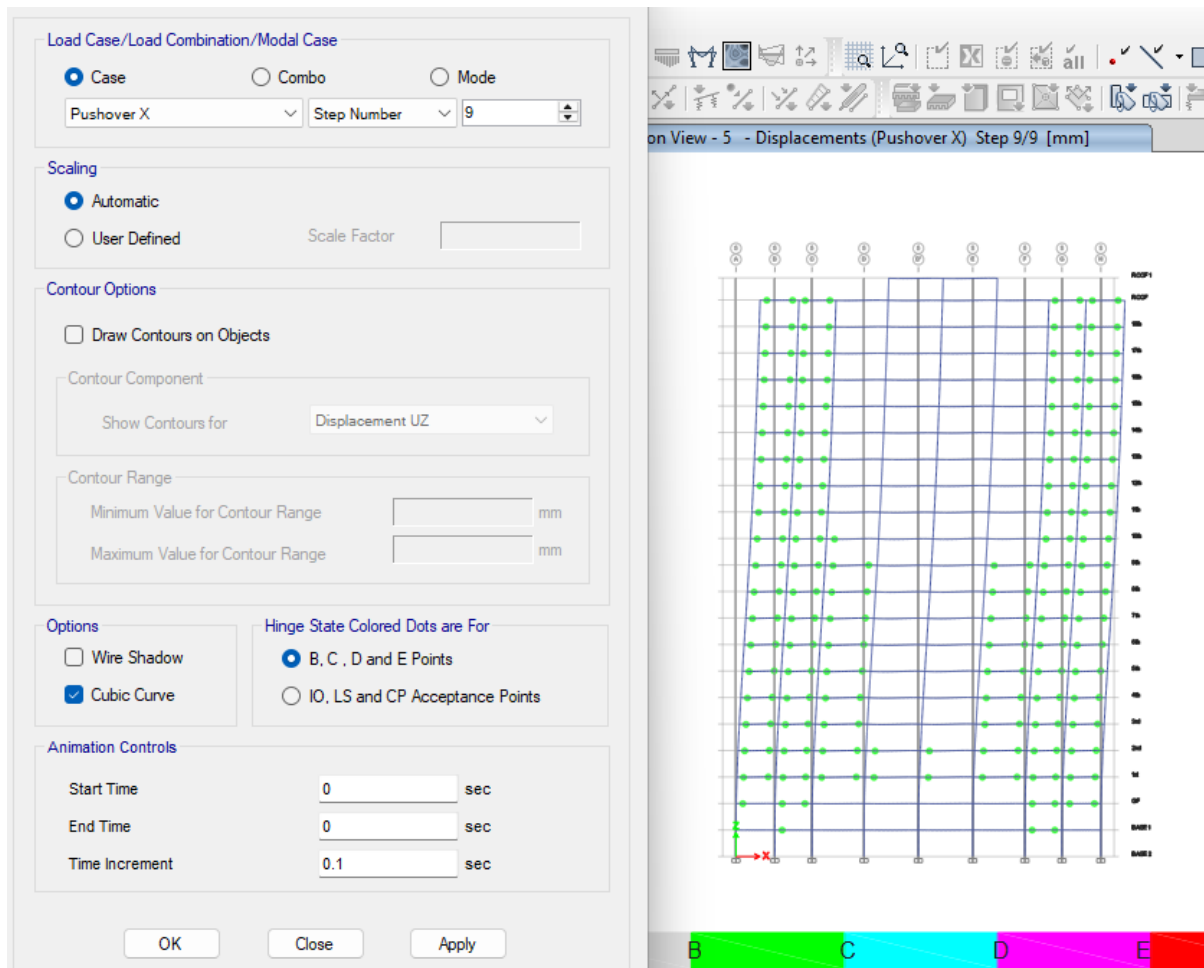


Figure A.18 The Plastic Hinge Formation Sample in columns and Beams 2B+G+18.

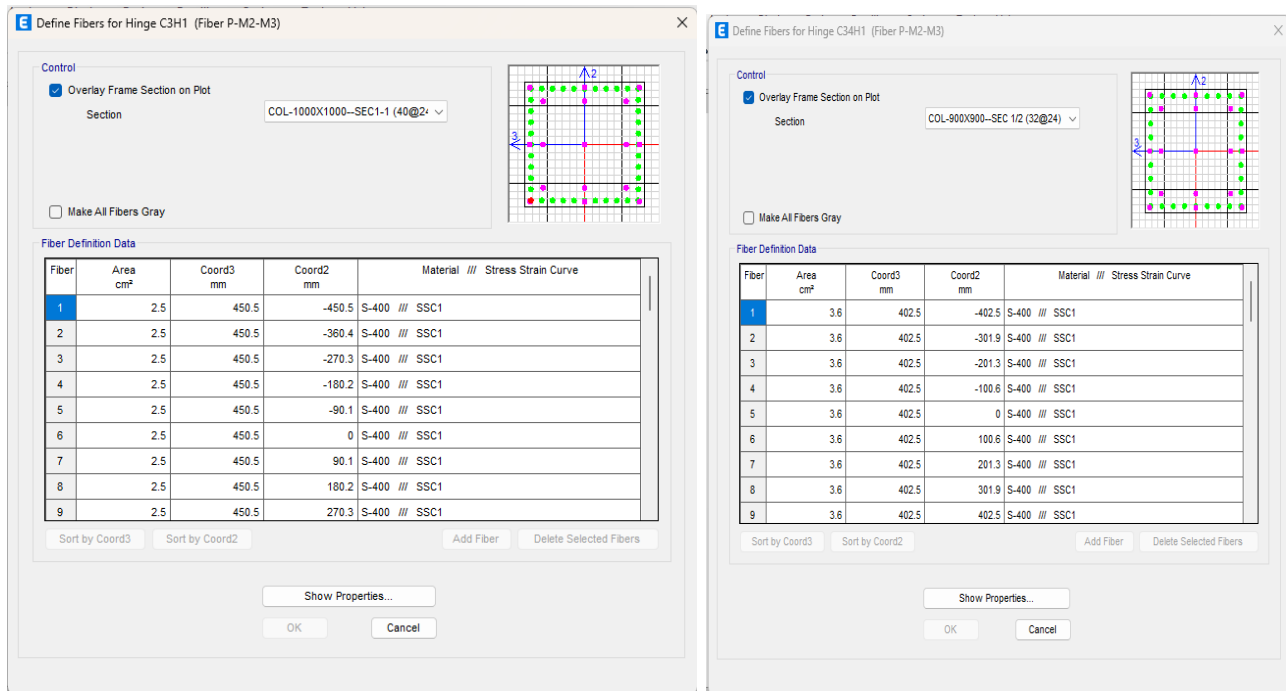


Figure A.19 Definition of Fiber hinge for column in 2B+G+18.

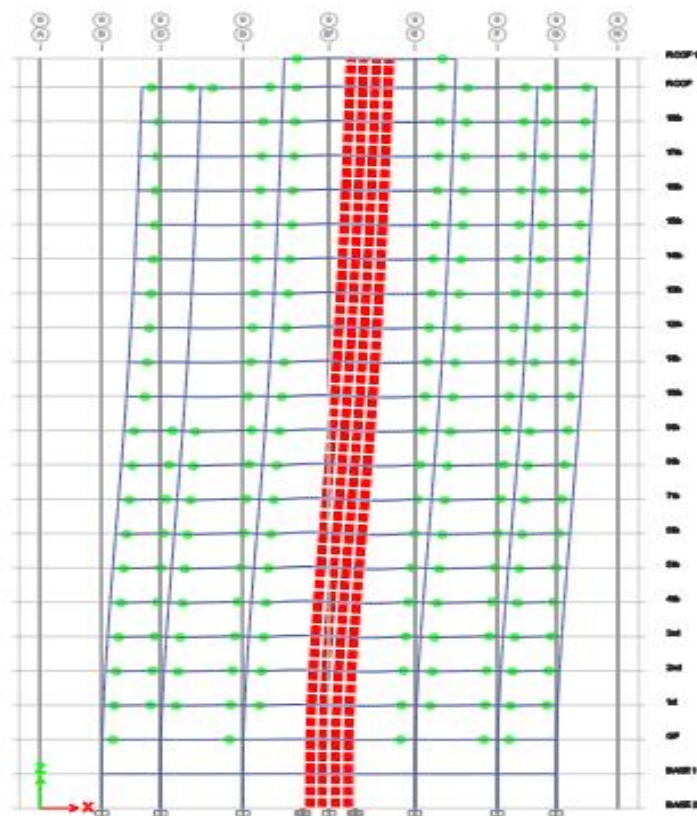


Figure A.18 The Plastic Hinge Formation Sample in columns and Beams 2B+G+18.

APPENDIX B: SOFTWARE VERIFICATION

B.1 Frame Verification

The verification of the finite element software ETABS V21 is conducted to assess its accuracy in modeling the nonlinear behavior of reinforced concrete structures. This process involves a two-dimensional nonlinear analysis of a shear-critical reinforced concrete frame.

The model is meticulously calibrated against a benchmark experimental study conducted by “Kien Vinh Duong, Shamim A. Sheikh, and Frank J. Vecchio”. To ensure a direct and meaningful comparison, all model parameters are directly sourced from the experimental program. This includes:

Material properties for concrete and reinforcement.

Concrete Material Properties				
f'_c (MPa)	ϵ_0 ($\times 10^{-3}$)	E_c (MPa)	G_c (MPa)	μ
42.9	2.31	30058	13069	0.2

Table B.1 Concrete Material Properties

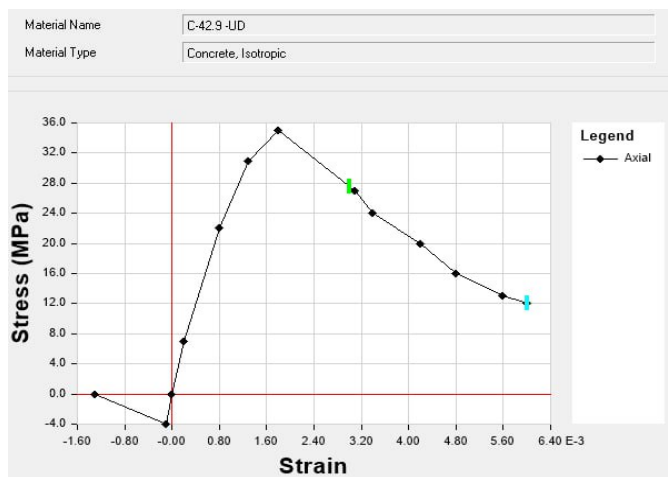


Figure B.1 Concrete Material Stress Strain curve

Bar Size	Diameter, mm (in.)	Cross-sectional area, mm ² (in. ²)	ϵ_y , $\times 10^{-3}$	ϵ_{sh} , $\times 10^{-3}$	f_y , MPa (ksi)	f_u , MPa (ksi)	E_s , MPa (ksi)	E_{sh} , MPa (ksi)
No. 10	11.3 (0.44)	100 (0.16)	2.38	22.8	455 (66)	583 (84.6)	192,400 (27,904)	1195 (173)
No. 20	19.5 (0.77)	300 (0.47)	2.25	17.1	447 (64.8)	603 (87.5)	198,400 (28,774)	1372 (199)
US No. 3	9.5 (0.375)	71 (0.11)	2.41	28.3	506 (73.4)	615 (89.2)	210,000 (30,457)	1025 (149)

Table B.2 Rebar Material Properties

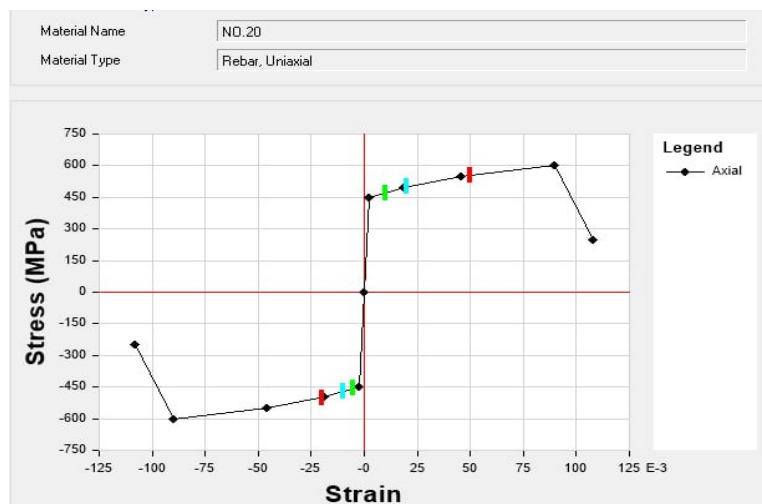


Figure B.2 Rebar Material Stress Strain curve

Cross-sectional dimensions and member sizes.

Member	b , mm (in.)	h , mm (in.)	Bottom steel	Top steel	Stirrup spacing	$\rho_x(A_s/bd)$, %	ρ_y , %
Beam	300 (11.8)	400 (15.7)	4 No. 20	4 No. 20	US No. 3 at 300 mm (single hoop)	1.143	0.158
Column	300 (11.8)	400 (15.7)	4 No. 20	4 No. 20	No. 10 at 130 mm (double hoop)	1.111	1.018
Column (at base)	300 (11.8)	400 (15.7)	8 No. 20	8 No. 20	No. 10 at 130 mm (double hoop)	2.39	1.018
Base	800 (31.5)	400 (15.7)	8 No. 20	8 No. 20	No. 10 at 175 mm (triple hoop)	0.857	0.429

Table B.3 Cross-sectional dimensions and member sizes.

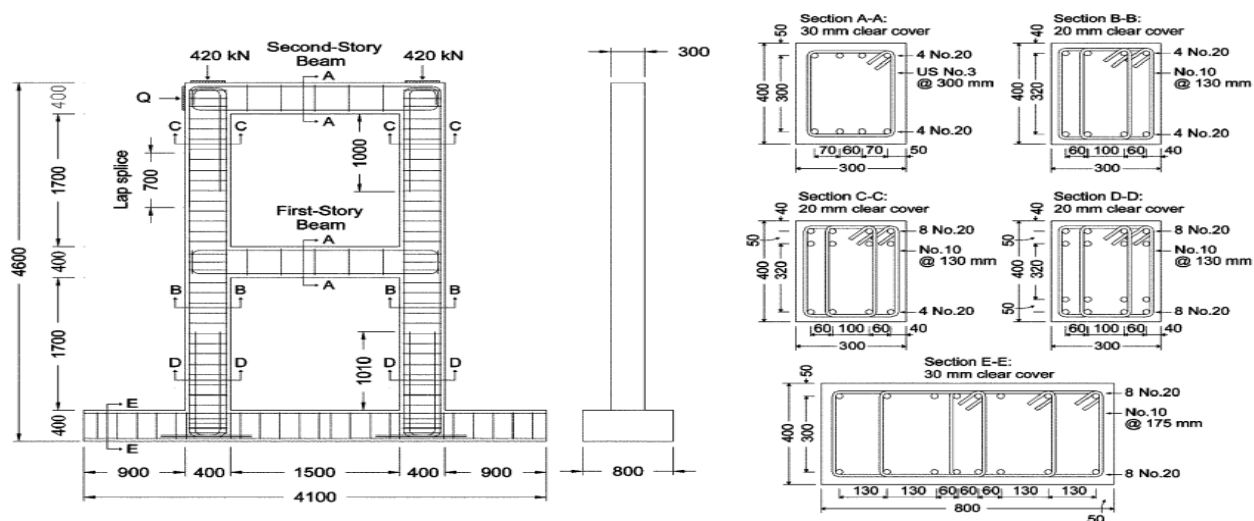


Figure B.3 Frame Reinforcement Layout and Cross Sections detail

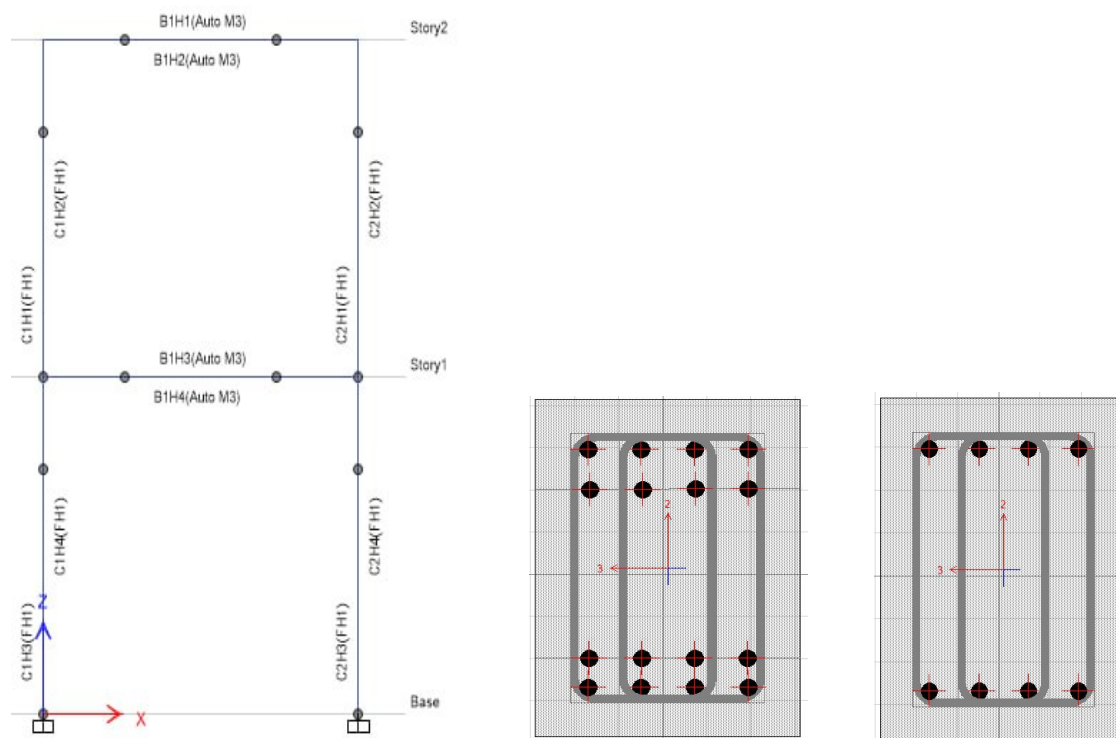


Figure B.4 Modelling of Geometry & Hinge Definitions, Section c-c, section D-D

By replicating the experimental conditions within the ETABS environment, the analysis aims to verify the software's capability to accurately predict the structural response, particularly for shear-dominated failure mechanisms.

Applying push over analysis on ETABS and the experimental result the ‘hinges’ model predicted a higher peak load (328.26 KN) than the experimental result (327.36 KN) this is a highly accurate estimate with a small deviation of -1.13%. However, this model also predicted a much smaller displacement capacity (34.016 mm) than that of the experimental result (44.7 mm).

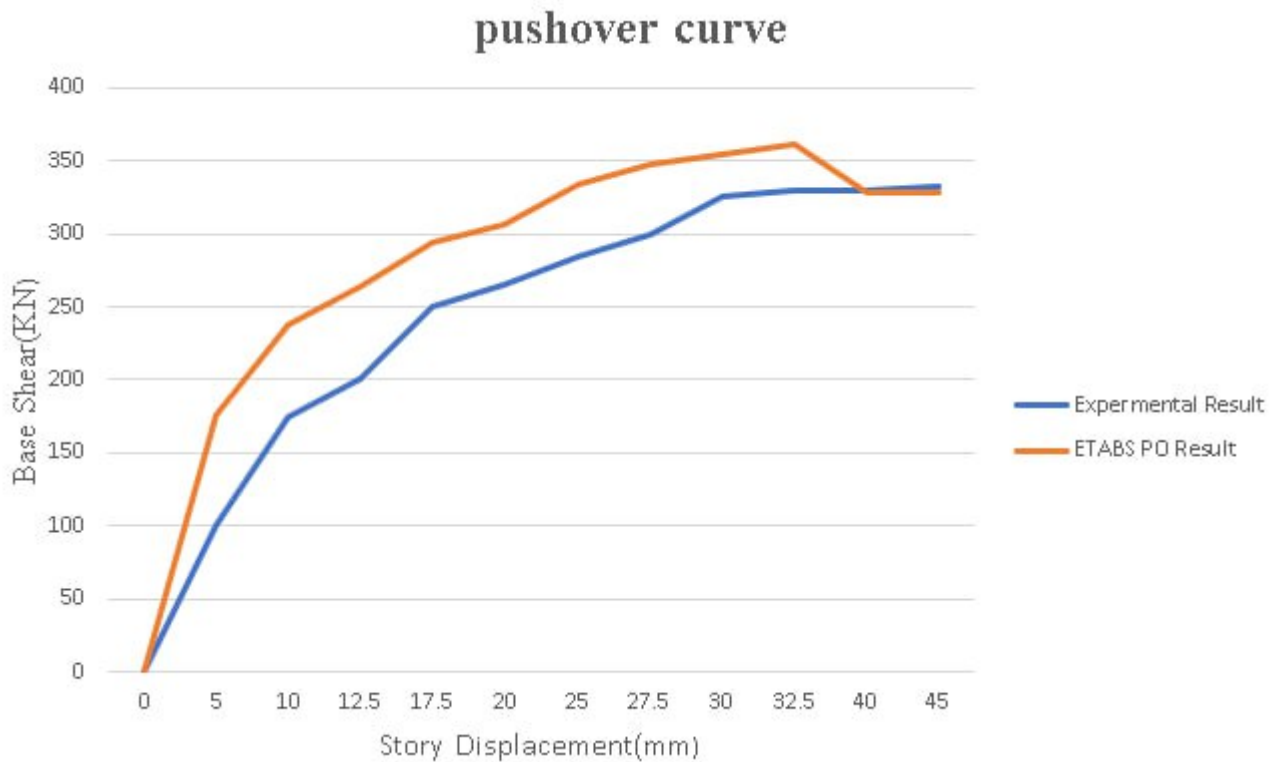


Figure B.5 Pushover curve