ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Seismic Performance Evaluation of Wall Equivalent Frame System Reinforced Concrete Building Structure.

A Thesis in Structural Engineering

By Danait Tsegaye

June, 2025

Addis Ababa

A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Seismic Performance Evaluation of Wall Equivalent Frame System Reinforced Concrete Building Structure**' presented by **Danait Tsegaye**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

Dr.Bedilu Habte	____________________	June, 2025
Advisor	Signature	Date
Dr.Ing.Adil Zekaria	____________________	June, 2025
Internal Examiner	Signature	Date
Dr.Esayas G/Yohannes	____________________	June, 2025
External Examiner	Signature	Date
Dr.Abrham Gebre	____________________	June, 2025
Chair person	Signature	Date

UNDERTAKING

I, certify that research work titled "**Seismic Performance Evaluation of Wall Equivalent Frame System Reinforced Concrete Building Structure**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Danait Tsegaye

Signature: _________

Date: June, 2025

ABSTRACT

These days, predicting inelastic seismic responses and evaluating a structural building's seismic performance are crucial topics. This thesis evaluates the seismic performance of wall equivalent frame system reinforced concrete building structures, categorized as low rise, medium rise, and high rise, with a particular emphasis on this idea. The pushover analysis approach is used to evaluate the nonlinear response of the RC buildings under consideration in this thesis. Various shear wall locations are used, along with the presence or absence of slab openings and the introduction of irregularities in plan and elevation.

Twelve Wall equivalent frame system reinforced concrete building models with ductility class medium have had examined their performance using the finite element analysis program ETABS 2021.1.1. Every model has been analyzed (response spectrum analysis) and designed (capacity design) in compliance with Ethiopian building codes and requirements. The performance for the models (G+5, G+15 and G+30) has been evaluated by discussing the results of the nonlinear static analysis.

The seismic performance evaluation is carried out for different shear wall location, presence of slab opening and introduction of irregularity. The building's top story displacement, inter-story drift, capacity curve and plastic hinge distribution are the parameters used to evaluate the seismic performance of the structures. Based on this, the Top displacement value shows increment in Y Direction for the parameters; placing shear wall at the center by 5% relative to the edge, effect of having opening increase the Top displacement by 2% and having a plan irregular building increase the result by 10%.Introducing Irregularity in the building decrease the performance level by 0.8%, 5%, 8% for low, medium and high rise building respectively and having shear wall at the center by 0.4%,4% and 6%, the least effect is shown for introducing opening which is 0.1%, 0.9% and 2% for the buildings under consideration.

Key words: Response spectrum analysis, Pushover analysis, Irregularity, Seismic performance

ACKNOWLEDGMENTS

My sincerest gratitude goes out to God Almighty for his boundless blessings, first and foremost.

I want to take this opportunity to express my sincere gratitude to Dr. Bedilu Habte in particular for his invaluable time spent helping me with my challenges and his modest demeanor in accepting my challenge and providing me with guidance.

Besides my advisor, I would like to thank Mr. Berhane Meskel E. and Mr. Asemerom Weldegerima for giving valuable time and continuous support.

My sincere thanks also goes to My father Ato Tsegaye Negash and My mother W/ro Nigisti G/eyesus for their unlimited encouragement and support.

In conclusion, there are no adequate words to convey my heartfelt gratitude and thanks to my family and friends for their wonderful support.

TABLE OF CONTENTS

LIST OF TABLE

LIST OF FIGURES

LIST OF NOTATIONS

ψ_{Ei} Combination coefficient for a variable action i, to be used when determining the effects of the design seismic action

$\psi_{2,I}$ Combination coefficient for the quasi-permanent value of a variable action i

q Behavior factor

Ag Design ground acceleration

DCM Ductility Class Medium

DCH Ductility Class High

EBCS Ethiopian Building Code of Standard

ULS Ultimate Limit State

IO Immediate Occupancy

LS Life Safety

CP Collapse Prevention

RC Reinforced Concrete

CHAPTER 1. INTRODUCTION

1.1 Background

In multistory reinforced concrete buildings, weaknesses in the lateral load-bearing components are usually the source of seismic damage. The behavior of multi-story wall-frame buildings during strong seismic events is determined by the distribution of mass, stiffness, and strength in the horizontal and vertical planes of the buildings. Weakness related issues are brought on by discontinuities in mass, strength, or stiffness along the diaphragm. These disruptions in the diaphragms are often associated with sudden changes in the building's frame geometry. Structural engineers now feel confident designing buildings with comparatively uniform mass, stiffness, and strength distributions. There is less confidence in the design of structures with irregular geometric layouts and diaphragm discontinuities.

Every structural element between separation joints needs to be connected in order to establish a continuous transmit to the lateral force-resisting system, and those connections need to be able to transfer the lateral forces that the connected elements cause. Any smaller structural component must be connected to the main structure with materials that are sufficient to handle the applied force.

Building structural irregularity is a serious issue that might cause disaster during a strong earthquake. While it is impossible to completely prevent irregularities in building construction, it is nonetheless important to study how these irregularities behave during earthquakes. It is necessary to precisely assess seismic demands in order to prevent damages caused by irregularity problems.

Within the engineering community engaged in developing seismic design procedures, there exists a widespread conviction that the traditional elastic analysis and design techniques fall short in capturing numerous crucial elements that govern the seismic behavior of structures during strong earthquakes. Furthermore, the other potent method, inelastic time-history analysis, is costly to compute and impractical in many situations. These days, engineers are looking for a method to address the aforementioned problems. [1]

Engineers may now compute design forces and displacements using either linear or nonlinear analysis thanks to modern seismic design codes. Specifically, there are four types of analysis found in Euro code 8, 2003: modal analysis, nonlinear pushover analysis, nonlinear time-history analysis, and simplified static analysis. Design engineers must have access to advance nonlinear methods and complicated models in order to apply the two nonlinear techniques correctly.

Simplified nonlinear static analysis, or pushover analysis, is a less complicated method of assessing a structure's performance; yet, it also requires the utilization of numerous complicated mathematical models of MDOF systems. Because of the method's relative simplicity, its application in practice is constantly growing. This approach makes the assumption that the first vibrational mode, or the first few, that occur during a structure's response period can be utilized for predicting the structure's response.

By applying loads incrementally in a predefined pattern, the method allows for the identification of the structure's failure modes and the creation of a force-displacement relationship, or capacity curve that clearly illustrates the nonlinear response. The structural performance of the elements is next evaluated and the displacement needs that resulted from the previous analysis are verified.

Introduction to Story height classification

Structures vary widely in height and dimension, each with a distinct function and set of architectural difficulties. The terms “low-rise”, “medium-rise” and “high-rise” are often used to categorize buildings based on their number of floors, and these distinctions have a significant impact on design, construction, and functionality.

Low-rise buildings

Typically occur in residential and suburban areas. They offer a more individualized and community-centered way of living. Because low-rise buildings are constructed using traditional materials and methods, they often integrate in with their surroundings. They are usually less expensive to build and maintain due of their size.

Medium-Rise Buildings

Medium-rise buildings, which usually are situated between low-rise and high-rise constructions. These structures are adaptable and can be used for both commercial and residential uses. Mid-rise designs enable denser urban growth without the excessive height of high-rises, thanks to the increasing prevalence of elevators. They frequently prioritize usability and accessibility in their design.

High-Rise Buildings

High-rise structures are those that rise above the cityscape. They represent modernization and urbanization; can be utilized for a range of uses from housing to offices. high-rise buildings are very complicated due to the huge number of structural components and elements unlike low-rise and medium-rise buildings, as well as these high-rise buildings demand high structural stability for safety and design requirements. Modern engineering techniques are frequently used in high-rise structures to withstand wind forces and create comfortable living and working spaces. [1]

The main idea of the paper behind the classification of building models as low, medium and high rise is; in order to create an approximate boundary condition regarding the seismic response of structures with different story height level.

In addition to this, based on this classification the purpose of the building, the load the structure experience, design level requirement and also its performance is way different one another. so, the grouping helps to have a closure boundary range regarding purpose, analysis, design and response of building structures.

The terms low rise, medium rise and high rise buildings refer to the vertical height of a building. The number of stories in a building helps in fulfilling the requirements for its design and safety evaluation.

The behavior of the low rise, medium rise, and high rise RC Wall equivalent frame system building models with regard to seismic performance is evaluated in this work. A wall equivalent frame system is a type of structural system with shear resistance of walls at the building base is higher than 50% of the total seismic resistance of the whole structural system. A 6-story, 16-story, and 31-story building model were considered to be low rise, medium rise, and high rise, respectively, for the purposes of the evaluation. ETABS 2021.0.0 was the software used for modeling, analyzing, and designing the buildings. In order to study and discuss the building models, criteria such as Top displacement, Story drift, Capacity curve, Performance point, and hinge formation were examined.

1.2 Statement of the problem

Since architects cannot avoid certain irregularities in the design and construction of buildings, it is necessary to investigate how these structures behave during earthquakes. These days, it's usual to see most structures with irregularities both in plan and elevation therefore, it is necessary to precisely assess the seismic demand. A stiffness irregularity resulting from variations in height, mass, and configuration has to be studied as the majority of researches now concentrate on the seismic response of regular structures and irregularities for low rise structures using frame systems. A gap exists in the analysis and evaluation on seismic response of the RC building structures incorporating wall system. Seismic response considering shear wall at different location with both in plan and elevation irregularities needs to be studied.

1.3 Objectives

1.3.1 General objective

The general objective of this study is to evaluate the seismic performance of a wall equivalent frame system irregular RC building structure with different story height level.

1.3.2 Specific Objectives

The specific objectives of this research are:

- To evaluate the effect of having different shear wall location on the performance of the structures.
- To evaluate effect of presence of opening in slab on seismic performance of the structure.
- To evaluate seismic demands for models with different story height due to change in configuration.
- To evaluate and compare the seismic performance of regular and irregular building.

1.4 Scope of the study

This thesis paper focus on evaluation of the seismic performance of low rise (G+5), medium rise (G+15) and high rise (G+30) RC building with wall equivalent frame structural system. A wall equivalent frame system is a type of structural system with shear resistance of walls at the building base is higher than 50% of the total seismic resistance of the whole structural system. Each story height is evaluated by introducing different shear wall location (at the core and edge), introduction of opening (to study the effect of presence of opening on seismic performance) and also by adding in plan and elevation irregularities. There is a total of 12 models considered to be analyzed using the linear dynamic method and carefully design prior to evaluation. ETABS 2021.0.0 software was used to conduct a nonlinear static (pushover) analytical approach performance evaluation of the structure.

1.5 Thesis organization

This thesis composed of five chapters. The first chapter is about a broad introduction, the necessity for the inquiry, its goal and scope. The second chapter gives an overview of earlier studies on irregularities related to mass and stiffness. Description of analytical models and methodology used for structural analysis is stated in the third chapter. The result of analysis is compiled and presented in appropriate manner in chapter 4. comparison between the result obtained from different shear wall location, plan regular building with and without opening and irregular building has been carried out. The seismic demands; floor displacement profiles, inter story drift, story shear, capacity curve and plastic hinge distribution for each RC buildings are presented and discussed. Chapter 5 contains the summary and the conclusions of the study, and the recommendations for future studies.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

Among the various natural disasters that have a negative influence on society and the economy, earthquakes are among the most devastating. An earthquake's potential for destruction can have a significant impact on infrastructure and lifelines. Therefore, following severe earthquakes that caused significant damage, deaths, and property destruction; researchers have been reevaluating the design techniques by using Earthquake engineering as the major tool. Within the field of civil engineering, seismic engineering examines how earthquakes affect structures and aims to reduce the number of deaths. This field has come a long way over the past 40 years thanks to the quick advancements in computers and computation, better experimental setups, and the creation of innovative techniques for seismic design and structural assessment. But this advancement hasn't come close to mitigating the devastating damage that earthquakes cause. However, because inelastic displacements are seen to be more indicative of various structural performance levels, it has resulted in a slight improvement of design and assessment techniques, moving away from conventional force-based procedures and towards displacement-based procedures.

Because of the needs for services and architectural design, structural irregularity in a structure is now fairly widespread. Therefore, it has become crucial to evaluate how well RC buildings perform during earthquakes. In addition, more irregular buildings are being built every day for architectural, economic, and a variety of other reasons.

To make sure that the structures withstand repeated minor earthquakes, we must be aware of the seismic capability of structural buildings and take safety measures when an earthquake happens. In order to save as many lives as possible. There are numerous such rules on this issue that have been modified frequently all over the world. A building's ability to withstand the effects of seismic activity depends on a number of variables including enough lateral strength, rigidity, flexibility, and structural layout.

Live load and dead load are typical gravity relate loads. A safe RC structure must have enough stiffness to withstand lateral and vertical loads. The performance of a building during an earthquake depends on a number of response factors, including adequate lateral strength, plasticity, stiffness, simple and regular configuration. Compared to a structure with an atypical layout, the structure with orderly geometry and evenly distributed stiffness and mass in elevation and plan sustains significantly less damage. [1]

Previous seismic events have showed that irregular buildings have underperformed and sustained more damage in comparison to their regular counterparts. This is because the seismic stresses that the ground motion imposes on the structure are often unevenly distributed throughout. This will ultimately result in damage and energy dispersion. This isn't the case, though, for irregular structures, where the concentration of seismic demands tends to occur in the irregularity zone, leading to significant damage and structural element failure in these regions. [1]

2.2 Introduction to Irregularities

Future research towards more practical and logical design processes will be crucial. Design has always involved striking a balance between truth and simplicity. Due to significant uncertainty in the imposed demands and available capacity, the first term, reality, appears to be exceedingly complex. The latter term, simplicity, is required due to computational expense and the constrained ability to execute complexity with the knowledge and resources at hand. [2]

The arrangement of a building's structural components determines its behavior. The building's size, shape, and geometry all play a significant role in determining the building's structural layout. Under dynamic loads, inertia forces build up and are concentrated close to a structure's center of mass. The forces generated by the resistance of vertical structures such as columns, to horizontal inertia forces are typically concentrated at a point called the center of stiffness. When the center of mass and the center of stiffness are not the same, eccentricity in the structure presents. The asymmetrical layout of the structural configuration leads to eccentricity, which twists the structure. [3]

Due to the rapid global expansion, it is common to see irregular buildings being built in every nation. During an earthquake, the building should not sustain any damage and should stay existing. [3]

Regular structures are those in which the configuration is nearly symmetrical, and irregular structures are those in which the building is irregular in the plan, load-bearing members, or elevation. Plan, elevation, and load-bearing member discontinuities should all be viewed as irregularities in a building. An irregular building is one that has different distributions of stiffness, strength, mass, etc. along its story height. [4]

2.3 Literature Review On Pushover Analysis

Within the field of civil engineering, seismic engineering examines how earthquakes affect structures and aims to reduce the number of fatalities. This field has come a long way over the past 40 years thanks to the quick advancements in computers and computation, better experimental setups, and the creation of innovative techniques for seismic design and structural assessment. However, this progress has not been sufficient to withstand the disastrous effects that earthquakes bring about. Nonetheless, it has resulted in a move away from conventional force-based methods and toward displacement-based procedures, which have been found to be more representational of varying structural performance levels. This has helped to enhance design and assessment processes.

Nevertheless, because forces and displacements are closely linked, it is still challenging to physically separate these processes. However, the most current approach to seismic design and assessment is performance-based earthquake engineering, which was made possible by the characterization of the various performance levels. [5]

The idea of acceptable levels of damage and performance level under the incoming earthquake motion is the foundation of earthquake resistant building design. The building's performance goal is determined by the client's and designer's requirements based on an acceptable degree of damage. An acceptable integrated probability of the building breaching specific limit states during the maximum designed earthquake events that the building is expected to experience in the designed period should be the performance specification.

It is a complicated process to specify an integrated probability, and the criteria are frequently limited to a given intensity. For instance, the goal could be to ensure that, in the event of an earthquake, the building remains completely operational with little to no damage. [6]

The widely acknowledged goals of a building's earthquake resistance design are to prevent structural damage from frequent earthquakes and to guarantee the users' and the public's life safety in the event of the largest possible incoming earthquake that the building may experience during its design life. Further performance goals should be established for the structures that require extra care. In order to protect human life, a structure's seismic design must take into account the incoming reference earthquake load, restrict structural damage to an acceptable level, and maintain the operational continuity of civil works, all of which are crucial for civil safety. Two fundamental design phases are involved in the seismic design of structures: first, calculating the resulting seismic force applied to the structure, and second, designing the structures to meet all national codes' requirements. [7]

Different limit states requirements are met in the design of seismic resistance structures in order to ensure the structure's good performance and to prevent undesirable damage. The first prerequisite for designing seismically resistant structures is the serviceability limit state, where regular, mildly intense earthquakes shouldn't interfere with the structures' ability to operate on a daily basis. Furthermore, structural damage does not require repair, nor do nonstructural elements. In order to provide sufficient strength to withstand the incoming seismic stress, the structural elements are also designed within a permissible story displacement and within an elastic range. For some buildings, including hospitals and fire stations, this limit condition is crucial. [8]

The structure sustains some damage when the damage control limit state is reached for ground shaking that is more intense than the equivalent serviceable limit state. It is necessary to verify this limit state requirement. The structures' strength, ductility, and potential second order effect must all fall within an acceptable range. Particular actions relating to the parameters that result in deaths need to be avoided even during high-intensity earthquakes, and crucial regions are described in detail to facilitate with the transfer of incoming seismic energy. [9]

The majority of structures are built with elastic behavior primarily for economical reasons. The degree of energy dissipation of structures, which is a function of their capacity to absorb and dissipate energy by ductile deformation; determines the response amplitudes of vibrations caused by earthquakes. Structural systems may experience stresses corresponding to comparatively significant lateral loads when they have low energy dissipation. In order for these structures to stay within the elastic range, they must be built to bear lateral forces equal to the same percentage of their weight. [10]

Using non-linear static analysis, the performance of a building structural can be examined. This entails estimating the requirements for structural strength and deformation and comparing them to the capacities that are available at the desired performance levels. Involves assessing and contrasting the performance of two reinforced concrete building systems utilizing various approaches, specifically those outlined in the FEMA-273, ATC-40, and Euro Code 8 design code, which use nonlinear static procedures with specified acceptance criteria. The approaches are used in the framework of performance-based seismic design methods for four and eight story frame systems that are both developed in accordance with Euro standards. According to the outcome, non-linear static analysis is more suited for high frequency and low rise buildings. For example, an elastic analysis may not reveal shortcomings in design like excessive deformation demands, strength deviations that are responsible for structures vibrating largely in the fundamental mode. [11]

In performance-based seismic design, predicting inelastic seismic responses and assessing a building structure's seismic performance are crucial topics. Nonlinear static analysis (pushover and modal pushover analyses) and nonlinear time history analysis are used to evaluate the seismic performances of RC buildings. Several significant effects are taken into account when creating a finite element model that can accurately simulate the nonlinear behavior of a building. These effects include p-delta, masonry in-fill walls, soil-structure interaction, and beam-column joints, which can be thought of as rigid zones where poor joint detailing can lead to joint failure. Both local and global effects, such as inter-story drift and system ductility demand are studied and done a numerical example on a nine-story reinforced concrete structure. Lastly, a comparison is made between the results of the nonlinear dynamic analysis of the MDOF system and the global and local responses derived from the modal pushover analysis. Comparing the results with the nonlinear dynamic analysis of the MDOF system, it is evident that the modal pushover analysis is accurate enough for real-world applications in seismic performance evaluation. [12]

Floor plays a significant part in carrying and distributing gravity loads. It also uses a diaphragm to distribute loads caused by earthquakes to systems that can withstand them. The in-plane flexibility of the floor diaphragms in RC buildings is commonly disregarded for practical design simplicity (i.e., the floor systems are often viewed as fully rigid diaphragms). When the diaphragm plan aspect ratio is more than 3:1, prior research acknowledged in modern building standards has demonstrated that this assumption can lead to a significant amount of error when forecasting the seismic response of RC buildings.

The architecture of two three-story reinforced concrete structures is based on the building frame system. Four cases two with and without floor openings are examined for each building. Under both static lateral loads (push-over) and dynamic ground motions (time-history), the inelastic behavior of the buildings is examined. To simulate modest ground motions, a suite of three well-known earthquakes is scaled. In the diaphragm parametric analysis, three different types of floor diaphragm models rigid, elastic, and inelastic are assumed for two opening sizes and locations as well as two lateral load-resisting frame stiffness and locations. It was determined that the adoption of an inelastic diaphragm model is required in order to effectively represent the seismic response of RC buildings with floor diaphragm openings. [13]

In the investigation of traditional structures' seismic behavior which is affected by the stiffness of their in-plane floors; both the connections between the horizontal diaphragms and the masonry walls as well as the in-plane stiffness of the floors have a significant impact on the structural behavior of an existing masonry building that is susceptible to seismic action. The specimens were chosen to be 5 m in length and 4 m in width. A particular test setup has been developed and implemented in order to permit the free in-plane deformation of the floor itself when subjected to lateral load, taking into account the size of the specimens and the requirement to ascertain the in-plane strength and stiffness of the floor. The load configuration applied to the floor replicates the impact of seismic action on the floor. The study seeks to calibrate engineered models for use in examining current structures. The outcome demonstrates that the structural behavior of an existing masonry building during seismic activity is significantly influenced by the in-plane stiffness of the floors. It outlines the request displacement for confirming the out-of-plane mechanism of walls as well as the seismic distribution of forces on lateral walls. [14]

A particular test setup has been developed and implemented in order to permit the free in-plane deformation of the floor itself when subjected to lateral load, taking into account the size of the specimens and the requirement to ascertain the in-plane strength and stiffness of the floor. The load configuration applied to the floor replicates the impact of seismic action on the floor. The experiment seeks to calibrate engineered models for use in examining current structures. The outcome demonstrates that the structural behavior of an existing masonry building during seismic activity is significantly influenced by the in-plane stiffness of the floors. It outlines the request displacement for confirming the out-of-plane mechanism of walls as well as the seismic distribution of forces on lateral walls.

When a local mechanism forms or a structure has been in place for a long time, there will be a visible mistake, and the dynamic features of the structure will alter correspondingly. The approximation of the distribution of the inertia force derived from the dynamic analysis results of the story equivalent MDOF system of the structure, which is time dependent, is used in this work to represent a lateral load pattern.

After that, the structure is examined part by part using the static pushover analysis until the top displacement meets the target value. The analysis of a number of frame structures shows that, compared to the general pushover analysis method, the presented method's calculations for base shear force and story displacement agree better with the results of dynamic analysis and require significantly less computer time. [15]

In a study on seismic performance of irregularly arranged, high-rise RC structures; One of the most destructive and unpredictable natural disasters is earthquakes. Nevertheless, because of their unpredictable nature, it can be challenging to stop the loss of life and damage to property when buildings are not built to withstand the forces of an earthquake. The diaphragm discontinuity and re-entrant corners in the frame structure are the two sorts of plan irregularities to study. These irregularities are the result of the IS code's Article 7.1. ETABS was utilized to assess a range of irregular models featuring reentrant corners and diaphragm discontinuity in order to ascertain the building's seismic reaction. Static and dynamic approaches were employed to analyze the models, with displacement, base shear, and fundamental natural period being the parameters taken into consideration. The model that is most likely to fail in an extremely seismic zone is identified from the current study, and ETABS is used for modeling and analysis. The outcome shows that irregular structural configurations are significantly impacted by earthquakes, particularly in high seismic zones, and that the response spectrum method's results are accurate when compared to those of an equivalent static method as it solely uses empirical formulas. [16]

The different distribution of mass and stiffness affect the elastic and in elastic response of multi-story buildings. It was observed that, in contrast to high rise (ten story) buildings, the type of earthquake record had no discernible impact on the response of low rise (5 story) buildings. Additionally, it was shown that a 20% to 100% increase in first story drift occurred when first story stiffness was reduced by 17% to 67%. Generally speaking, it was found that when a structure's mass and stiffness varied continuously throughout its height, the structure performed well. [17]

In the study how reinforced concrete structures with vertical imperfections behaved during earthquakes; sixteen-story buildings with three distinct horizontal layouts and five different vertical arrangements of were examined. The structures were intended as a collection of stiff diaphragm-connected planar moment-resisting frames with shear walls. The ductility requirement in the shear wall was significantly raised by a discontinuity in the frame. Furthermore, with the exception of the stories right above a discontinuity, where there was a noticeable rise in frame ductility demand, the distribution of ductility demand was erratic in shear walls but reasonably regular in the frames. The ductility requirements for irregular buildings were found to be almost twice as high as those for regular buildings. The shear wall generally showed an increase in ductility demand if the irregularity was found in the frame, and vice versa. [18]

The effect of mass irregularity on seismic performance of structure showed that it was not significantly impacted by mass irregularity, assuming that all column ends had plastic hinges and that beams were rigid. Furthermore, they demonstrated that an increase in mass at the top floor of the building had a comparatively greater impact on storey drifts than an increase in the middle or bottom floors. [19]

Evaluation on the impact of vertical irregularities had been made by taking height-related changes in seismic demands into account. For the parametric analysis, they employed a model of a 10-story building constructed in accordance with the strong-beam-weak-column (column hinge model) concept and a collection of 15 strong ground motions that were recorded on rock or firm soil during Western U.S. earthquakes before 1983. The seismic response of irregular structures was evaluated through the use of elastic and inelastic dynamic analyses. The impacts of vertical irregularities in the distributions of mass, stiffness, and strength were taken into consideration individually and in combination. The effects of mass irregularity are determined to be the least, strength irregularity to be larger than stiffness irregularity, and combined stiffness and strength irregularity to be the largest. The vertical irregularity has little effect on roof displacement. [20]

Study on irregular reinforced concrete frames nonlinear response; the two unevenly tall reinforced concrete frames were idealized as single stick models with masses accumulated at the floor levels. It was discovered that an irregular elevation causes a two-fold increase in the ductility demand. Where the stiffness distribution abruptly changed along the building's vertical height, this effect was more noticeable. [21]

By taking low rise buildings, notably the Addis Ababa condominium building as a case study, nonlinear analysis was used to examine the performance of RC planar frames and arrive at the following conclusion.

- The structural performance of existing condominium buildings is negatively impacted by the revised building code's (EBCSEN-2014) increased peak ground acceleration value. As a result, these buildings need to be thoroughly examined and appropriate mitigation measures taken to prevent catastrophic failure in the event that the expected seismic excitation occurs.
- Pushover analysis is useful for both detecting progressive hinge creation and anticipating the structures' real-time response. The progressive hinge construction is a useful tool for purposes of strengthening.
- The definition of a plastic hinge determines how well a structure performs; using a plastic hinge zone improves the structure's performance when compared to using hinge points. [22]

Pushover analysis's simplicity is making it more and more popular. In stiff and irregular structures, nonlinear effects and high frequency modes could be significant factors. In pushover analysis, the role of higher modes is not yet fully realized. It is unknown how high frequency model responses behave in nonlinear seismic analysis of buildings. The behavior of high frequency model responses in nonlinear seismic analysis of structures is studied.

An increasingly common method among engineers, researchers, and professionals worldwide is nonlinear static pushover analysis, which is used as an approximation to nonlinear time history analysis. High frequency modes have the potential to make a substantial contribution to the seismic analysis of stiff and irregular structures. Structural engineers may incorporate high frequency modes in the non linear static pushover analysis to account for the contribution of higher modes. In non linear static pushover analysis of irregular structures, the behavior of high frequency modes is examined. The nonlinear dynamic analysis's responses converge to the nonlinear static pushover analysis at high frequencies. Consequently, a non linear static push over study using an empirical force pattern determined by the modal mass contribution times zero period acceleration can be used to assess the nonlinear response of high frequency modes. When compared to the damped periodic modes, the higher modes with stiff content as a significant contributing factor show improved accuracy in non linear pushover analysis of structures. [23]

Post-elastic deformability is a tool for redistributing stresses in a structure to increase the ultimate load but, more importantly, to absorb and dissipate energy during major earthquakes. The ductility of RC elements has been extensively studied both experimentally and theoretically since its evaluation is fundamental to carrying out a reliable non-linear analysis of structures. The issue is still unresolved, and models still require advancement in two areas. On the one hand, constructive features, shear-flexure interaction, size effects, and non-linear material constituent relationships and the steel-concrete bond can all be accounted for by mechanical models.

However, in order to provide a simple yet trustworthy ductility evaluation without the need for any complex analytical models which are typically not particularly designer-friendly; simplified alternatives must be evaluated. In order to expand on a trustworthy formulation for the plastic hinge length of R.C columns subjected to axial and flexural load, a broad parametric analysis using an improved model is conducted. A point-by-point model that includes an explicit formulation of the bond slip relationship and is able to account for the effects of concentrated and distributed non-linearity, such as the spread of plasticity along the member and the fixed end rotation, was used to analyze the non-linear behavior of the element and estimate the plastic rotation.

The model's evaluation of rotational capacity with various parameters provides a comprehensive grasp of the future influences pertaining to the structural problem. The behavior of the cracked section, which is well characterized by the moment curvature relationship, determines the ductility of R.C. components; section ductility is defined as the ratio of the ultimate curvature to the one at initial yielding. The simplicity of the technique makes models based on the evaluation of a plastic hinge length highly helpful if the rotational capacity needs to be evaluated in real-life circumstances. [24]

Despite not frequently being taken into account during structural analysis and design, infills have a significant impact on how infilled frame structures behave seismically. The strut model is used to do a parametric analysis of specific infilled frames in order to capture the overall effects of the infills. Three five-story concrete planar frames with three bays each are under consideration; their designs adhere to Turkish codes. The assessment of the frames' seismic response employs pushover analysis. Every frame undergoes four distinct loading scenarios. The cases' outcomes are briefly discussed and contrasted. It was looked at how two sample frames with varied infill arrangements behaved seismically in relation to the infill walls. The findings show that masonry infills must be taken into account when evaluating moment-resisting reinforced concrete frames seismically, particularly when predicting the structure's final state. Infills that have a uniform elevation also benefit buildings and seem to have a major impact on lowering global lateral displacements.

Pushover analysis can be thought of as a sequence of incremental static assessments performed to investigate the deformation and damage pattern, as well as the non-linear behavior of the structure. There are two steps to the process. The structure's target displacement is first determined. When a building is subjected to the intended seismic excitation, its seismic top displacement is estimated using the target displacement. After that, the structure is subjected to a pushover analysis until the displacement at the top of the building reaches the desired displacement. It is believed that the degree of damage incurred by the structure at the target displacement is indicative of the damage incurred by the structure under design-level ground shaking. For the aim of evaluation, a decision is made regarding the degree of damage to an existing structure or whether the structural behavior of the new building is acceptable.

The findings lead to the following conclusions.

- When evaluating moment-resisting RC frames for seismic activity, it is crucial to take into account the impact of masonry infills, particularly when estimating the final state of the structure.
- Infills with a uniform elevation have a positive impact on buildings. The levels where the discontinuity occurs are where damage was discovered to concentrate in infilled frames with irregularities, such as soft stories.
- As infills increase the frames' initial stiffness and lateral resistance, they seem to have a major impact on lowering the global lateral displacement. [25]

On the study of various methods of analysis, including response spectrum analysis, modal pushover analysis, and conventional pushover analysis, for low- and mid-rise concrete moment frame structures on stiff soil that have plan irregularities.

The findings indicate that;

- The height-wise distribution of storey shear, storey displacement, and inter-storey drifts predicted by MPA is better than the traditional pushover analysis (first 'modal' pair) result when a sufficient number of 'modal' pairs are included.
- Realistic results are obtained using conventional pushover analysis for five-story regular RC structures, fair results for ten-story regular RC buildings, and bad results for ten-story irregular RC buildings. As a result, the CPA's accuracy declines with increasing building height and irregularity.
- In the majority of MPA scenarios, the first two 'modal' pairs' contributions are sufficient and meet the targeted goals.
- The chosen RC buildings satisfied the seismic code's limitations on design and serviceability (drift).
- When RSA and MPA techniques employ the same number of modes, MPA can yield better results for both regular and irregular buildings that are pushed into the inelastic range.[26]

A new RC frame has been evaluated for its applicability using the nonlinear static analytical approach (Pushover), which was proposed by ATC-40. When RC frames were subjected to a moderate seismic loading, potential structural irregularities were assessed using pushover techniques and the code's seismic-resistant design. Using a single seismic combination, the design was redesigned in the first method to determine which members would need more reinforcement. The majority of the columns were found to be vulnerable to seismic forces and to require a large amount of additional strengthening. Conversely, the nonlinear pushover approach demonstrates that the frame can sustain the estimated seismic force at all beams and one column with a considerable amount of yielding. The location of vulnerabilities in the two techniques varies dramatically. The apparent mismatch, which stems mostly from the software's implementation of the method's default assumptions versus the code's assumptions about maximum allowable limits and reduction factors are examined. The code always preserves a certain level of safety in the design of new buildings, derived from load factors, materials reduction factors, and the omission of some post-yielding characteristics (hardening). Hardening must be taken into account, and a reduction factor of one is assumed in the ATC-40 modeling assumptions. Therefore, the article recommends that engineers use caution when doing pushover analysis, adhere to code restrictions when constructing new buildings, and, depending on the circumstances, implement specific reductions and limits when designing existing buildings. To put it briefly, software should not take the place of engineering judgment and code requirements.[27]

2.4 Research Gap

Prior from different related reviews although there are studies on the areas of RC building with irregularities, Studies on RC building with dual system having both in plan and elevation irregularity, considering different shear wall location with different story height seems limited. Also repeated studies were done on seismic response of regular structures with either frame or wall system. On this study, the findings go a long way towards filling the shortfalls regarding the response of RC building structures with both frame and wall system, dealing with both in plan and elevation irregularity by placing shear wall at different location.

CHAPTER 3. METHOD OF STRUCTURAL ANALYSIS AND DESIGN

3.1 General

Seismic analysis is a key method in earthquake engineering that helps to forecast how buildings will respond to seismic excitations in a more understandable way. Buildings were formerly just intended to withstand gravity forces, and seismic analysis is a more modern innovation.

The standard elastic analysis and design methods, according to the engineering community involved in the development of seismic design processes, are generally thought to be unable to capture many significant factors that govern the seismic performance of structures in strong earthquakes. Inelastic time-history analysis is another effective approach, although it is computationally costy and not practical in many situations. Design has always involved striking a balance between truth and simplicity. Due to significant ambiguities in the imposed demands and accessible capacity, the latter term, reality, appears to be exceedingly complicated. The first term, simplicity, is required due to computational expense and the constrained ability to execute complexity with the knowledge and resources at hand.

There are numerous available approaches that can be used to estimate demands. The main goal is to identify forces and deformations both globally and locally when the structure is experiencing seismic loads that represent the risk at the construction site. There are so four approaches that can be used to examine a mathematical model of a building structure. Depending on how the response or the loads are treated, they can be divided into two major types. One distinguishes between static and dynamic application of the seismic loads, the other results in a differentiation between linear and nonlinear techniques of analysis.

Linear static and dynamic procedures; These methods are recommended when high-mode and torsion effects are negligible, similar to those of ordinary buildings. The underlying assumption is that the displacements estimated with linear equivalent elastic stiffness will roughly match the displacements that could occur inelastically under the design loads.

Nonlinear static and dynamic procedures; Nonlinear approaches are generally applicable to all types of structures, with the exception of buildings where high-mode effects are small. The FEMA document states that high mode effects are significant if the shear in any story that results from a modal analysis that considers the modes necessary to obtain 90% mass participation is greater than the corresponding story shear that considers only the first-mode response by a factor of 1.3.[28]

3.2 Analysis and Design of structures

A Dual system building structures with different shear wall location, introduction of opening and irregularity are evaluated in order to assess the seismic response. In this study, the reinforced concrete buildings G+5, G+15 and G+30 for mixed-use buildings are chosen. The buildings are composed of both in plan and elevation irregular and also plan regular and elevation irregular one; it is supposed that all of the floors are rigid. Every building structure has been examined in accordance with EBCS EN 1998-1, 2014. The analysis is conducted using ETABS 2021.0.0 Software. The structure is a hypothetical one which is prepared for this project. The structures were assessed for ductility classes medium (DCM) and designed accordingly.

3.2.1 Modelling

Structural analysis and building modeling were done with ETABS 2021.0.0 software. For the frame and wall section properties; the section size, material properties, and section property modifier were determined. The section's ductility requirement and the incoming design load determined the section's size.

In this study, a stiffness modifier for the section's elastic flexural and shear properties was applied; it was equal to one-half of the section's gross stiffness when it wasn't fractured. In the load pattern option of the ETABS 2021.0.0 program all incoming loads, including dead load, live load and seismic loads as well as imperfection loads were defined.

In this research Response spectrum method of analysis was used. All the requirements used for linear dynamic analysis method were satisfied; like contribution of higher modes for the structural response was considerable. All possible load combination was defined to get maximum action effect of the structural elements.

Table 3. 1 List of Materials Used for the Analysis and Design

Structure	Concrete	E of concrete	yield strength of steel	E of steel
Beam	C20/25	30Gpa	450	200Gpa
Column	C25/30	31Gpa	450	200Gpa
Wall	C25/30	30Gpa	450	20Ggpa
Slab	C20/25	30Gpa	450	200Gpa

Table 3. 2 List of Parameters and Software Used for the Analysis and Design

Parameters	**Value**
Peak ground acceleration a_g	0.1g
Importance factor	1
Damping ratio	5%
Earthquake zone	Zone 3
Soil class	C
Structure type	Wall equivalent frame system
Modulus of elasticity of concrete	31Gpa (C25/30)and 30Gpa (C20/25)
Modulus of elasticity of steel	200Gpa
Spectrum Type	1
building type	multi-bay and multistory
Regular in plan	irr/regular
Regular in Elevation	No
Software used	Etabs 2021.0.0

3.2.2 Geometry and description of the building

Figure 3.1-3.4 below shows the building's general layout. The Ethiopian building code's requirements for medium ductility classes and the incoming load were taken into consideration when choosing the section size used in this thesis. The rest 3D and plan drawing is shown in appendix B.

Table 3. 3 Cross sectional dimension designed as DCM

Section	Structure	G+5		G+15		G+30
		Section Size(cm)		Section Size(cm)		Section Size(cm)
Column	G – 3	50*50	G -3	90*90	G – 2	150*150
	3 – roof	40*40	4 - 7	80*80	3 – 6	140*140
			8 - 11	70*70	7 – 10	130*130
			12 - 14	60*60	11 – 14	120*120
			15 - roof	50*50	15 – 19	110*110
					20 – 23	100*100
					24 – 27	90*90
					28 – roof	80*80
Beam(b*d)		30*50,25*40		30*50,30*40, 25*40		40*60,30*50,25*40,25*35
Wall (mm)		200		400		600
Slab (mm)		200		200		200

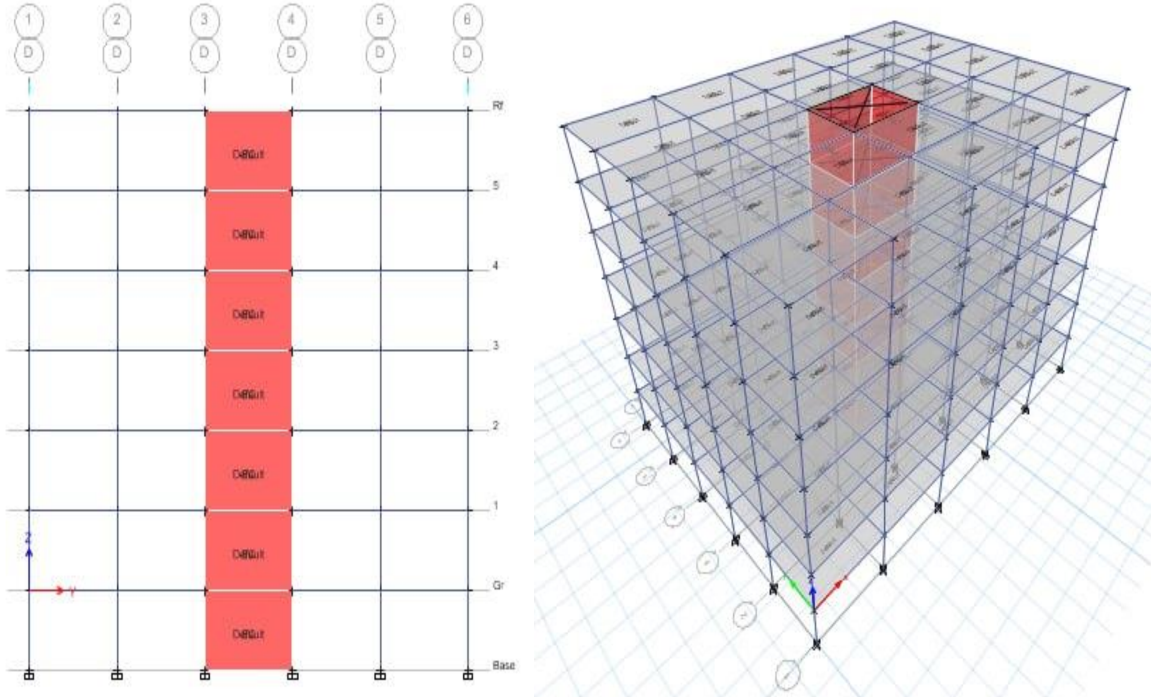

Figure 3. 1 G+5 building with shear wall at the center

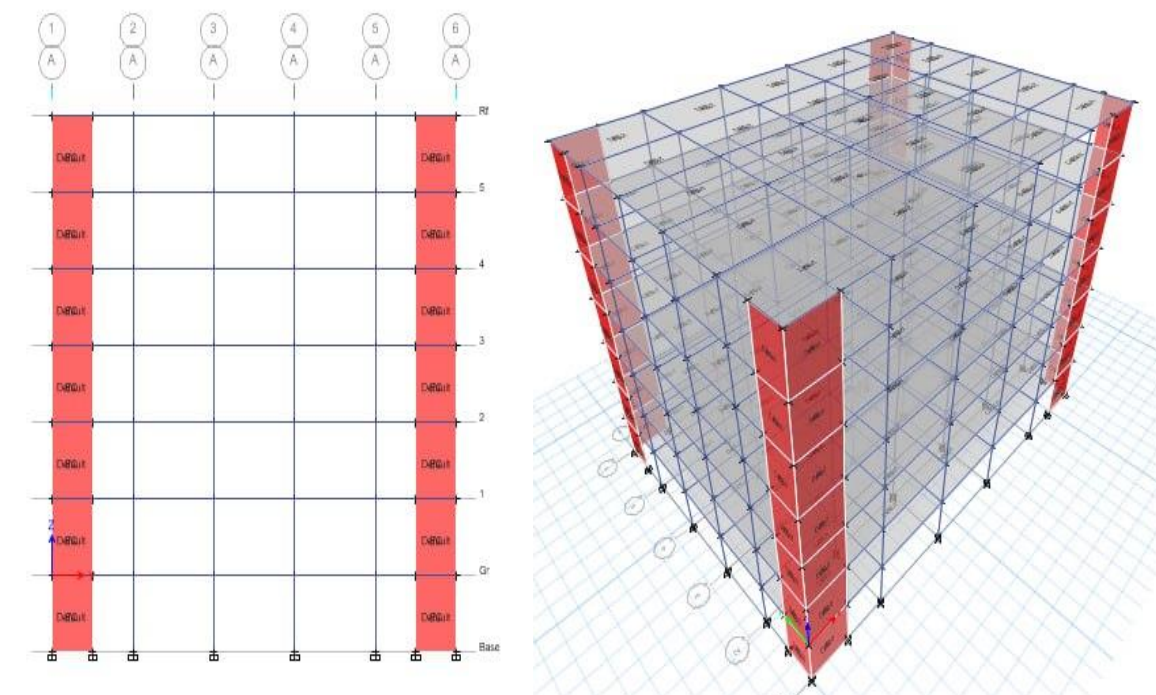

Figure 3. 2 G+5 building with Shear wall location at edge with no opening

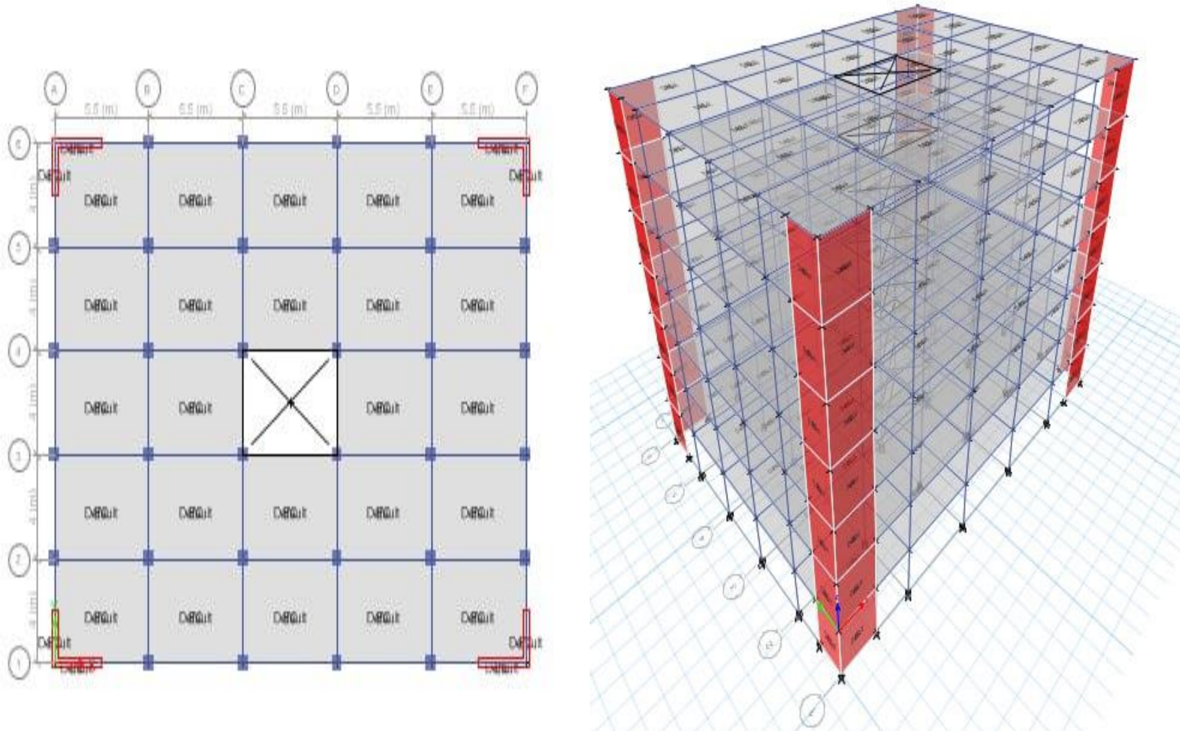

Figure 3. 3 G+5 building with Shear wall location at edge with opening

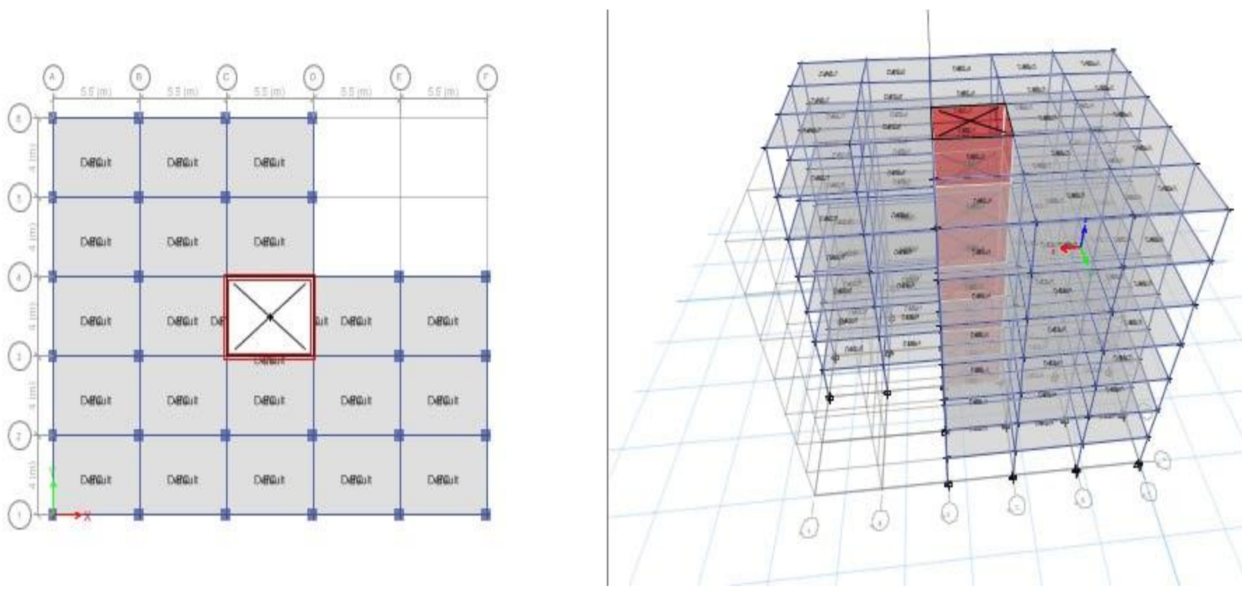

Figure 3. 4 G+5 Irregular building with Shear wall location at the center

Table 3. 4 Description of codes used

Code	No of stories	No of span to x-direction	No of span to y-direction	Bay width in x direction	Bay width in y direction	Description
G+5	6	6	5	5.5	4	Story six building
G+15	16	6	5	5.5	4	Story sixteen building
G+30	31	6	5	5.5	4	Story thirty one building
SWC						Shearwall located at the core
SWE						Shearwall located at the edge
SWE OS						Shearwall located at the edge with slab opening
SWE CS						Shearwall located at the edge with no slab opening
SWC PR						Shearwall located at the core with plan regular and elevation irregular
SWC PI						Shearwall located at the core with plan irregular and elevation irregular
						SWC= SWC PR , SWE= SWE OS

3.2.3 Loads and load combinations

Gravity Load

The design gravity loads are computed using the dead load and live load. The weight that came up from the self-weight of the structural elements is known as the dead load. The self-weights of the structural elements are influenced by the dimensions and tensile strength of the material employed in the model.

Normal weight concrete employed in this study has a characteristic strength of 24 kN/m^3. Based on building code (ES EN 1991-1-1, 2015) the imposed load is calculated by multiplying it by the floor's area to determine the gravity load.

Dead Load

The load that results from the structural components used in the model's self-weight is known as the dead load. The size and characteristic strength of the material used for the model determine the self-weights of the structural parts.

Table 3. 5 Dead load

Material	Size(m)	Unit Weight(kN/m^3)	Load(kN/m^2)		
Cement screed(t=3cm)	0.03	23	0.69		
Marble ff(t=2cm)	0.02	27	0.48		
Bottom plastering(t=1.5cm)	0.015	23	0.345		
Concrete		24			
		Sum	1.516		
Section	Size(m)	Thickness(m)	Number of slab/floor	load(kN)	Load(kN/m^2)
Slab	5.5*4	0.2	24	105.6	4.8
				Total	6.295

Imposed load used for the determination of gravity load is calculated multiplying by the area of the floor.

Live Load

The structure's function determines the live load of the structure. This thesis makes use of mixed-use buildings and is based on the 2015 Building Code (ES EN 1991-1-1).

Earthquake Load

The design earthquake force is determined by the seismic weight of the structure and the analysis method employed. The software ETABS 2021.0.0 is used to perform the earthquake load. The following describes an example of calculating and analyzing seismic weight.

Seismic Weight Determination

The weights used to calculate seismic loads are comparable to those used to calculate gravity loads. Column self-weight in any story must be combined with beam self-weight present in that story and transferred evenly to the floors above and below the story. The seismic load is calculated using the reduced live load.

The Combination coefficient is determined based on the function of the building from ([ES EN 1998-1], 2015.

Combination coefficient, ψ_{Ei}, calculated from the following expression:

$$\psi_{Ei} = \varnothing \cdot \psi_{2,i} \qquad (3.1)$$

Where Ø ; value from Table B.1 of this paper

$\psi_{2,i}$; Recommended values for buildings from Table B.2 of this paper

Lumped weight, which acts at the floor's center of mass, is the seismic weight of the structure.

Table 3. 6 Load cases

Load case name	Load case type	Function	Load type	Scale Factor
Dead	Dead			1
Live	Live			1
RSXT		Response Spectrum	Acceleration(U1)	9.80675
RSXB		Response Spectrum	Acceleration(U1)	9.80675
RSYR		Response Spectrum	Acceleration (U2)	9.80675
RSYL		Response Spectrum	Acceleration (U2)	9.80675

Load combination

According to ES EN 1998-1:2015 for linear static analysis and design, the code recommended the following load combination; PD refers to gravity for seismic combo considering Dead load and the likelihood of the Live loads not being present over the entire structure during the earthquake.

PD=DL + %LL

Table 3. 7 Load combination

Number	Load Combination	load case
1	PD+RSXT+/-0.3RSYL+/-IMP X	Seismic case
2	PD+RSXT+/-0.3RSYR+/-IMP X	Seismic case
3	PD-RSXT+/-0.3RSYL+/-IMP Y	Seismic case
4	PD-RSXT+/-0.3RSYR+/-IMP Y	Seismic case
5	PD+RSXB+/-0.3RSYL+/-IMP X	Seismic case
6	PD+RSXB+/-0.3RSYR+/-IMP X	Seismic case

7	PD-RSXB+/-0.3RSYL+/-IMP Y	Seismic case
8	PD-RSXB+/-0.3RSYR+/-IMP Y	Seismic case
9	PD+RSYL+/-0.3RSXT+/-IMP X	Seismic case
10	PD+RSYL+/-0.3RSXB+/-IMP X	Seismic case
11	PD-RSYL+/-0.3RSXT+/-IMP Y	Seismic case
12	PD-RSYL+/-0.3RSXB+/-IMP Y	Seismic case
13	PD+RSYR+/-0.3RSXT+/-IMP X	Seismic case
14	PD+RSYR+/-0.3RSXB+/-IMP X	Seismic case
15	PD-RSYR+/-0.3RSXT+/-IMP Y	Seismic case
16	PD-RSYR+/-0.3RSXB+/-IMP Y	Seismic case
17	DL+LL	Serviceability limit state
18	1.35DL+1.5LL+IMPX	Static case
19	1.35DL+1.5LL-IMPX	Static case
20	1.35DL+1.5LL+IMPY	Static case
21	1.35DL+1.5LL-IMPY	Static case

3.3 Imperfection

Frame and member defects (local and global) needs to be considered in either the analysis stage or the design stage according to Euro codes 2 and 3.

In the analysis phase of a numerical model, accounting for both global and local imperfections can be achieved in one of two ways:

- Clearly representing the global and/or local imperfections in the model
- Applying a horizontal uniformly distributed load throughout the vertical member length for local imperfections
- Equivalent Horizontal Force is applied to every story to address global imperfection.
- The initial geometry for Euro code three can be determined by using the buckling mode forms.

3.3.1 Geometric imperfections

ES EN 1992-1-1:2015. The adverse impacts of any imperfections in the geometry of the structure and the placement of loads must be taken into account while analyzing members and structures. An tendency may stand in for imperfections., θi, given by:

$$\theta_i = \theta_0\, \alpha_h\, \alpha_m \qquad (3.2)$$

Where;

θ_0 ; is the basic value given as 0.005 (= 1/200)

α_h ; is the reduction factor for length or height:

$$\alpha_h = 2/\sqrt{l}\,; 2/3 \leq \alpha_h \leq 1 \qquad (3.3)$$

l is length

α_m is the reduction factor for number of members

$$\alpha_m = \sqrt{0.5(1 + 1/m)} \qquad (3.4)$$

m is the number of vertical members contributing to the total effect

Horizontal load resulting from a geometric imperfection applied to the diaphragm's center, and the loads in each of the two main directions computed as:

$$H = \theta_i\,(N_b + N_a)/\,2 \qquad (3.5)$$

where

N_a and N_b are longitudinal forces that contributing to lateral force.

H_i horizontal load due to geometric imperfection in both principal direction that applied on center of diaphragm.

The applied imperfection load is mentioned on appendix C Table C. 2

3.4 Behavior Factor

In order to account for the structure's ability to dissipate energy, which is mainly achieved by the ductile behavior of its parts and/or other mechanisms, an elastic analysis is carried out using a response spectrum that has been decreased in relation to the elastic one; this is referred to as a "design spectrum." By doing this, the behavior factor is defined and explicit inelastic structural analysis is avoided in the design process. To accomplish this decrease, the behavior factor q is added.

The behavior factor q is introduced in order to achieve this reduction. The following equation defines the behavior factor's value.

$$q = q_o \, k_w \qquad (3.6)$$

Where;

q_o =is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation

k_w= is the factor reflecting the prevailing failure mode in structural systems with walls.

Table 3. 8 Behavior factor value used

Parameter	Value
q For plan regular	2.88
q For plan irregular	1.6

3.5 Description of the study

Studies for G+5, G+15, and G+30 buildings with varying story levels have been conducted. These studies are thought to verify the behavior of a structure with varying shear wall locations (at the core and at the edge), for the introduction of an opening for shear wall locations at the edge, and, finally, for the introduction of plan irregularities on the existing elevation irregular structures. Based on these criteria, the structures' responses are displayed as follows;

- Maximum lateral Top displacement.
- Inter story drift.
- Capacity curve.
- Plastic hinge distribution.

3.6 Description of building structure

Three distinct story level Wall equivalent frame system RC building structures are selected and studied as G+5, G+15, G+30 buildings in order to achieve the study's goal with a story height of 3meter. A dual system is a structural system in which structural walls and the frame system work together to resist lateral loads. There is a total of 12 models with;

1.Plan regular and Elevation Irregular wall equivalent frame system Rc building structure with different shearwall location; at the center and at the edge for each story height.

2.Plan regular and Elevation Irregular wall equivalent frame system with shear wall location at the edge.by preparing models with the presence and absence of opening at the center for each story height.

3.A wall equivalent frame structural system with shear wall location at the center. By preparing models with Plan regular and Elevation irregular one and both in Plan and Elevation irregular for each story height.

The structures that was chosen has the same type and thickness of slab, construction location, grade of concrete and rebar, loading condition and service, and floor height as per required. These structures were specifically chosen so that the parameters on the building could offer a crucial structural arrangement for evaluating the previously mentioned factors.

For the purposes of this study, the wall equivalent frame system with a medium ductility class and a design bed rock acceleration value of 0.1g is the structural system chosen for these buildings.

3.7 Analysis of the structure

Following the modeling of each building structure and the loading of expected loads, a preliminary design is carried out for each model using Response spectrum analysis approach. The related ultimate and damage limit state requirements of the ES EN 1998:1-2015 code are verified and kept to following the completion of Response spectrum analysis. Results following Response spectrum analysis(for checking plan and elevation regularity and also structural stability) like; Story stiffness check, Story drift check, Torsion check has been done and mentioned in appendix C. Table C.3-C.6

The ductility class for each models dictate the size and stiffness of the elements, which were then achieved through an iterative procedure that kept the optimized sections within the bounds specified by the design codes. Every time, the elements are chosen to meet every criteria for capacity design. In order to have a safe structure that can withstand loads, Each element of design data, including longitudinal reinforcement, shear reinforcement, beam column capacity, tensional reinforcement, and shear failure, is verified to be within the required range.

3.8 Design of structures

Capacity design is the process of designing the flexural capacities of major sections of a building structure based on an assumed behavior of the structure in response to seismic activity. The assumptions that plastic hinging happens concurrently at predefined locations to create a collapse mechanism following ductile behavior and that the seismic action is of a static equivalent type increasing gradually until the structure reaches its state of near collapse reflect this fictitious behavior.

The actual behavior of a building structure during an intense earthquake differs greatly from what has been previously reported, with plastic hinging occurring fairly randomly and seismic activity having a vibratory nature. But as several powerful earthquakes have demonstrated, it is thought that by incorporating the Capacity Design idea into the flexural component design, the structure will have sufficient seismic resistance.

The ductility level of the structure, as expressed by the displacement ductility factor or, more succinctly, ductility factor, is a characteristic of the Capacity Design concept. This is the ratio of the structure's lateral displacement caused by the Design Earthquake at the point of initial yielding to that at the point of near collapse.

Strong column and weak beam design is the foundation of capacity-based architecture. The various beams and columns serve as conduits through which the seismic inertia forces generated at its floor levels are transmitted to the earth. While the collapse of a beam has regional effects, the collapse of a column might compromise the integrity of the entire structure. Therefore, it is preferable to make beams than columns the ductile weak links. The strong-column weak-beam design method is what this approach to RC building design is known as.

3.8.1 Design action effects

Using linear dynamic analysis of a chosen building structure and the exposure of the seismic effect, the values of the bending moment, axial force, and shear force are determined. These fundamental seismic beam and column design values are ascertained as follows:

1. Beam

For primary seismic beams, the capacity design rule must be used in determining the design shear forces. This is accomplished by considering the equilibrium of the beam under the transverse load applied in the seismic design scenario, as well as the End moments Mi, d (where i=1, 2 denotes the end sections of the beam), which are associated with the formation of plastic hinges in the seismic loading's positive and negative directions.

It is important to take attention of where the plastic hinges should form-at the ends of the beams, or, if they form first, in the vertical elements attached to the joints that the beam ends frame. The following can be used to calculate end moments Mi,d:

$$M_{i,d} = \gamma_{Rd}\ M_{Rb,i} \min(1, \Sigma M_{Rc}/ \Sigma M_{Rb}) \quad \text{...}(3.7)$$

where

γ_{Rd} is the factor accounting for possible over strength due to steel strain hardening, which in the case of DCM beams may be taken as being equal to 1.0;

$M_{Rb,i}$ is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action.

ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively. The value of ΣM_{Rc} should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

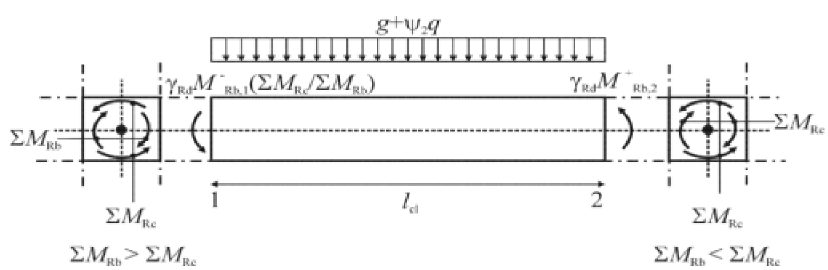

Figure 3. 5 Capacity design values of shear forces on beams [30]

2. Columns

Based on the column's equilibrium under end moments Mi,d (where i=1,2 indicates the column's end sections), the capacity design rule specifies that the design values of shear forces in primary seismic columns must be established and correspond to the creation of plastic hinges in both all directions of seismic loading, positive and negative. If the plastic hinges develop in the columns initially, it is advised that they be placed at the ends of the beams that are connected to the joints where the column end frames are inserted. With this expression, end moments Mi,d can be found:

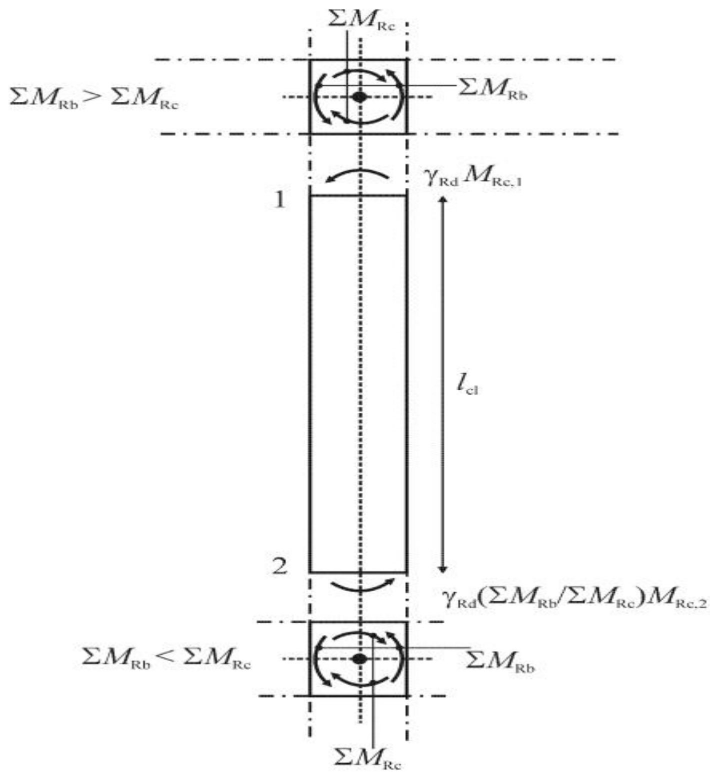

Figure 3. 6 Capacity design of shear force in column

$$M_{i,d} = \gamma_{Rd} M_{Rb},imin(1, \Sigma M_{Rc}/ \Sigma M_{Rb}) \text{ ……………………………..(3.8)}$$

Where

γ_{Rd} is the factor accounting for over strength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1.1;

M_{Rci} is the design value of the column moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;ΣMRc and ΣMRb are as defined in above.

3. Wall

It is permissible to redistribute seismic action effects between primary seismic walls by up to thirty percent, as long as the overall resistance demand remains unchanged. on order to ensure that the ratio of bending moments to shear forces is not significantly impacted on any particular wall, shear forces and bending moments should be redistributed together. Moments and shears in walls that experience significant axial force variations, such as linked walls, should be reallocated from the wall(s) experiencing low compression or net tension to the wall(s) experiencing high axial compression.

Redistribution of seismic action effects between coupling beams of various storeys in coupled walls is permitted up to a 20% increase, as long as it does not impact the seismic axial force at the base of each individual wall, which is the product of the shear forces in the coupling beams. An envelope of the analysis's bending moment diagram, vertically displaced (tension shift), should provide the design bending moment diagram along the wall's height. If the structure shows no appreciable discontinuities in mass, stiffness, or resistance along its height, the envelope can be taken to be linear.

3.9 Pushover Analysis of Buildings

One kind of non-linear static analysis is pushover analysis, which is carried out with constant gravity loads and monotonically increasing horizontal loads. Confirming the structural performance of both newly planned and existing buildings can be done for the following purposes:

- To evaluate the structural performance of newly constructed or rebuilt buildings.
- To calculate the expected plastic mechanisms and damage distribution.
- To verify or modify the values of the over strength ratio, $\alpha u/\alpha 1$.

Pushover analysis is a method that involves applying a specific lateral force (such as an inverted triangle or uniform load) to a computer model of the building. As the lateral load intensity is progressively increased, the sequence of cracks, yielding, plastic hinge development, and failure of various structural components is recorded.

Pushover analysis is a valuable tool for identifying a structure's weak points in terms of seismic performance. Usually, a number of iterations are needed, with each iteration rectifying any structural flaws found and being followed by another. The design and analysis process is iterated until the design meets predetermined performance requirements. Pushover analysis performance criteria are typically defined as the building's ideal condition given its spectral displacement amplitude or roof-top displacement.

In recent years, the most widely utilized technique for understanding the nonlinear behavior of building structures has been nonlinear static pushover analysis. Using this streamlined technique, a capacity curve that demonstrates the relationship between base shear and roof displacement is produced. The behavior of the building structure under increasing base shear forces is represented by this graph. The force-deformation curve's slope will alter as the lateral force resisting system's members' capacities surpass their yield limits as base shear forces increase, allowing the non-linear behavior to be expressed.

The pushover analysis regularly applies lateral forces to a model based on the initial load pattern. Each phase of the analysis involves the calculation of member forces, and in the subsequent step, the hinge characteristics are used to modify the stiffness of the members whose capacities are exceeded. When the structure becomes unstable, this process comes to a stop. Pushover analysis can be performed while controlling the force or displacement. When the load's exact size is known and the structure is expected to support it, the force control option can be helpful. When displacements are searched and the load's magnitude is uncertain, the displacement control is helpful.

The objective of non-linear static push-over analysis

Even though most buildings are designed for seismic resilience using elastic analysis, most will experience severe inelastic deformations during large earthquakes. methods for figuring out how structures will actually behave in these kinds of circumstances are required for modern performance-based design methods.

Pushover analysis uses static inelastic analysis to estimate the force and deformation demands in design earthquakes and compares these demands to available capacities at the performance levels of interest in order to assess the expected performance of structural systems. In the practice of structural earthquake engineering, nonlinear analysis is used for:

- Evaluate a building's performance in accordance with the needs of particular owners or stakeholders.
- Design new structures using systems, materials, or other elements that deviate from the standards of the most recent building code.
- evaluate and create seismic retrofit plans for already-constructed structures;

Procedure for Using ETABS Software in Static Non-linear/Pushover Analysis

Several forms of non-linear behavior can be taken into consideration in the static non-linear analysis of ETABS. This comprises geometric nonlinearity in all elements, material nonlinearity in the link elements, material nonlinearity at discrete, user-defined hinges in frame/line elements, and so forth. The program previously described was used to examine the worldwide performance of the buildings with different ductility classes. The actions that followed were as follows;

1. Modelling

The first stage in pushover analysis was to establish the structure's modeling, and this study used the building's three-dimensional models. The reinforcement derived from the element's capacity design was used to specify the material property and cross-section of the structure.

2. Define load case
3. Define pushover Load case

In this step the two load cases need to be defined, gravity nonlinear load case and the pushover load case in the two principal direction.

Gravity load case

Navigate to Define > load case > Gravity x/Gravity y > Add new case > load case (nonlinear static) > it began with a zero initial condition > load type (acceleration) (load pattern of DL+0.3LL) > load name Ux/Uy > scale factor 1 > Geometric nonlinearity parameters as P-Delta > load application (Full load) > used monitor displacement U1/U2 (kept as equal to 4% of the building height) > result saved > Final states only scenarios.

Push X / Y

Navigate to Define > load case > Push X / Push Y > Add new case > load case (nonlinear static) > continuous from state at end of nonlinear case > load type (acceleration) Ux/Uy > scale factor 1 > Geometric nonlinearity parameters as P Delta > load application (displacement control) > used monitor displacement U1/U2 (kept as equal to 4% of the building height) > Result saved > several states.

4. Frame hinge assignment

In the concentrated plasticity method, the impact of material yielding is "lumped" into a dimensionless plastic hinge. Elastic behavior is assumed in areas of the frame parts other than the plastic hinges, and in cases where the cross-section forces are smaller than the cross-section plastic capacity, elastic behavior is presumed. A plastic hinge that complies with the no hardening plasticity flow principles is created when the steady-forces reach the yield surface. The incremental elasto-plastic stiffness matrix can be constructed in accordance with established procedures of the non hardening plasticity flow theory in order to develop the incremental elasto-plastic relations.

By removing the need for cross-section integration and allowing for the use of fewer elements per member, the plastic hinge technique significantly lowers the computational effort. Nevertheless, it has been demonstrated that the approach overestimates the limit load when used to reinforced concrete buildings, where the propagation of plasticity effects is more substantial.

At the bottom end of the base column and the end of the beam element, the plastic hinges were modeled. Plastic hinges are also defined at the end of structural column elements for additional performance evaluation. When the plastic hinges were defined, it was thought that the forms of the hinges in the building's nonlinear deformation were concentrated or lumped in at the element's critical length, or single point.

A flexural hinge represents the plastic deformation behavior of a beam and column when subjected to lateral loads. In pushover analysis, flexural hinges are typically modeled at the ends of beam-column elements by defining and assigning the default hinge property type which is provided on the Etabs software.

Assign the M3 degree of freedom hinge property for beam and P-M2-M3 degree of freedom hinge property for column which are positioned at the top and bottom ends of the column and at both ends of the beam, or at the relative distances of 0.05 and 0.95.

.5. Shearwall hinge assignment

The plastic hinge type is a shear hinge one defined and assigned to mid-pier model and can be used for nonlinear static analysis of shear walls.

To assign the P-M3 hinges

Select > properties > wall sections > select (to which the fiber hinge is to be assigned). Assign > shell > Wall hinge > hinge property > auto fiber P-M3 > add > apply.

6.Analysis of the Structure

Using the information and method mentioned above, the structures in this study were analyzed. The ductility class medium-designed structures were examined and several parameters were extracted for the structure's performance comparison.

Select > Analyze > Don't run Earthquake, modal load cases, and set the load case to run. Run now (only the live and dead load scenarios, push X, and push Y).

Hinges are positioned at the top and bottom ends of the column and beam, or at the relative distances of 0.05 and 0.95, and at the center of shearwall. Then assign the M3 degree of freedom hinge property for beam and P-M2-M3 degree of freedom hinge property for column(flexural type) and assign the P-M3 hinges for walls (shear type).

Typically the first pushover load case is used to apply gravity load which starts from initial or unstressed condition and then subsequent lateral pushover load cases are specified to start from the final condition of the gravity pushover. Pushover load cases can be force controlled, that is, a gravity load pushover is force controlled and lateral pushovers are displacement controlled. As the lateral load intensity is progressively increased, the sequence of cracks, yielding, plastic hinge development, and failure of various structural components is recorded.

The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve. Also, the file menu in this display allows to print the coordinates of the capacity curve and the demand curve.

CHAPTER 4 RESULT AND DISCUSSION

4.1 General

The response of wall equivalent dual system reinforced concrete buildings with different story height and medium ductility classes has been evaluated. The seismic performance of buildings (G+5, G+15, and G+30) structures has been evaluated using static non-linear (pushover analysis) using the parameters Top story displacement, Story drift, Capacity curve and Plastic hinge formation.

4.2 Modelling

The building under discussion is modeled in ETABS2021.0.0 using nonlinear static (pushover) analysis. The use of nonlinear static pushover analysis for determining the nonlinear behavior of structures has grown in popularity. This process results in a capacity curve that illustrates the relationship between roof displacement and base shear. Pushover analysis is a computationally challenging procedure and may cause numerical difficulties for the study's instruments if the building of choice is complex. In an attempt to complete the analysis and reduce the execution time, simpler structures are therefore selected. The places where nonlinear behavior is expected to occur are called hinge locations.

4.3 Analysis of the Building

By predicting the strength and deformation demands for design earthquakes using static inelastic analysis and comparing these demands with existing capacities at the performance level of interest, pushover analysis may be used to evaluate the expected performance of a structural system. It provides an example of an analysis process that includes calculating the static equilibrium equations repeatedly and step-by-step in order to determine how a structure would react to a lateral pattern that increased monotonically.

Up to convergence, the stiffness matrix is updated and the structural resistance is assessed with each increment of the forcing function Until; a predetermined performance limit state is reached, structural collapse is noticed, or the programme fails to converge, the solution is carried out. This indicates that each point in the displacement vs. base shear capacity curve that emerges represents the effective and equilibrated stress states of the structure. that is, a state of deformation that directly correlates with the vector of the external force.

In order to obtain the greatest action effects on the structure for the design of structural elements, an analysis of the buildings presented in this thesis was conducted by specifying the necessary input material qualities, frame sections, loads, load patterns, and load combinations. ETABS 2021.0.0 computer software was used to analyze wall equivalent dual system reinforced concrete construction for medium ductility classes.

The analysis of slab and load transferring from slab to beam is done as per ES EN 1991-1-1, 2015.

4.4 Design of the Building

The capacity design philosophy was applied in the design of all the Regular and Irregular reinforced concrete elements. The following are the fundamental steps in building design;

1. Calculating Design Loads.
2. Analysis of the structure.
3. Design of Beam Elements.
4. Calculating the Beam's Flexural Capacities.
5. Calculating Columns' Overstrength Flexural Capacity and Column Design.
6. Shear Design of Column and Beam Elements.
7. Examine carefully the Detailing Difference Between Elastic Regions and Critical Regions..

4.5 Steps In Pushover Analysis

1. Using the ETABS graphical interface, create the basic computer model (without the pushover data) as per normal.
2. By taking end-offsets from supports into consideration, choose one or more frame members and assign them one or more hinge properties. The process for shearwall is the same.

3. Define the pushover load situation based on the types of loads applied to the modelled structure. These loads could be gravitational loads and/or lateral loads.
4. Perform a static nonlinear pushover analysis after completing the basic static analysis. A displacement controlled analysis is performed after applying a gravity load which starts from initial or unstressed condition
5. Show the pushover curve.
6. Show the capacity spectrum curve.
7. The point where the capacity curve and the single demand spectrum curve connect defines the performance point value of the analyzed structure.
8. Examine the pushover displaced shape and hinge formation sequence on structural elements step-by-step.
9. Additionally, examine member forces separately.
10. Print the pushover analysis results in tabular form for the entire modal or for a specific building structural element. During the pushover analysis, these outputs include joint displacement at each step, frame member force at each step, plus and hinge force, displacement, and state.

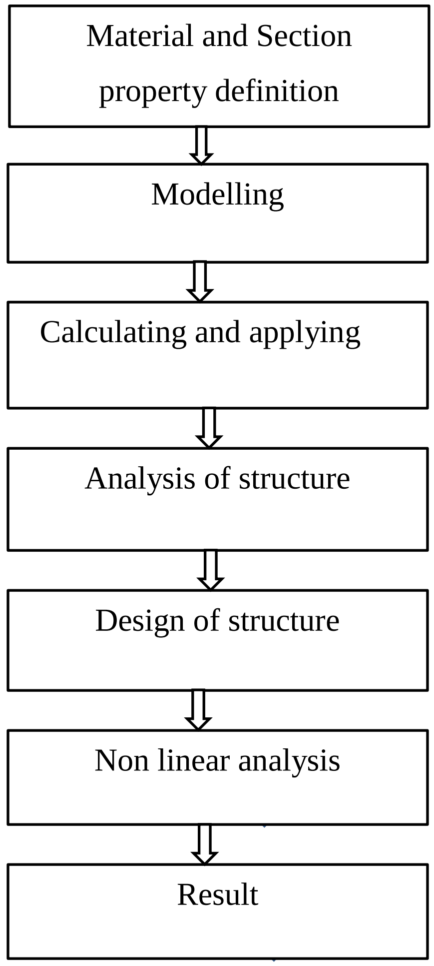

Figure 4. 1 Total Analysis steps

4.6 Sesmic Performance Evaluation of Wall Equivalent Frame structures

Static nonlinear pushover analysis was used to evaluate the seismic performance of the reinforced concrete structure that was designed using the capacity design method. In the following sections, the building's performance for the medium ductility class was examined in terms of structural performance parameters, including plastic hinge distribution in both Principal X and Y directions, capacity curve, inter-story drift, and story displacement.

4.6.1 Comparison of Story Top Displacement

The first parameter used to evaluate the performance of the structure under consideration was top story displacement. Top story displacement for G+5, G+15 and G+30 (plan regular) with shear wall location variation (at the center and at the edge), presence and absence of opening for building with shear wall location at the edge and for building which are both in plan and in elevation irregular are included in the comparison. The following figure displays the results of top story displacement: 4.2 - 4.7, in that order.

The top story displacement value increases for shear wall location at the center relative to the edge and also the graph display's increment as the story level increases. Additionally, the displacement value in the y direction is quite high in comparison to the x direction. These results remain true for the other two factors, which are the shear wall at the edge with the opening and the irregular building in both plan and elevation.

The highest top displacement result is observed for building with both in plan and elevation irregular with increasing in story level.

1.Shearwall location (SWC Vs SWE)

1.Push-X

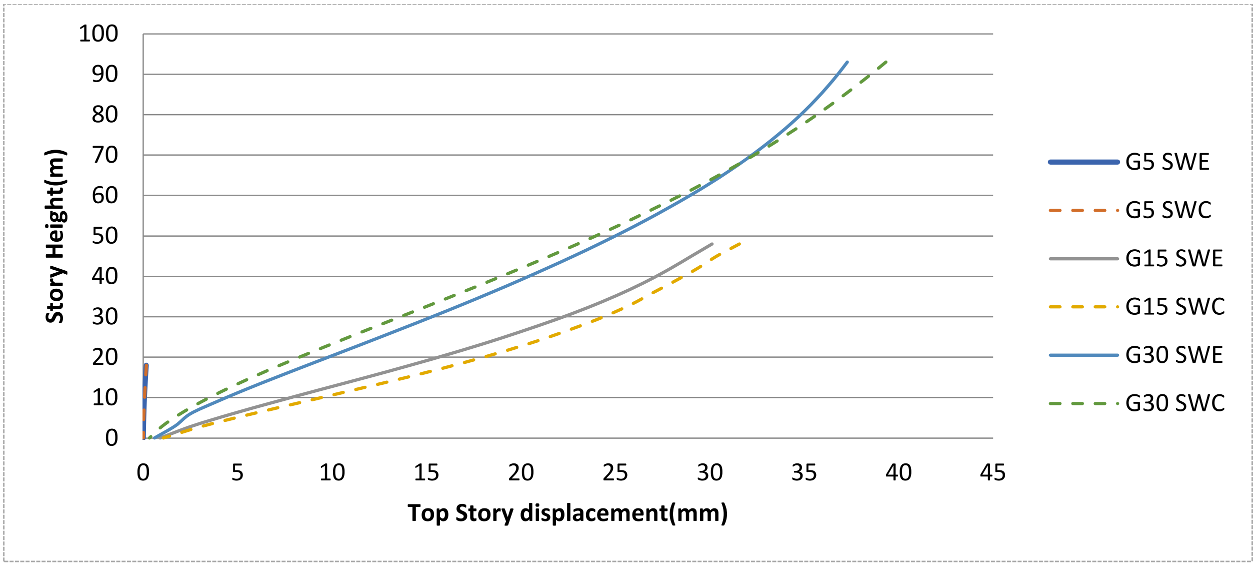

Figure 4. 2 Story Top Displacement for Shear wall location in X Direction

2.Push-Y

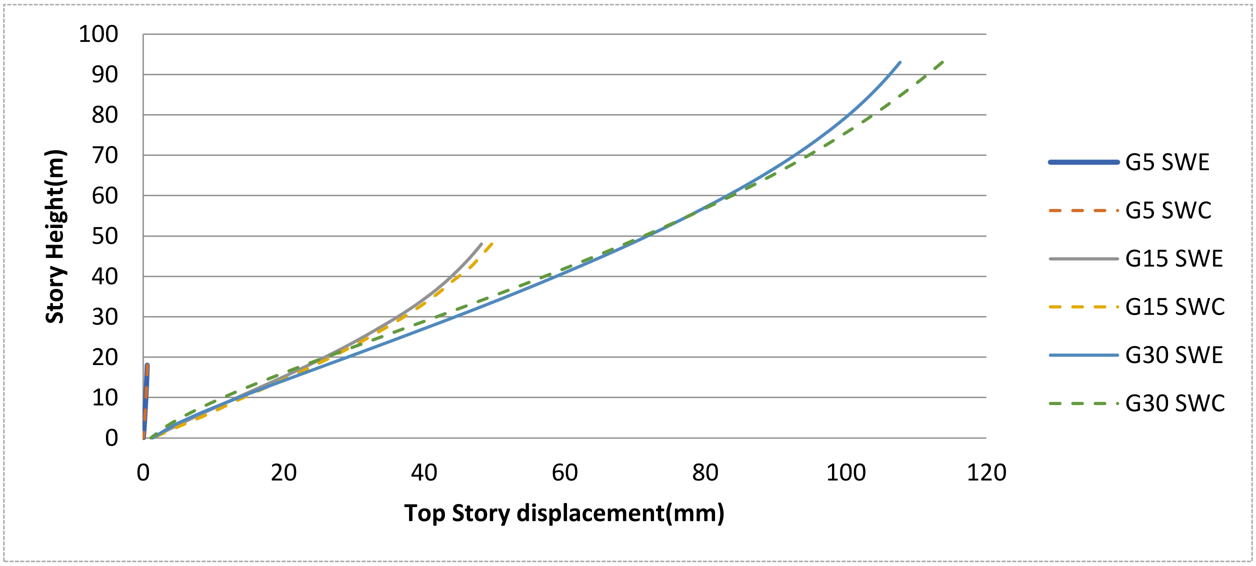

Figure 4. 3 Story Top Displacement for Shear wall location in Y Direction

2.Opening presence (SWEOS Vs SWECS)

1.Push-X

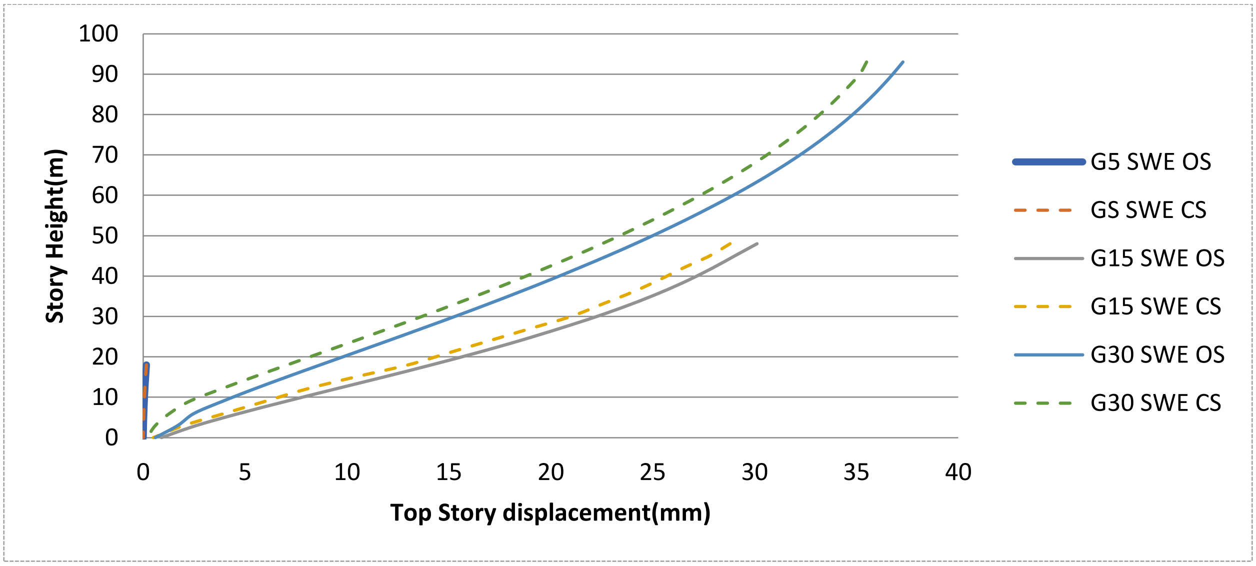

Figure 4. 4 Story Top Displacement for Introducing opening in X Direction

2.Push-Y

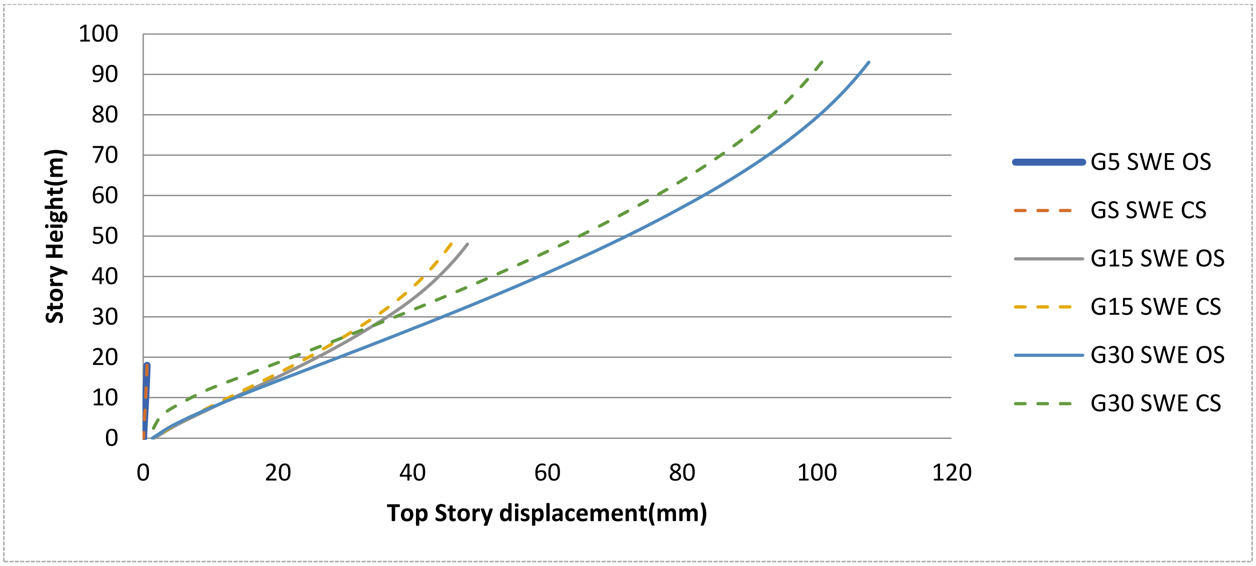

Figure 4. 5 Story Top Displacement for Introducing opening in Y Direction

3.Plan regular V_S Plan irregular (SWCPR Vs SWCPI)

1.Push-X

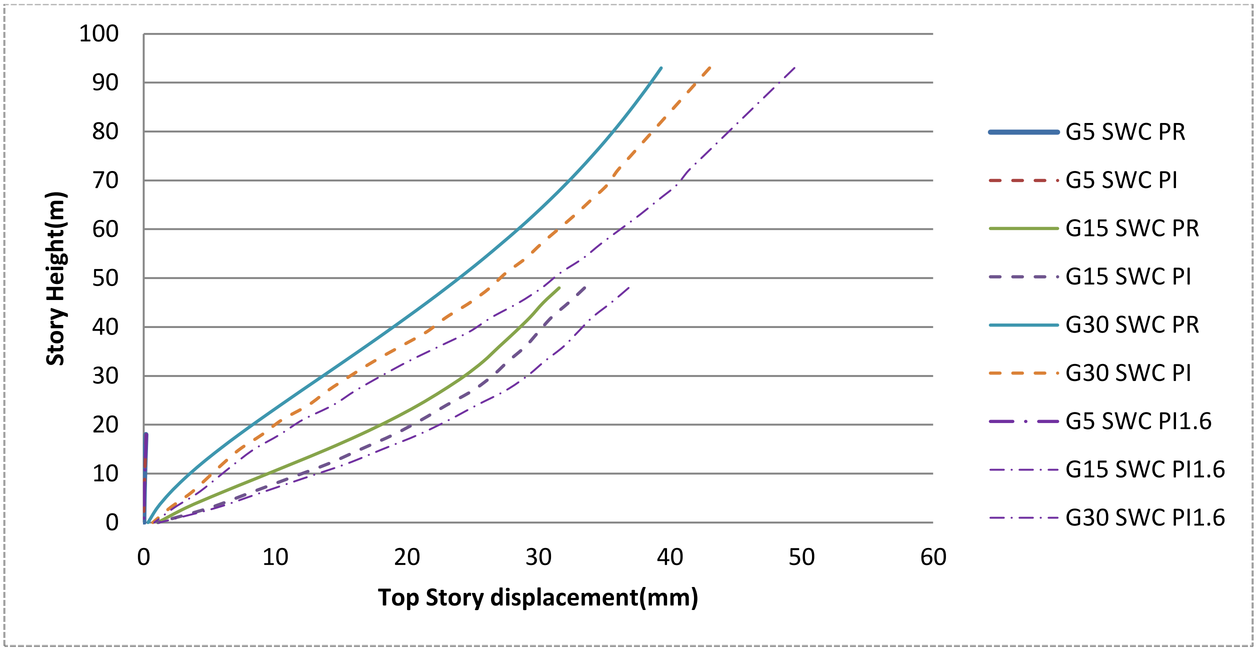

Figure 4. 6 Story Top Displacement for Introducing Irregularity in X Direction

2.Push-Y

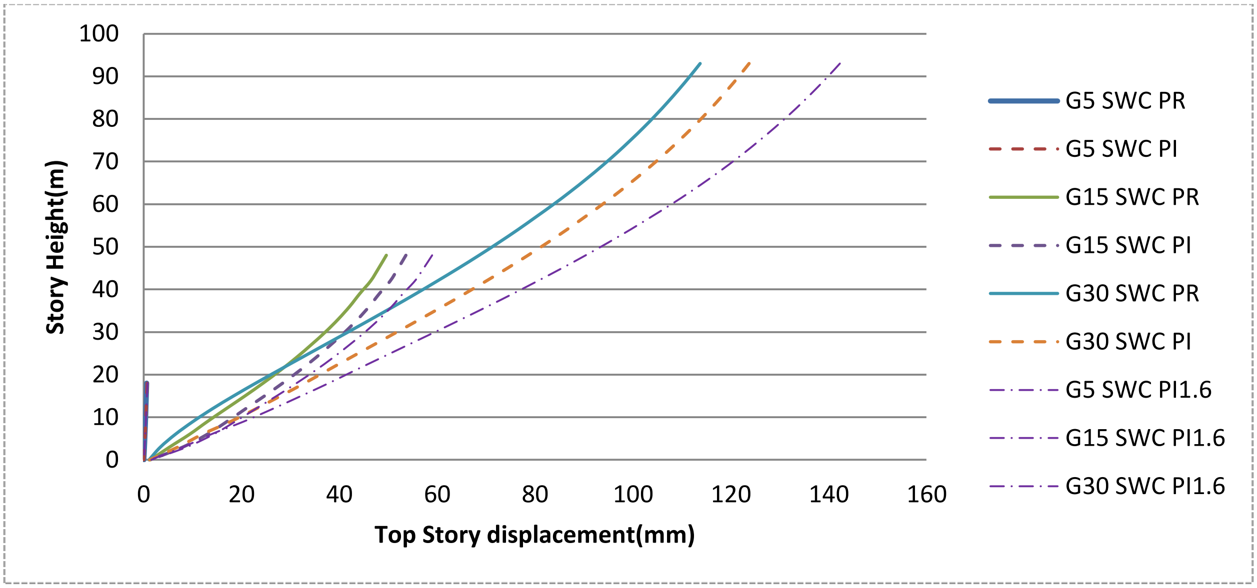

Figure 4. 7 Story Top Displacement for Introducing Irregularity in Y Direction

4.6.2 Comparison of Inter-story Drift

The second parameter used to describe the building's damage level is interstory drift, which is calculated by dividing the displacement between two successive floors by the story's height. G+5, G+15 and G+30 with (plan regular) shear wall location variation (at the center and at the edge), presence and absence of opening for building with shear wall location at the edge and for building which are both in plan and in elevation irregular within plan regular and in elevation irregular are included in the comparison.

The result of inter-story drift value shown in figure below 4.8 - 4.13 respectively, the graph shows nonlinear relationship of floor height and inter story drift value. The interstory drift value is higher on the y direction for all the studied parameters under consideration. Changing shearwall location have relatively showed an increased value of 3% ,7% and 9% for low, medium and high rise building respectively and also the result shows increment as irregularity introduced to the building.

1.Shearwall location (SWC Vs SWE)

1.Push-X

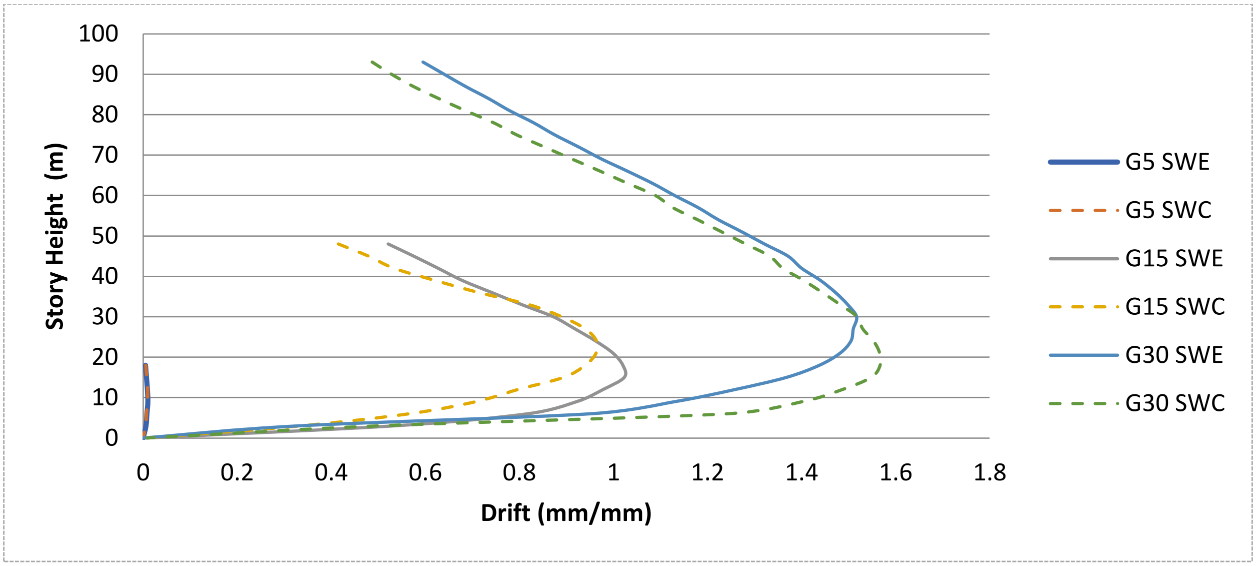

Figure 4. 8 Inter Story Drift for Shear wall location in X Direction

2.Push-Y

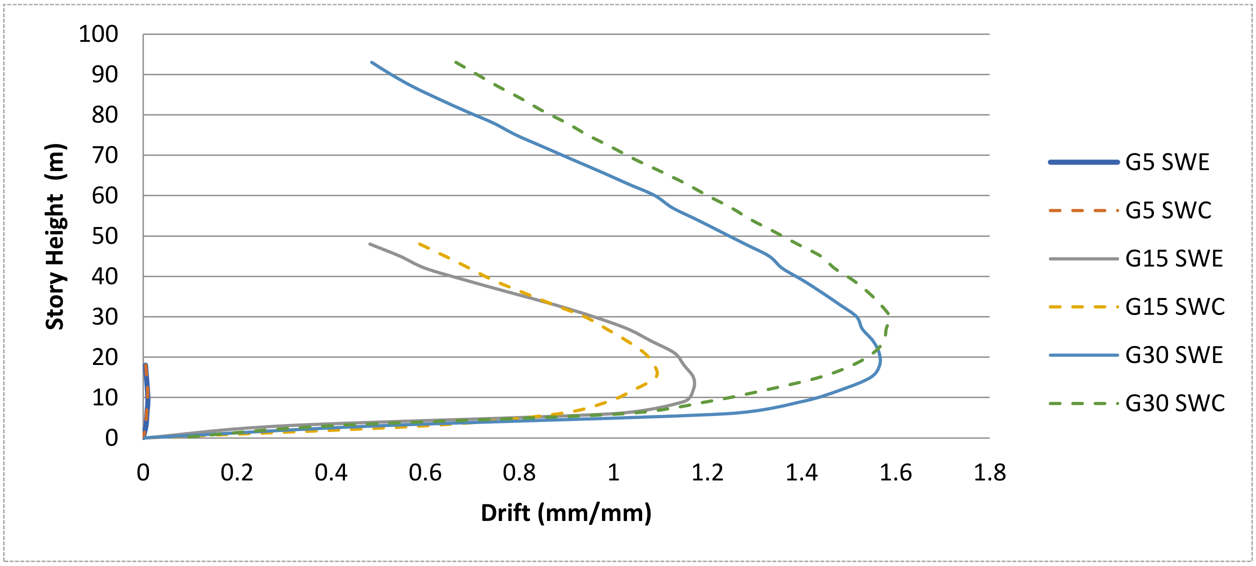

Figure 4. 9 Inter Story Drift for Shear wall location in Y Direction

2.Opening presence (SWEOS Vs SWECS)

1.Push-X

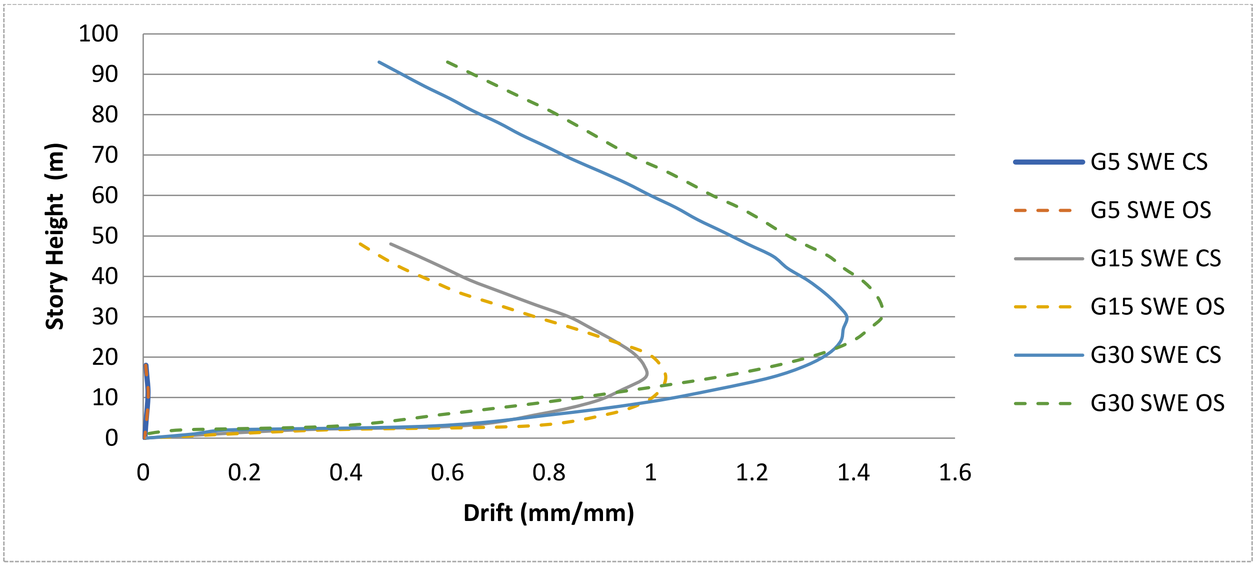

Figure 4. 10 Inter Story Drift for Introducing opening in X Direction

2.Push-Y

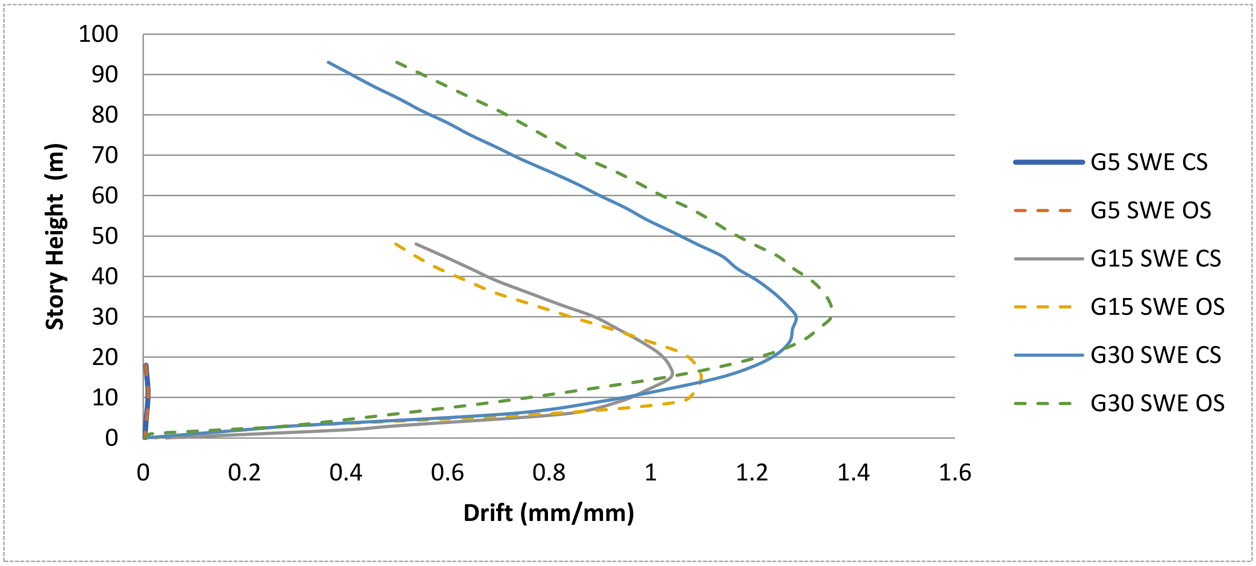

Figure 4. 11 Inter Story Drift for Introducing opening in Y Direction

3. Plan regular V_S Plan irregular (SWCPR Vs SWCPI)

1.Push-X

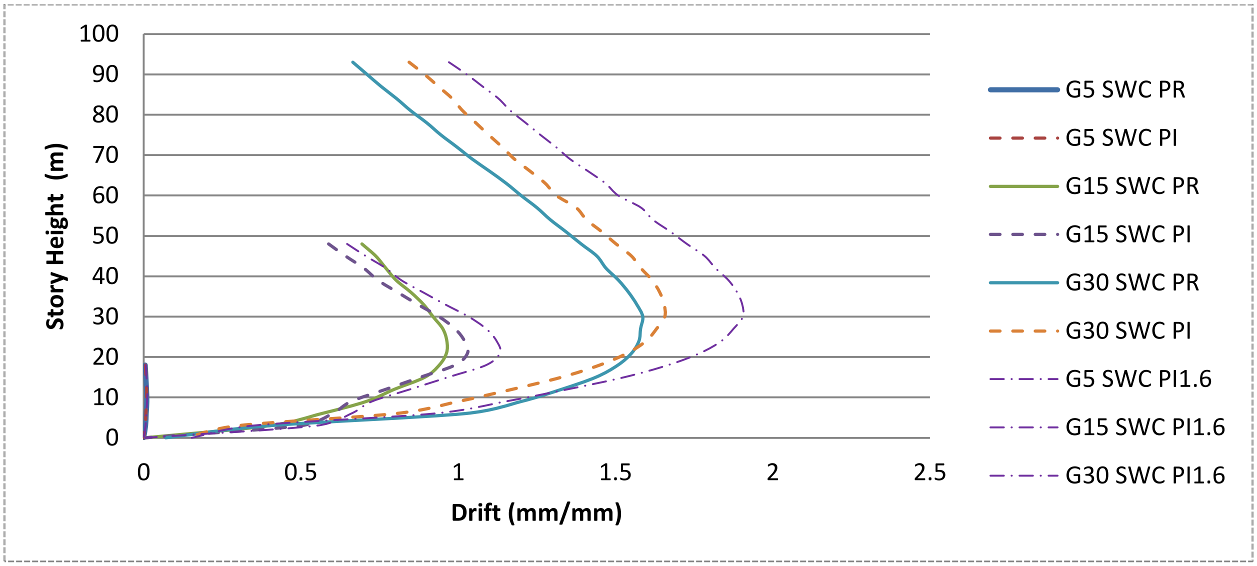

Figure 4. 12 Inter Story Drift for Introducing Irregularity in X Direction

2.Push-Y

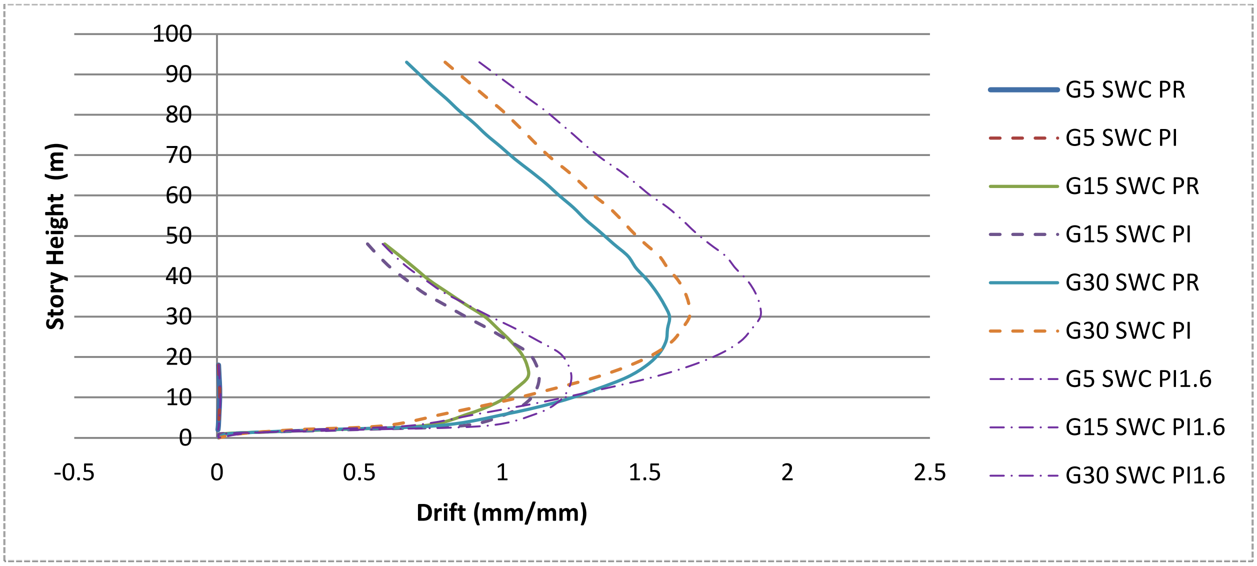

Figure 4. 13 Inter Story Drift for Introducing Irregularity in Y Direction

4.6.3 Comparison of Capacity Curve

The third parameter to describe the performance of the structure was capacity curve, where the result shown in the graph below 4.20 - 4.23 Respectively. The capacity curve illustrates how the building's capacity has progress as a result of the imposed load. The graph shows base shear verses the displacement value, where both base shear and displacement value is high in Y direction relative to the x direction.

The graph pattern tends to be linear straight line up to elastic limit but after elastic limit the graph of building with shearwall at the edge deform more than buildings with shearwall location at the center. The same is true for buildings with in elevation only irregular.

1.Shearwall location (SWC Vs SWE)

1.Push-X

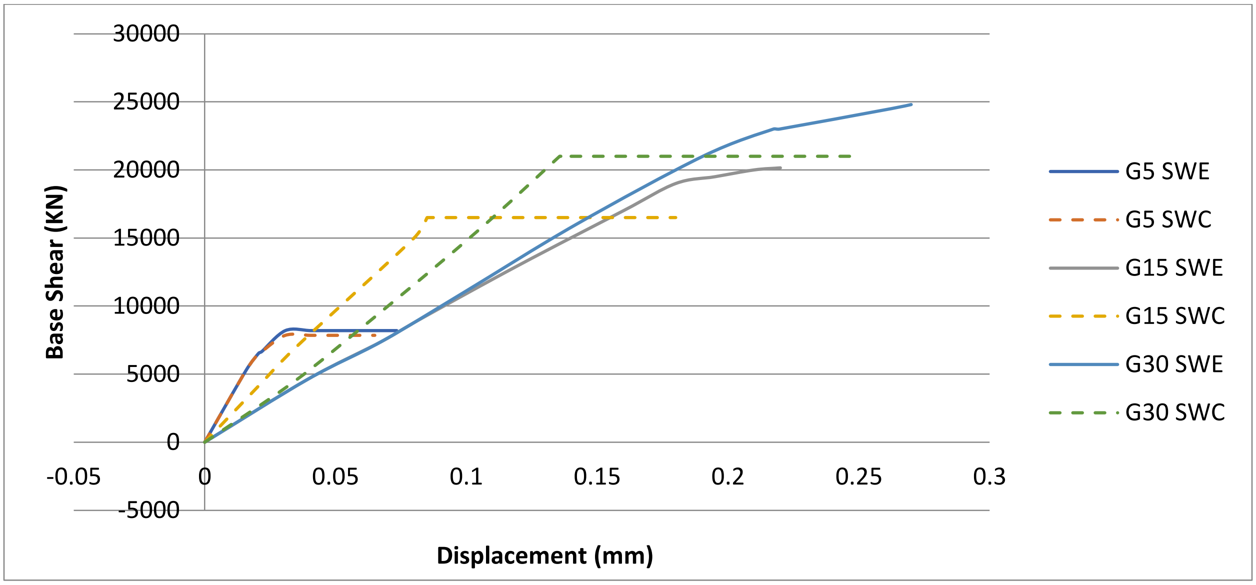

Figure 4. 14 Capacity Curve for Shear wall location in X Direction

2.Push-Y

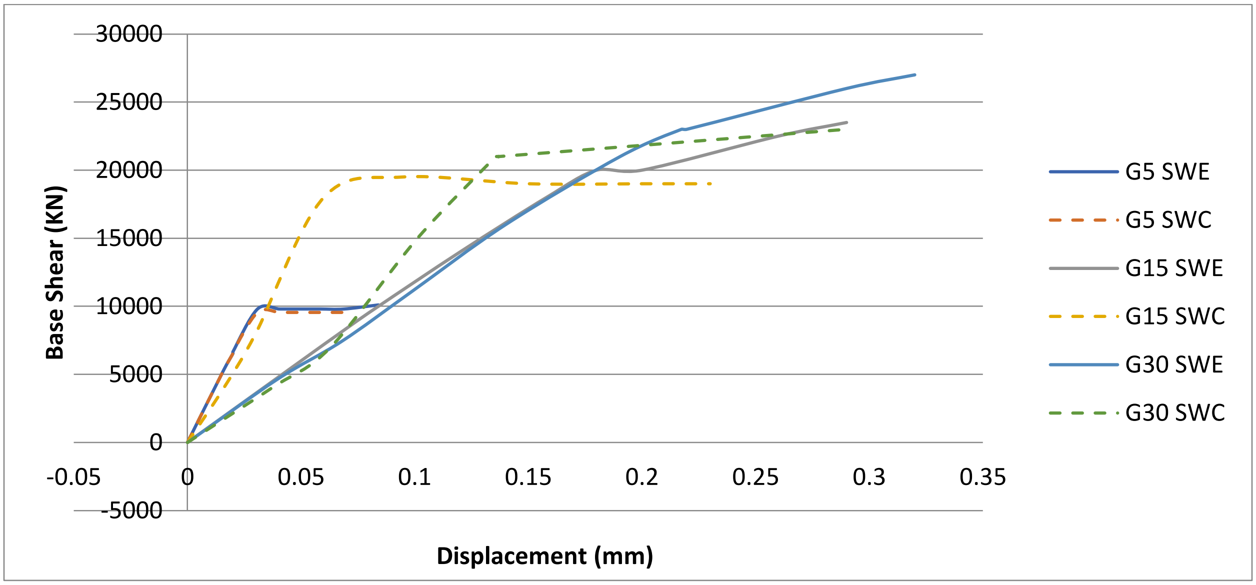

Figure 4. 15 Capacity Curve for Shear wall location in Y Direction

2. Plan regular V_S Plan irregular (SWCPR Vs SWCPI)

1.Push-X

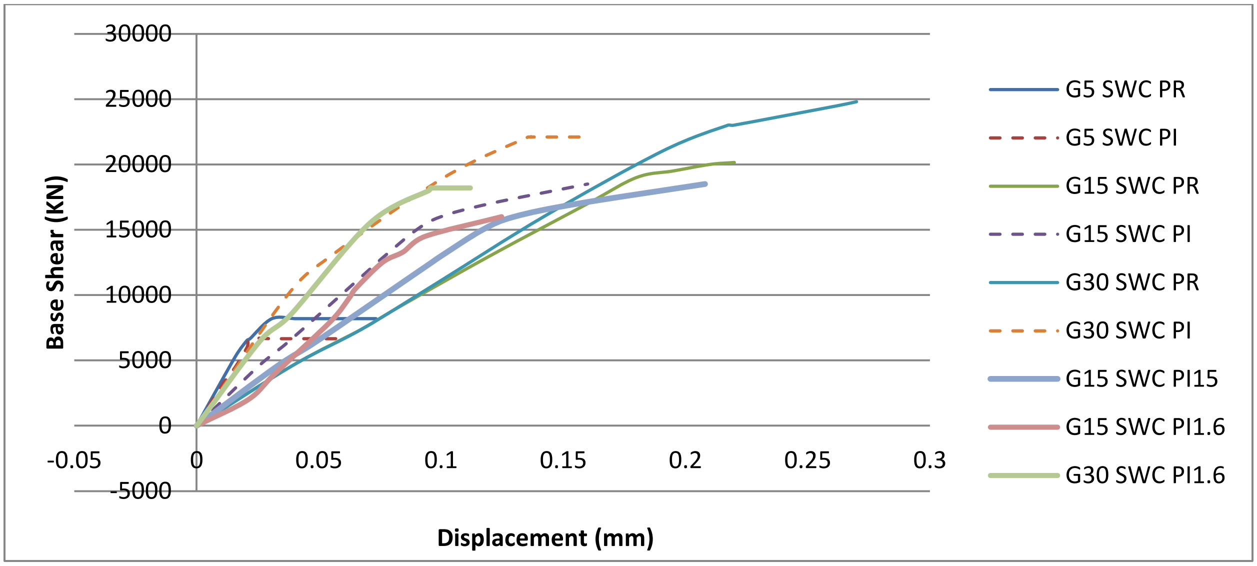

Figure 4. 16 Capacity Curve for Introducing Irregularity in X Direction

2.Push-Y

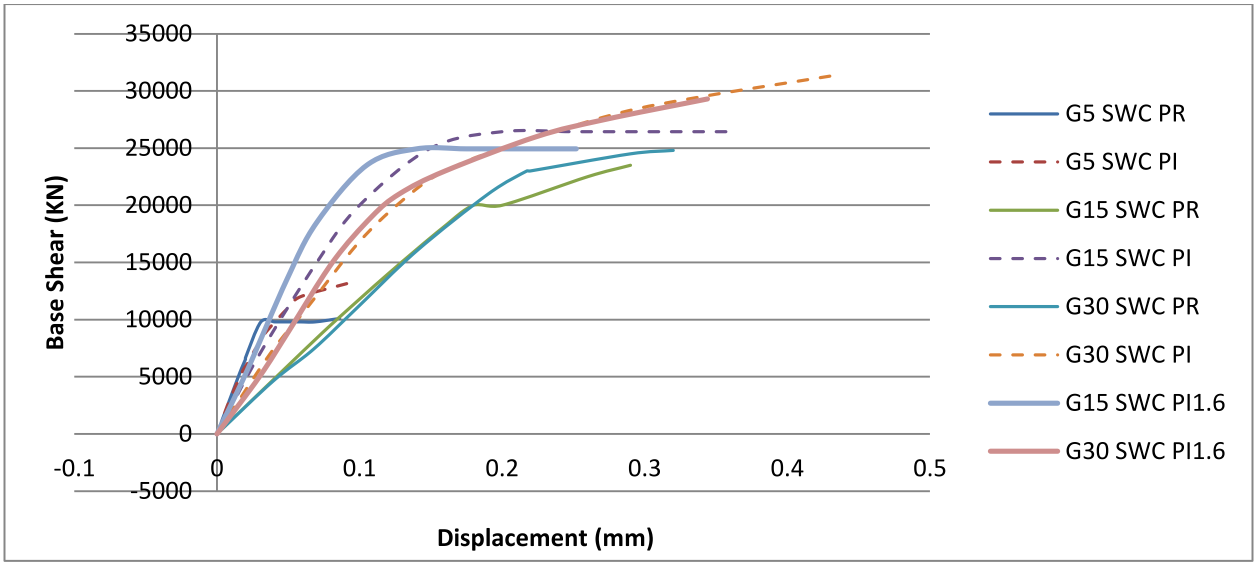

Figure 4. 17 Capacity Curve for Introducing Irregularity in Y Direction

4.6.4 Comparison of Plastic Hinge distribution

The plastic hinges are located and defined at the ends of each beam and the top and bottom of each story's columns.

When a horizontal earthquake force is applied, irregular buildings are susceptible to two actions at once, even if push over analysis is only done in the direction of larger displacement. Generally, all the structures with different shearwall location, effect of opening and also irregular ones considered under evaluation are subjected to the defined load combination have remained with in the immediate occupancy performance level.

The performance level of irregular buildings exhibits some life safety and a small number of collapse points but no damage points at all, in contrast to the performance level of plan regular buildings, which displays a large number of life safety. The results are presented in the following table 4.2 - 4.5 respectively below.

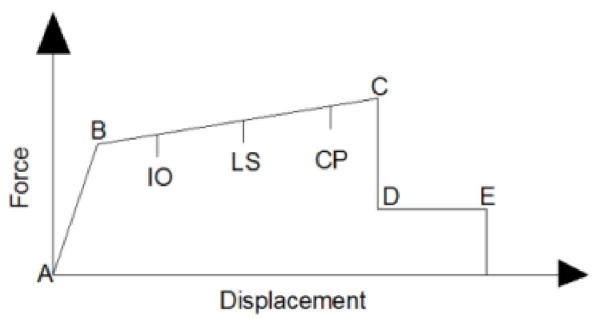

Figure 4. 18 Force-Displacement Relation of Performance levels [28]

1.SHEARWALL LOCATION (SWC Vs SWE)

1.Push-X

Table 4. 1 Number of Plastic hinge formation at different performance level with different shearwall location in X Direction

a) G+5 SWE

Step	A-B	B-C	C-D	D-E	>E
0	1084	1548	0	0	0
1	1082	1550	0	0	0
2	1062	1570	0	0	0
3	1020	1612	0	0	0
4	1016	1616	0	0	0
5	1002	1630	0	0	0
6	988	1644	0	0	0
7	970	1662	0	0	0
8	934	1698	0	0	0
9	914	1718	0	0	0
10	903	1729	0	0	0

b) G+5 SWC

Step	A-B	B-C	C-D	D-E	>E
0	1050	1582	0	0	0
1	1038	1594	0	0	0
2	1008	1624	0	0	0
3	956	1676	0	0	0
4	942	1690	0	0	0
5	928	1704	0	0	0
6	914	1718	0	0	0
7	896	1736	0	0	0

8	860	1772	0	0	0
9	840	1792	0	0	0
10	829	1803	0	0	0

a) G+15 SWE

Step	A-B	B-C	C-D	D-E	>E
0	6392	0	0	0	0
1	6392	0	0	0	0
2	6384	8	0	0	0
3	6356	36	0	0	0
4	6356	36	0	0	0
5	6356	36	0	0	0
6	6352	40	0	0	0
7	6352	40	0	0	0
8	6352	40	0	0	0
9	6352	40	0	0	0
10	6352	40	0	0	0
11	6352	40	0	0	0

b) G+15 SWC

Step	A-B	B-C	C-D	D-E	>E
0	6324	0	0	0	0
1	6324	0	0	0	0
2	6320	4	0	0	0
3	5976	348	0	0	0
4	4880	1444	0	0	0
5	4475	1849	0	0	0
6	4373	1951	0	0	0
7	4317	2007	0	0	0

8	4237	2087	0	0	0
9	4197	2127	0	0	0
10	4195	2129	0	0	0

a) G+30 SWE

Step	A-B	B-C	C-D	D-E	>E
0	12032	0	0	0	0
1	12032	0	0	0	0
2	12032	0	0	0	0
3	12028	4	0	0	0
4	11476	556	0	0	0
5	11466	566	0	0	0
6	11446	586	0	0	0
7	11446	586	0	0	0
8	11446	586	0	0	0
9	11446	586	0	0	0

b) G+30 SWC

Step	A-B	B-C	C-D	D-E	>E
0	11904	0	0	0	0
1	11904	0	0	0	0
2	11904	0	0	0	0
3	11796	108	0	0	0
4	10887	1017	0	0	0
5	9961	1943	0	0	0
6	9529	2375	0	0	0
7	9503	2401	0	0	0

8	9127	2777	0	0	0
9	8979	2921	0	4	0

2.Push-Y

Table 4. 2 Number of Plastic hinge formation at different performance level with different shearwall location in Y Direction

a) G+5 SWE

Step	A-B	B-C	C-D	D-E	>E
0	1084	1548	0	0	0
1	1082	1550	0	0	0
2	1020	1612	0	0	0
3	1018	1614	0	0	0
4	1018	1614	0	0	0
5	1016	1616	0	0	0
6	1012	1620	0	0	0

b) G+5 SWC

Step	A-B	B-C	C-D	D-E	>E
0	1050	1582	0	0	0
1	1062	1570	0	0	0
2	1010	1622	0	0	0
3	1009	1623	0	0	0
4	1009	1623	0	0	0
5	1007	1625	0	0	0
6	1005	1627	0	0	0
7	986	1646	0	0	3

a) G+15 SWE

Step	A-B	B-C	C-D	D-E	>E
0	6392	0	0	0	0
1	6386	6	0	0	0
2	6380	12	0	0	0
3	6378	14	0	0	0
4	6372	20	0	0	0
5	6370	22	0	0	0
6	6362	30	0	0	0
7	6362	30	0	0	0
8	6354	38	0	0	0
9	6354	38	0	0	0
10	6354	38	0	0	0
11	6346	46	0	0	0

b) G+15 SWC

Step	A-B	B-C	C-D	D-E	>E
0	6324	0	0	0	0
1	6316	8	0	0	0
2	6294	30	0	0	0
3	6294	30	0	0	0
4	6280	44	0	0	0
5	6268	56	0	0	0
6	6252	72	0	0	0
7	6226	98	0	0	0
8	6202	122	0	0	0
9	6186	138	0	0	0
10	6184	140	0	0	0

a) G+30 SWE

Step	A-B	B-C	C-D	D-E	>E
0	12032	0	0	0	0
1	12026	6	0	0	0
2	12016	16	0	0	0
3	12014	18	0	0	0
4	11994	38	0	0	0
5	11914	118	0	0	0
6	11904	128	0	0	0

b) G+30 SWC

Step	A-B	B-C	C-D	D-E	>E
0	11904	0	0	0	0
1	11894	10	0	0	0
2	11870	34	0	0	0
3	11844	60	0	0	0
4	11774	130	0	0	0
5	10674	1230	0	0	0
6	10636	1268	0	0	0
7	10576	1328	0	0	0

4.6.5 Comparison of Target Displacement

The performance point shown in Table below 4.6 and 4.7 respectively.

- Having shearwall location at the edge shows an increased value on performance point than at the center and the same is true for absence of opening.

- The regular building show high performance point in the X direction while the irregular one in the Y direction.

Table 4. 3 Summary of performance point for Regular building structures

Direction	story level	Shearwall	Performance Points	
		location	Base Shear (KN)	Displacement(mm)
X	5	SWC	4669.0559	0.0123
Y			6956.89	0.0141
X		SWE	4759.231	0.013988
Y			6969.2389	0.0145
X		SWE CS	4765.184	0.01301
Y			6975.21	0.014002
X	15	SWC	6794.123	0.026443
Y			7009.56	0.031
X		SWE	6806.759	0.029301
Y			7102.373	0.0312
X		SWE CS	6812.542	0.029495
Y			7187.23	0.0317
X	30	SWC	6891.9839	0.061587
Y			7123.423	0.0693
X		SWE	6962.935	0.064687
Y			7205.9342	0.0689
X		SWE CS	7056.1908	0.065579
Y			7330.4903	0.061234

Table 4. 4 Summary of performance point for Irregular building structures

Direction	story height	Shearwall location	Performance Points	
			Base Shear (KN)	Displacement(mm)
X	5	SWC	4563.123	0.0112

Y		SWC	6975.3214	0.014785
X	15	SWC	5953.258	0.0212
Y		SWC	7115.001	0.03012
X	30	SWC	6799.235	0.0594
Y		SWC	7229.987	0.0701

CHAPTER 5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In the previous section Top story displacement, Inter story Drift, Capacity curve and Plastic hinge formation has been discussed for all the structures under consideration. The result has been displayed on the graphs, figure and tables. Based on the analysis performed and results for the buildings considered under this thesis paper the following conclusions are drawn;

- The Top displacement value shows increment in Y Direction for the parameters; placing shearwall at the center by 5% relative to the edge, effect of having opening increase the Top displacement by 2% and having a plan irregular building increase the result by about 10%. The value of this parameter increases as the story level increase and have relatively more impact on High rise building under consideration.
- The Inter story drift of the buildings considered have shown an increased value on the Y Direction relative to the X Direction. The value shows increment as the story level increased.
- Introducing Irregularity in the building decrease the performance level by 0.8%, 5%, 8% for low, medium and high rise building respectively and having shear wall at the center by 0.4%,4% and 6%, the least effect is shown for introducing opening which is 0.1%, 0.9% and **2**% for the buildings under consideration.
- The plastic hinges development in all of the sample structures begins with the base columns and beam ends of the lower levels. Compared to regular buildings, the performance level for irregular buildings has decreased.
- Pushover analysis is useful for forecasting the response of the structures as well as progressive hinge creation. In general, the progressive hinge development is helpful for purposes of strengthening.
- Based on the results we can conclude that placing shearwall at the edge, decreasing the opening area and having a regular building can give a better performance of a building.

5.2 Recommendation

- Further evaluation is necessary to fully understand the impact of shear wall arrangement and shape on seismic performance of the structure.
- The structures considered in this study are new buildings; therefore, future study can cover existing buildings to check its performance and take measures.
- Further evaluation needs to be done for buildings with different structural system and also irregularity type.

REFERENCES

[1] Eurocode 4, "Eurocode 4: Design of composite steel and concrete structures Part 1-1: General rules and rules for buildings," CEN national Members, Brussels, 2004.

[2] Sengupta, A. K., &Asokan, A. (2008). Modeling of masonry infill walls for nonlinear static analysis of buildings. Indian concrete journal, 82(2).

[3] Dutta, S. C., Das, P. K., & Sengupta, P. (2017). Seismic behavior of irregular structures. Structural Engineering International, 27(4), 526-545.

[4] Shah Dhara and Shrivastava Anjuri, (2018). Critical Analysis of Building with Vertical Irregularities as per IS 1893 part-1,2002", International Journal of Civil, Structural, Environment and Infrastructure Engineering Research and Development, 8(2), 11- 22.

[5] All-Ali AAK, Krawinkler H (1998). "Effect of Vertical Irregularities on Seismic Behavior of Building Structure", Blume earthquake engineering center, Stanford University, Report No. 130.

[6] Das, S. and Nau, J.M. (2003). "Seismic Design Aspects of Vertically Irregular Reinforced Concrete Buildings", Earthquake Spectra, Vol. 19, No. 3, pp. 455-477.

[7] Satrajit das. (August 2000) Seismic "response of Vertically Irregular Reinforced Concrete Structures" .

[8] Altuntop, Mehmet Alper, (October 2007) Analysis of Building Structures with Soft Stories, AtilimUniversity,.

[9] S.Varadharajan. (2014) Effect of Vertical Irregularity in RC Framed Buildings in Severe Seismic Zone.

[10] Design, B. (2003). Construction. White Paper on Sustainability, 6-7.

[11] Code, P. (2005). Eurocode 8: Design of structures for earthquake resistance-part 1: general rules, seismic actions and rules for buildings. Brussels: European Committee for Standardization

[12] Moehle, J. P., Hooper, J. D., Kelly, D. J., & Meyer, T. R. (2010). Seismic Design of cast-in-place concrete diaphragms, chords, and collectors. Seismic design technical brief, US Department of Commerce, Building and Fire Research Laboratory, National Institute of Standards and Technology

[13] Taranath, B. S. (2004). Wind and earthquake resistant buildings: Structural analysis and design.

[14] Elnashai, A. S., & Di Sarno, L. (2008). Fundamentals of earthquake engineering. New York: Wiley

[15] Mohamed Arega, (December 2006) “How high can buildings made of ribbed or flat slab construction without shear walls be built”, Thesis in partial fulfilment of degree for master of science, Addis Ababa University.

[16] Marie Jose Nollet,(February 1991) “Behavior of wall frame structures”, McGill University.

[17] Mihaela Anechitel, Doina Stefan, Ioana Olteanu,(2010) “The torsional eccentricity of the structure in miscellaneous design codes”, Universitatea Technica.

[18] A.J. Zielinski, (1984) “Concrete structures under impact loading Rate effects”, The Netherlands: Delft University of technology.

[19] Dr-ing. A.J. Zielinski, (1984) “Concrete structures under impact loading rate effects”, Netherlands; Delft University of Technology, Stevin Laboratory.

[20] Abdi Mohamed, (March 2012) “Comparison of conventional and modal pushover analysis of buildings”, Thesis in partial fulfilment of degree for master of science, Addis Ababa institute

of Technology.

[21] Anil K. Chopra, (1995) “Dynamics of Structures – Theory and Application to Earthquake Engineering”, Printice-Hall.

[22] Mihaela Anechitel, Doina Stefan, Ioana Olteanu,(2010) “The torsional eccentricity of the structure in miscellaneous design codes”, Universitatea Technica,, Gheorghe Asachi”.

[23] Mehmet Inel, Hayli Baytan Ozmen. (30 march 2006.) “Effects of Plastic Hinge Properties in Non linear Analysis of Reinforced Concrete Buildings” , Department of Civil Engineering, Pamukkale University, 20070 Denizli, Turkey.

[24] Rohit Bansal. Pushover Analysis of Reinforced concrete Frame , Department of civil engineering, Thapar University , Patiala-147001, PUNJAB, Indi

[25] Rana R.,Jin L. and Zekio A.(2004) “Pushover Analysis of a 19 story Concrete Shear Wall Building” 13th World Conference on Earthquake Engineering Vancouver,B.C, paper No.133, Canada.

[26] X.-K. Zou, C.-M. Chan.(10 May 2005) Optimal Seismic Performance-Based Design of Reinforced Concrete Buildings Using Nonlinear Pushover Analysis. Department of Civil Engineering, Hong Kong University of Science and Technology, Kowloon, Hong Kong, China.

[27] Habibulah, Ashraf and Stephen, Pyle,(1998) “Practical Three-Dimensional Non linear Static Pushover Analysis”, Structure Magazine, U.S.A, PP.1-2.

[28] Chaudhari, D. J., & Dhoot, G. O. (2016). Performance Based Seismic Design of Reinforced Concrete Building, (6), 188–194.

[29] Paulay, T., & Priestley, M. J. N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. New York, United States of America (USA).

[30] Ethiopian Building Code Standard-8 [EBCS EN 1998-1]. (2014). Design of Structures for

Earthquake Resistance (Vol. part-1). Addis Ababa, Ethiopia.

[31] Ravikumar, C. M., Babu Narayan, K. S., Sujith, B. V., & Venkat Reddy, D. (2012). Effect of irregular configurations on seismic vulnerability of RC buildings. Architecture Research, 2(3), 20-26.

APPENDIX

Appendix A-Verification of software ETSBS 2021.0.0. for pushover analysis

The purpose of the verification is to show the pushover or static nonlinear analysis functionality of ETABS 2021.0.0. The experiment investigated by Vecchio and Emara (1992) is used for verification. On the selected frame, Vecchio and Emara (1992) carried out experimental testing. The frame was constructed with a 3500 mm center-to-center span, a 2000 mm story height, and a 4600 mm overall height, as shown in Figure A-1. The dimensions of each column and beam were 300 mm in width and 400 mm in depth. Every participant received the same kind of support: Shear reinforcement was provided by four No. 20 deformed bars on top, four No. 20 deformed bars below, and ten No. 10 closed stirrups placed 125 mm apart.

During the frame's testing, each column was subjected to a continuous axial force of 700 kN, and the top story beam was subjected to a monotonic lateral load until the frame's maximum capacity was reached. The frame was loaded to a lateral displacement of 155 mm in the experimental setup, and it was subsequently unloaded to a net zero lateral load. [20].

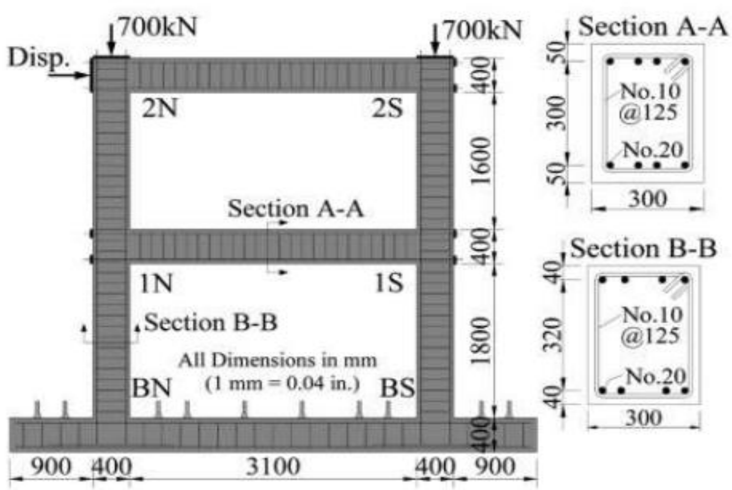

Figure A. 1 structural detail of Vecchio and Emara (1992) frame

Table A. 1 Vecchio and Emara (1992) frame Material properties

Material	Property								
Reinforcement	Bar No.	As	dia	fy	Fu	Es	Esh	εsh	εsu
unit		(mm^2)	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(x10-3)	(x10-3)
	20	300	19.5	418	596	192500	3100	9.5	66.9
	10	100	11.3	454	640	192500*	3100*	9.5*	69.5
Concrete	f'c	ε0	Ec	Gc	Y				
unit	(MPa)	(x10-3)	(MPa)	(MPa)					
	30	1.85	23674	9864	0.2				

Two hinge characteristics were used to imitate the nonlinearity aspects of the above experimental frame: default and user-defined.

Figure A. 2 Default hinge property

Figure A. 3 User defined hinge property

Capacity curve results

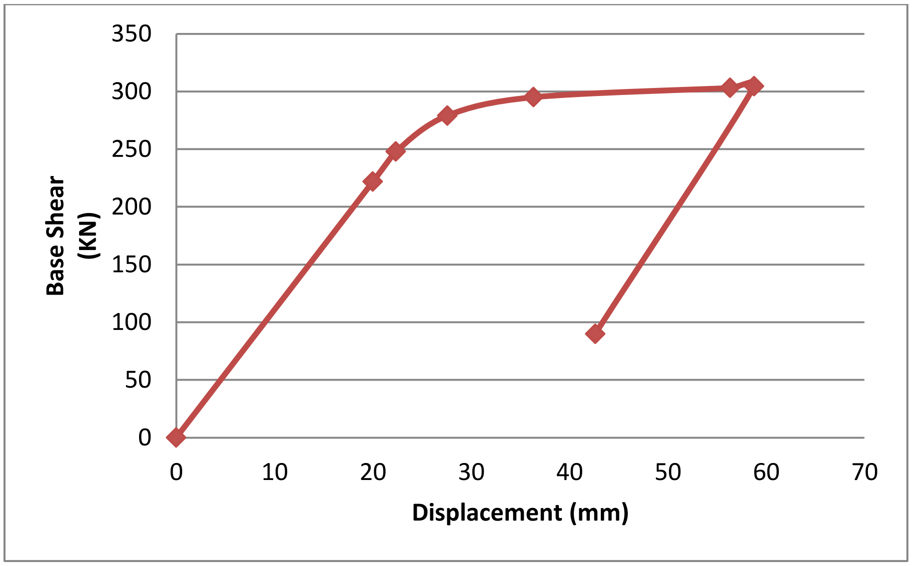

Figure A. 4 Capacity curve using Default hinge

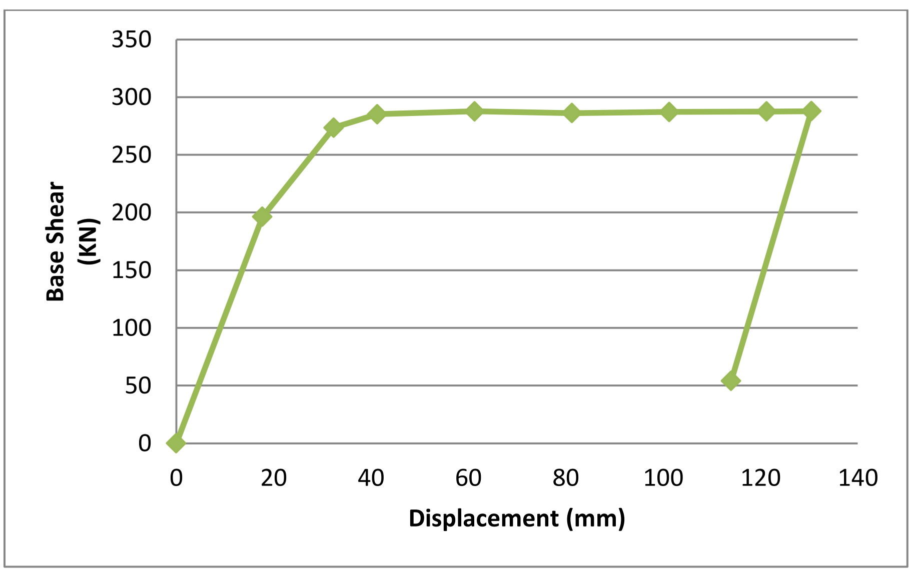

Figure A. 5 Capacity curve using user defined hinge

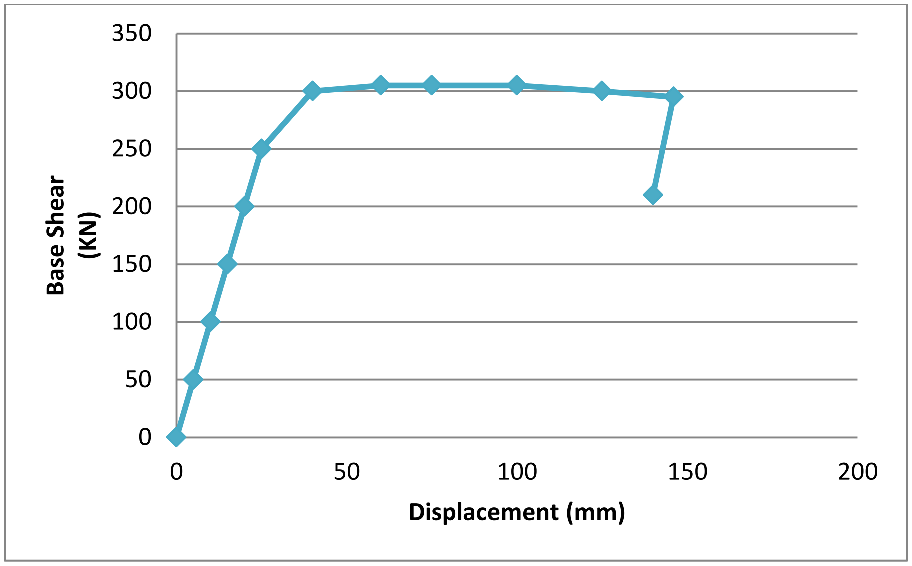

Figure A. 6 Capacity curve for Experimental

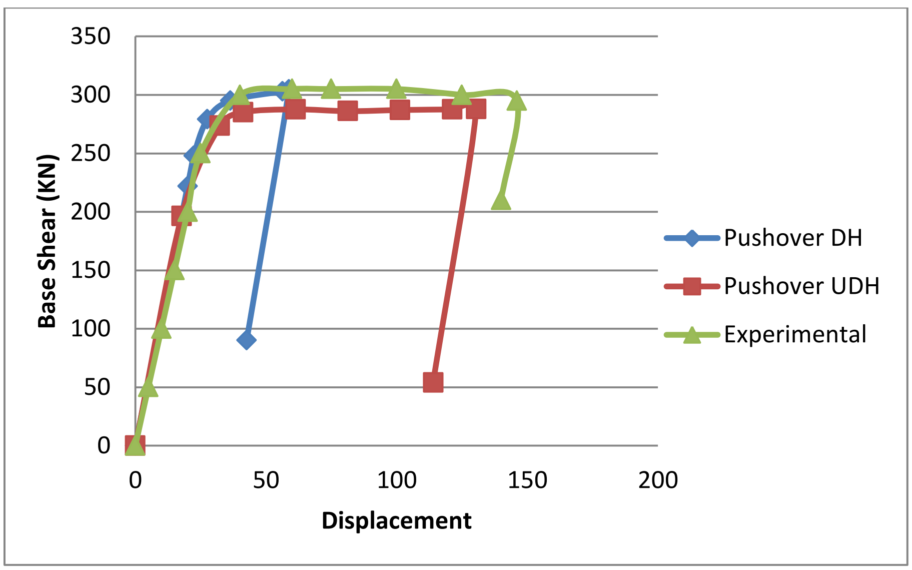

Figure A. 7 Capacity curve comparison of the selected frame

The comparison curve shows that there is very little difference in the analytical results between the user-defined hinge parameters and the experimental results. However, when default hinge characteristics are used, the outcome differs significantly from the results of experimental and user-defined hinge properties. According to the aforementioned findings, the deformation capacity of a structure is different for using different type of hinge definition which also affect the performance evaluations of structures .

APPENDIX B- Description of The Building

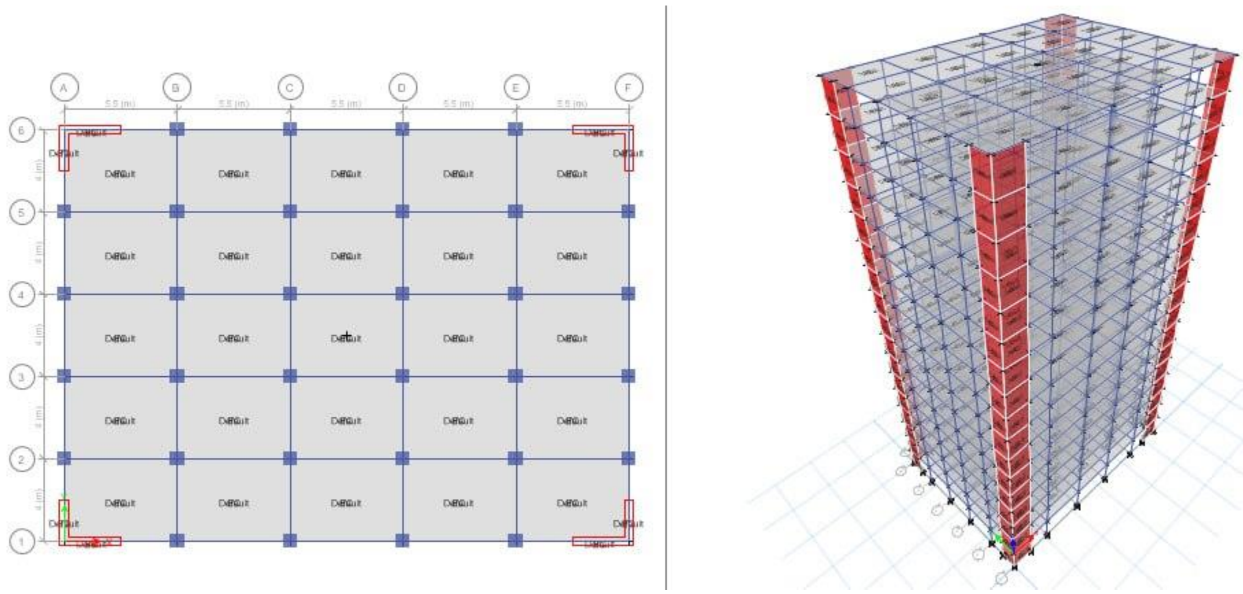

Figure B. 1 G+15 SWE CS

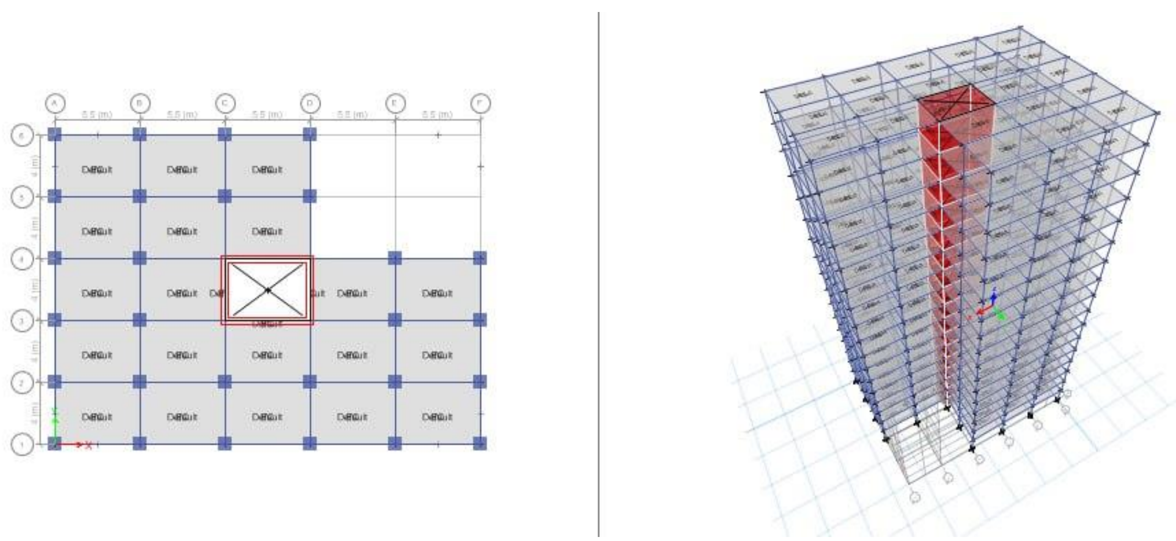

Figure B. 2G+15 SWC PI

Table B. 1 Values of φ for Calculating ψEi

Type of variable action	Storey	φ
Categories A-C*	Roof	1.0
	Storeys with correlated occupancies	0.8
	Independently occupied storeys	0.5
Categories D-F* and Archives		1.0

Table B. 2 Recommended Value of Ψ Factors for Buildings

Action		Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings				
Category A: Domestic, Residential areas		0.7	0.5	0.3
Category B: Office areas		0.7	0.5	0.3
Category C: Congregation areas		0.7	0.7	0.6
Category D: Shopping areas		0.7	0.7	0.6
Category E: Storage areas		1.0	0.9	0.8
Category F:	Traffic areas; vehicle weight ≤30kN	0.7	0.7	0.6
Category G:	Traffic areas; 30kN<vehicle weight ≤30kN	0.7	0.5	0.3
Category H: Roofs		0	0	0

Plastic Hinge distribution

2.SWC PLAN+ELEV REG V IRR

1.Push-X

Table B. 3 Number of Plastic hinge formation at different performance level with introduction of Irregularity in X Direction

a) G+5 SWC PI

Step	A-B	B-C	C-D	D-E	>E
0	1060	1572	0	0	0
1	1058	1574	0	0	0
2	1025	1607	0	0	0
3	978	1654	0	0	0
4	968	1664	0	0	0
5	955	1677	0	0	0
6	930	1702	0	0	0
7	923	1709	0	0	0
8	888	1744	0	0	0
9	865	1767	0	0	0
10	841	1791	0	0	0

b) G+15 SWC PI

Step	A-B	B-C	C-D	D-E	>E
0	6324	0	0	0	0
1	6324	0	0	0	0
2	6320	4	0	0	0
3	5985	339	0	0	0
4	4889	1435	0	0	0
5	4490	1834	0	0	0
6	4400	1924	0	0	0

7	4359	1965	0	0	0
8	4255	2069	0	0	0
9	4225	2099	0	0	0
10	4215	2109	0	0	0

c) G+30 SWC PI

Step	A-B	B-C	C-D	D-E	>E
0	11904	0	0	0	0
1	11900	4	0	0	0
2	11895	9	0	0	0
3	11810	94	0	0	0
4	10995	909	0	0	0
5	9994	1890	20	0	0
6	9599	2284	15	6	0
7	9578	2297	16	5	8
8	9178	2665	44	9	8
9	9000	2835	48	11	10

2.Push-Y

Table B. 4 Number of Plastic hinge formation at different performance level with introduction of Irregularity in X Direction

a) G+5 SWC PI

Step	A-B	B-C	C-D	D-E	>E
0	1090	1542	0	0	0
1	1078	1554	0	0	0
2	1032	1600	0	0	0
3	1022	1610	0	0	0
4	1045	1587	0	0	0

5	1039	1593	0	0	0
6	999	1633	0	0	0
7	955	1677	0	0	3

b) G+15 SWC PI.

Step	A-B	B-C	C-D	D-E	>E
0	6324	0	0	0	0
1	6320	4	0	0	0
2	6315	9	0	0	0
3	6320	4	0	0	0
4	6300	24	0	0	0
5	6289	35	0	0	0
6	6275	49	0	0	0
7	6248	76	0	0	0
8	6220	104	0	0	0
9	6199	125	0	0	0
10	6215	109	0	0	0
11	6180	144	0	0	0

a) G+30 SWC PI

Step	A-B	B-C	C-D	D-E	>E
0	11904	0	0	0	0
1	11899	5	0	0	0
2	11895	9	0	0	0
3	11866	38	0	0	0
4	11822	82	0	0	0

5	10895	993	11	5	0
6	10896	990	12	4	2
7	10757	1127	10	7	3
8	10555	1325	11	8	5

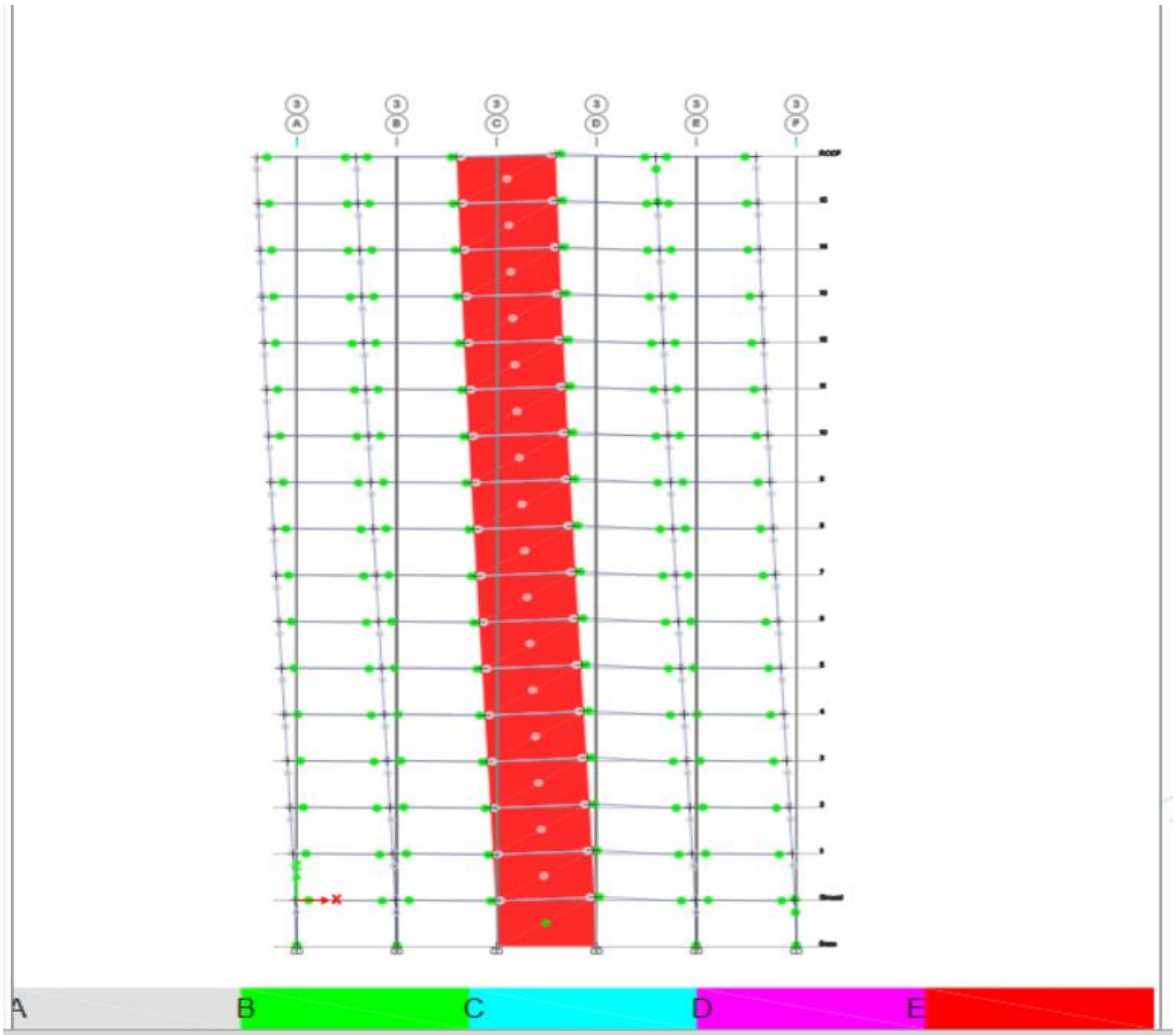

Figure B. 3 Plastic hinge distribution for G+15 SWC building

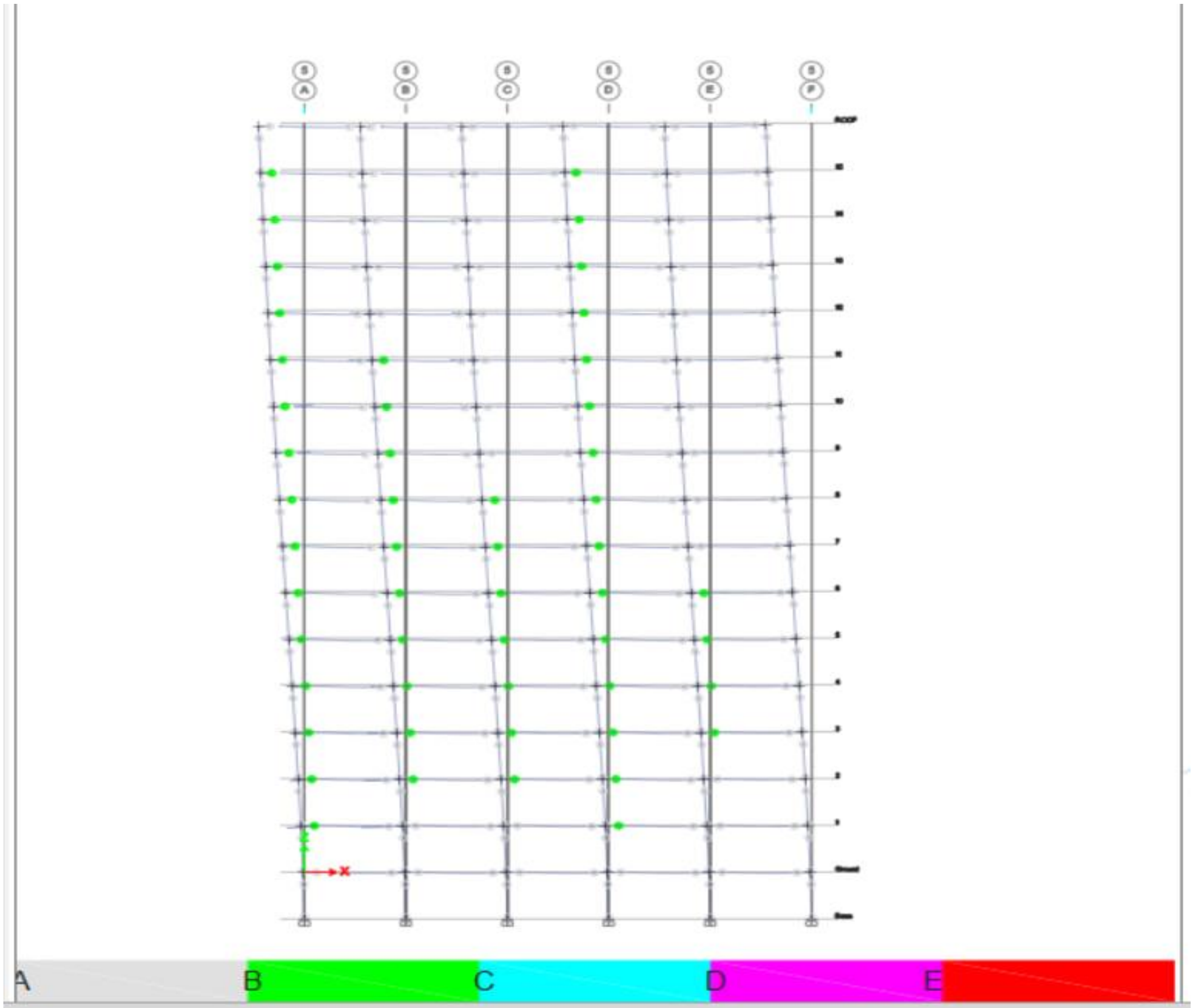

Figure B. 4 Plastic hinge distribution for G+15 SWE building

APPENDIX C

Table C. 1 Structural system classification

TABLE: Element Forces - Columns				
Story	Column	Output Case	V2	V3
Ground	C1	FY	0.5966	430.1127
Ground	C2	FY	0.174	505.9963
Ground	C3	FY	0.2381	508.8056
Ground	C4	FY	-0.2381	508.8058
Ground	C5	FY	-0.174	505.9976
Ground	C6	FY	-0.5966	430.1183
Ground	C7	FY	0.3742	428.5209
Ground	C8	FY	1.4843	504.5701
Ground	C9	FY	14.4315	505.4752
Ground	C10	FY	-14.4315	505.4755
Ground	C11	FY	-1.4843	504.5683
Ground	C12	FY	-0.3742	428.522
Ground	C13	FY	0.2527	540.4204
Ground	C14	FY	1.3534	540.4204
Ground	C17	FY	-1.3534	540.4204
Ground	C18	FY	-0.2527	540.4204
Ground	C19	FY	-0.2469	540.4204
Ground	C20	FY	-1.3499	540.4204
Ground	C23	FY	1.3499	540.4204
Ground	C24	FY	0.247	540.4204
Ground	C25	FY	-0.3683	540.4204
Ground	C26	FY	-1.4808	504.5747

Ground	C27	FY	-14.4303	505.308
Ground	C28	FY	14.4303	505.402
Ground	C29	FY	1.4808	504.6965
Ground	C30	FY	0.3683	428.5635
Ground	C31	FY	-0.591	430.126
Ground	C32	FY	-0.1707	506.0057
Ground	C33	FY	-0.237	508.6238
Ground	C34	FY	0.237	508.6336
Ground	C35	FY	0.1707	506.0231
Ground	C36	FY	0.591	430.1435
			x	y
			13555.23	14125.69
			18444.77	20000
		Column	42.36009	44.14278
		wall	57.63991	55.85722

Table C. 2 Imperfection Load

Story	Load Case/Comb	P (KN)	θo	l	m	αm	αh	θi	Hi
Roof	SLS	14631.61	0.005	3	32	0.71807	1	0.00359	26.26631
30	SLS	31391.06	0.005	3	32	0.71807	1	0.00359	30.08616
29	SLS	48150.51	0.005	3	32	0.71807	1	0.00359	30.08616
28	SLS	64909.96	0.005	3	32	0.71807	1	0.00359	30.08616
27	SLS	81669.41	0.005	3	32	0.71807	1	0.00359	30.08616

26	SLS	98428.86	0.005	3	32	0.71807	1	0.00359	30.08616
25	SLS	115746.7	0.005	3	32	0.71807	1	0.00359	31.08858
24	SLS	133064.5	0.005	3	32	0.71807	1	0.00359	31.08858
23	SLS	150382.4	0.005	3	32	0.71807	1	0.00359	31.08858
22	SLS	167700.2	0.005	3	32	0.71807	1	0.00359	31.08858
21	SLS	185018.1	0.005	3	32	0.71807	1	0.00359	31.08858
20	SLS	202335.9	0.005	3	32	0.71807	1	0.00359	31.08858
19	SLS	220277	0.005	3	32	0.71807	1	0.00359	32.20729
18	SLS	238218	0.005	3	32	0.71807	1	0.00359	32.20729
17	SLS	256159	0.005	3	32	0.71807	1	0.00359	32.20729
16	SLS	274100	0.005	3	32	0.71807	1	0.00359	32.20729
15	SLS	292041.1	0.005	3	32	0.71807	1	0.00359	32.20729
14	SLS	310992.2	0.005	3	32	0.71807	1	0.00359	34.02064
13	SLS	329943.4	0.005	3	32	0.71807	1	0.00359	34.02064
12	SLS	348894.5	0.005	3	32	0.71807	1	0.00359	34.02064
11	SLS	367845.7	0.005	3	32	0.71807	1	0.00359	34.02064

10	SLS	386796.8	0.005	3	32	0.71807	1	0.00359	34.02064
9	SLS	406587.7	0.005	3	32	0.71807	1	0.00359	35.52819
8	SLS	426378.6	0.005	3	32	0.71807	1	0.00359	35.52819
7	SLS	446169.6	0.005	3	32	0.71807	1	0.00359	35.52819
6	SLS	465960.5	0.005	3	32	0.71807	1	0.00359	35.52819
5	SLS	485751.4	0.005	3	32	0.71807	1	0.00359	35.52819
4	SLS	506170.5	0.005	3	32	0.71807	1	0.00359	36.65589
3	SLS	526589.6	0.005	3	32	0.71807	1	0.00359	36.65589
2	SLS	547008.7	0.005	3	32	0.71807	1	0.00359	36.65589
1	SLS	567427.9	0.005	3	32	0.71807	1	0.00359	36.65589
Ground	SLS	587847	0.005	3	32	0.71807	1	0.00359	36.65589

Table C. 3 Story Stiffness check

Story	Output Case	Stiff X	Stiff Y	
Roof	FY	0	234426.8	NOT OK
30	FY	0	436718.3	OK
29	FY	0	611239.1	OK
28	FY	0	754010.2	OK
27	FY	0	869766.5	OK
26	FY	0	959786.5	OK
25	FY	0	1046130	OK
24	FY	0	1109710	OK
23	FY	0	1163534	OK
22	FY	0	1209321	OK
21	FY	0	1249220	OK
20	FY	0	1280407	OK
19	FY	0	1324488	OK
18	FY	0	1354751	OK
17	FY	0	1384186	OK
16	FY	0	1412591	OK
15	FY	0	1437285	OK
14	FY	0	1477787	OK
13	FY	0	1509263	OK
12	FY	0	1544333	OK
11	FY	0	1583626	OK
10	FY	0	1627056	OK
9	FY	0	1692997	OK
8	FY	0	1760730	OK
7	FY	0	1848705	OK
6	FY	0	1965724	OK
5	FY	0	2129904	OK

4	FY	0	2378028	OK
3	FY	0	2731831	OK
2	FY	0	3359114	OK
1	FY	0	4745847	NOT OK
Ground	FY	0	11194647	OK

Table C. 4 Story Drift check

Story	Output Case	Direction	Drift	(Dr/h)*q	α	ν	α/ν	
Roof	EQ ENVELOP	X	0.000709	0.0011344	0.005	0.5	0.01	Ok!
Roof	EQ ENVELOP	Y	0.000853	0.0013648	0.005	0.5	0.01	Ok!
Roof	EQ ENVELOP	X	0.000772	0.0012352	0.005	0.5	0.01	Ok!
Roof	EQ ENVELOP	Y	0.000791	0.0012656	0.005	0.5	0.01	Ok!
Roof	RS ENVELOP	X	0.000837	0.0013392	0.005	0.5	0.01	Ok!
Roof	RS ENVELOP	Y	0.000769	0.0012304	0.005	0.5	0.01	Ok!
Roof	RS ENVELOP	X	0.000749	0.0011984	0.005	0.5	0.01	Ok!
Roof	RS ENVELOP	Y	0.000771	0.0012336	0.005	0.5	0.01	Ok!
30	EQ ENVELOP	X	0.000757	0.0012112	0.005	0.5	0.01	Ok!
30	EQ	Y	0.000899	0.0014384	0.005	0.5	0.01	Ok!

	ENVELOP							
30	EQ ENVELOP	X	0.000824	0.0013184	0.005	0.5	0.01	Ok!
30	EQ ENVELOP	Y	0.000835	0.001336	0.005	0.5	0.01	Ok!
30	RS ENVELOP	X	0.000892	0.0014272	0.005	0.5	0.01	Ok!
30	RS ENVELOP	Y	0.000808	0.0012928	0.005	0.5	0.01	Ok!
30	RS ENVELOP	X	0.000795	0.001272	0.005	0.5	0.01	Ok!
30	RS ENVELOP	Y	0.00081	0.001296	0.005	0.5	0.01	Ok!
29	EQ ENVELOP	X	0.000809	0.0012944	0.005	0.5	0.01	Ok!
29	EQ ENVELOP	Y	0.000949	0.0015184	0.005	0.5	0.01	Ok!
29	EQ ENVELOP	X	0.000881	0.0014096	0.005	0.5	0.01	Ok!
29	EQ ENVELOP	Y	0.000881	0.0014096	0.005	0.5	0.01	Ok!
29	RS ENVELOP	X	0.000953	0.0015248	0.005	0.5	0.01	Ok!
29	RS ENVELOP	Y	0.000845	0.001352	0.005	0.5	0.01	Ok!
29	RS ENVELOP	X	0.000841	0.0013456	0.005	0.5	0.01	Ok!
29	RS ENVELOP	Y	0.000847	0.0013552	0.005	0.5	0.01	Ok!
28	EQ ENVELOP	X	0.000868	0.0013888	0.005	0.5	0.01	Ok!
28	EQ	Y	0.001007	0.0016112	0.005	0.5	0.01	Ok!

	ENVELOP							
28	EQ ENVELOP	X	0.000946	0.0015136	0.005	0.5	0.01	Ok!
28	EQ ENVELOP	Y	0.000936	0.0014976	0.005	0.5	0.01	Ok!
28	RS ENVELOP	X	0.001022	0.0016352	0.005	0.5	0.01	Ok!
28	RS ENVELOP	Y	0.000885	0.001416	0.005	0.5	0.01	Ok!
28	RS ENVELOP	X	0.000889	0.0014224	0.005	0.5	0.01	Ok!
28	RS ENVELOP	Y	0.000887	0.0014192	0.005	0.5	0.01	Ok!
27	EQ ENVELOP	X	0.00093	0.001488	0.005	0.5	0.01	Ok!
27	EQ ENVELOP	Y	0.001071	0.0017136	0.005	0.5	0.01	Ok!
27	EQ ENVELOP	X	0.001013	0.0016208	0.005	0.5	0.01	Ok!
27	EQ ENVELOP	Y	0.000995	0.001592	0.005	0.5	0.01	Ok!
27	RS ENVELOP	X	0.001094	0.0017504	0.005	0.5	0.01	Ok!
27	RS ENVELOP	Y	0.000925	0.00148	0.005	0.5	0.01	Ok!
27	RS ENVELOP	X	0.000934	0.0014944	0.005	0.5	0.01	Ok!
27	RS ENVELOP	Y	0.000927	0.0014832	0.005	0.5	0.01	Ok!
26	EQ ENVELOP	X	0.000996	0.0015936	0.005	0.5	0.01	Ok!
26	EQ	Y	0.001141	0.0018256	0.005	0.5	0.01	Ok!

	ENVELOP							
26	EQ ENVELOP	X	0.001086	0.0017376	0.005	0.5	0.01	Ok!
26	EQ ENVELOP	Y	0.00106	0.001696	0.005	0.5	0.01	Ok!
26	RS ENVELOP	X	0.001172	0.0018752	0.005	0.5	0.01	Ok!
26	RS ENVELOP	Y	0.000966	0.0015456	0.005	0.5	0.01	Ok!
26	RS ENVELOP	X	0.000978	0.0015648	0.005	0.5	0.01	Ok!
26	RS ENVELOP	Y	0.000969	0.0015504	0.005	0.5	0.01	Ok!
25	EQ ENVELOP	X	0.00103	0.001648	0.005	0.5	0.01	Ok!
25	EQ ENVELOP	Y	0.001188	0.0019008	0.005	0.5	0.01	Ok!
25	EQ ENVELOP	X	0.001123	0.0017968	0.005	0.5	0.01	Ok!
25	EQ ENVELOP	Y	0.001103	0.0017648	0.005	0.5	0.01	Ok!
25	RS ENVELOP	X	0.001213	0.0019408	0.005	0.5	0.01	Ok!
25	RS ENVELOP	Y	0.000988	0.0015808	0.005	0.5	0.01	Ok!
25	RS ENVELOP	X	0.000994	0.0015904	0.005	0.5	0.01	Ok!
25	RS ENVELOP	Y	0.00099	0.001584	0.005	0.5	0.01	Ok!
24	EQ ENVELOP	X	0.001091	0.0017456	0.005	0.5	0.01	Ok!
24	EQ	Y	0.00126	0.002016	0.005	0.5	0.01	Ok!

	ENVELOP							
24	EQ ENVELOP	X	0.001192	0.0019072	0.005	0.5	0.01	Ok!
24	EQ ENVELOP	Y	0.00117	0.001872	0.005	0.5	0.01	Ok!
24	RS ENVELOP	X	0.001287	0.0020592	0.005	0.5	0.01	Ok!
24	RS ENVELOP	Y	0.001026	0.0016416	0.005	0.5	0.01	Ok!
24	RS ENVELOP	X	0.001031	0.0016496	0.005	0.5	0.01	Ok!
24	RS ENVELOP	Y	0.001028	0.0016448	0.005	0.5	0.01	Ok!
23	EQ ENVELOP	X	0.001148	0.0018368	0.005	0.5	0.01	Ok!
23	EQ ENVELOP	Y	0.001328	0.0021248	0.005	0.5	0.01	Ok!
23	EQ ENVELOP	X	0.001255	0.002008	0.005	0.5	0.01	Ok!
23	EQ ENVELOP	Y	0.001232	0.0019712	0.005	0.5	0.01	Ok!
23	RS ENVELOP	X	0.001356	0.0021696	0.005	0.5	0.01	Ok!
23	RS ENVELOP	Y	0.001059	0.0016944	0.005	0.5	0.01	Ok!
23	RS ENVELOP	X	0.001062	0.0016992	0.005	0.5	0.01	Ok!
23	RS ENVELOP	Y	0.001062	0.0016992	0.005	0.5	0.01	Ok!
22	EQ ENVELOP	X	0.001202	0.0019232	0.005	0.5	0.01	Ok!
22	EQ	Y	0.001395	0.002232	0.005	0.5	0.01	Ok!

	ENVELOP							
22	EQ ENVELOP	X	0.001315	0.002104	0.005	0.5	0.01	Ok!
22	EQ ENVELOP	Y	0.001294	0.0020704	0.005	0.5	0.01	Ok!
22	RS ENVELOP	X	0.001422	0.0022752	0.005	0.5	0.01	Ok!
22	RS ENVELOP	Y	0.001092	0.0017472	0.005	0.5	0.01	Ok!
22	RS ENVELOP	X	0.001092	0.0017472	0.005	0.5	0.01	Ok!
22	RS ENVELOP	Y	0.001095	0.001752	0.005	0.5	0.01	Ok!
21	EQ ENVELOP	X	0.001251	0.0020016	0.005	0.5	0.01	Ok!
21	EQ ENVELOP	Y	0.001458	0.0023328	0.005	0.5	0.01	Ok!
21	EQ ENVELOP	X	0.001371	0.0021936	0.005	0.5	0.01	Ok!
21	EQ ENVELOP	Y	0.001352	0.0021632	0.005	0.5	0.01	Ok!
21	RS ENVELOP	X	0.001483	0.0023728	0.005	0.5	0.01	Ok!
21	RS ENVELOP	Y	0.001123	0.0017968	0.005	0.5	0.01	Ok!
21	RS ENVELOP	X	0.001117	0.0017872	0.005	0.5	0.01	Ok!
21	RS ENVELOP	Y	0.001126	0.0018016	0.005	0.5	0.01	Ok!
20	EQ ENVELOP	X	0.001305	0.002088	0.005	0.5	0.01	Ok!
20	EQ	Y	0.001527	0.0024432	0.005	0.5	0.01	Ok!

	ENVELOP							
20	EQ ENVELOP	X	0.001431	0.0022896	0.005	0.5	0.01	Ok!
20	EQ ENVELOP	Y	0.001416	0.0022656	0.005	0.5	0.01	Ok!
20	RS ENVELOP	X	0.00155	0.00248	0.005	0.5	0.01	Ok!
20	RS ENVELOP	Y	0.001159	0.0018544	0.005	0.5	0.01	Ok!
20	RS ENVELOP	X	0.001147	0.0018352	0.005	0.5	0.01	Ok!
20	RS ENVELOP	Y	0.001162	0.0018592	0.005	0.5	0.01	Ok!
19	EQ ENVELOP	X	0.001293	0.0020688	0.005	0.5	0.01	Ok!
19	EQ ENVELOP	Y	0.001543	0.0024688	0.005	0.5	0.01	Ok!
19	EQ ENVELOP	X	0.00142	0.002272	0.005	0.5	0.01	Ok!
19	EQ ENVELOP	Y	0.001429	0.0022864	0.005	0.5	0.01	Ok!
19	RS ENVELOP	X	0.001539	0.0024624	0.005	0.5	0.01	Ok!
19	RS ENVELOP	Y	0.001161	0.0018576	0.005	0.5	0.01	Ok!
19	RS ENVELOP	X	0.00113	0.001808	0.005	0.5	0.01	Ok!
19	RS ENVELOP	Y	0.001165	0.001864	0.005	0.5	0.01	Ok!
18	EQ ENVELOP	X	0.001337	0.0021392	0.005	0.5	0.01	Ok!
18	EQ	Y	0.001603	0.0025648	0.005	0.5	0.01	Ok!

	ENVELOP							
18	EQ ENVELOP	X	0.00147	0.002352	0.005	0.5	0.01	Ok!
18	EQ ENVELOP	Y	0.001483	0.0023728	0.005	0.5	0.01	Ok!
18	RS ENVELOP	X	0.001595	0.002552	0.005	0.5	0.01	Ok!
18	RS ENVELOP	Y	0.001193	0.0019088	0.005	0.5	0.01	Ok!
18	RS ENVELOP	X	0.001157	0.0018512	0.005	0.5	0.01	Ok!
18	RS ENVELOP	Y	0.001197	0.0019152	0.005	0.5	0.01	Ok!
17	EQ ENVELOP	X	0.00137	0.002192	0.005	0.5	0.01	Ok!
17	EQ ENVELOP	Y	0.001651	0.0026416	0.005	0.5	0.01	Ok!
17	EQ ENVELOP	X	0.001509	0.0024144	0.005	0.5	0.01	Ok!
17	EQ ENVELOP	Y	0.001527	0.0024432	0.005	0.5	0.01	Ok!
17	RS ENVELOP	X	0.00164	0.002624	0.005	0.5	0.01	Ok!
17	RS ENVELOP	Y	0.001217	0.0019472	0.005	0.5	0.01	Ok!
17	RS ENVELOP	X	0.001176	0.0018816	0.005	0.5	0.01	Ok!
17	RS ENVELOP	Y	0.001222	0.0019552	0.005	0.5	0.01	Ok!
16	EQ ENVELOP	X	0.001402	0.0022432	0.005	0.5	0.01	Ok!
16	EQ	Y	0.001695	0.002712	0.005	0.5	0.01	Ok!

	ENVELOP							
16	EQ ENVELOP	X	0.001545	0.002472	0.005	0.5	0.01	Ok!
16	EQ ENVELOP	Y	0.001566	0.0025056	0.005	0.5	0.01	Ok!
16	RS ENVELOP	X	0.001682	0.0026912	0.005	0.5	0.01	Ok!
16	RS ENVELOP	Y	0.00124	0.001984	0.005	0.5	0.01	Ok!
16	RS ENVELOP	X	0.001195	0.001912	0.005	0.5	0.01	Ok!
16	RS ENVELOP	Y	0.001246	0.0019936	0.005	0.5	0.01	Ok!
15	EQ ENVELOP	X	0.001434	0.0022944	0.005	0.5	0.01	Ok!
15	EQ ENVELOP	Y	0.001739	0.0027824	0.005	0.5	0.01	Ok!
15	EQ ENVELOP	X	0.001583	0.0025328	0.005	0.5	0.01	Ok!
15	EQ ENVELOP	Y	0.001606	0.0025696	0.005	0.5	0.01	Ok!
15	RS ENVELOP	X	0.001725	0.00276	0.005	0.5	0.01	Ok!
15	RS ENVELOP	Y	0.001264	0.0020224	0.005	0.5	0.01	Ok!
15	RS ENVELOP	X	0.001215	0.001944	0.005	0.5	0.01	Ok!
15	RS ENVELOP	Y	0.00127	0.002032	0.005	0.5	0.01	Ok!
14	EQ ENVELOP	X	0.001437	0.0022992	0.005	0.5	0.01	Ok!
14	EQ	Y	0.001756	0.0028096	0.005	0.5	0.01	Ok!

	ENVELOP							
14	EQ ENVELOP	X	0.001589	0.0025424	0.005	0.5	0.01	Ok!
14	EQ ENVELOP	Y	0.00162	0.002592	0.005	0.5	0.01	Ok!
14	RS ENVELOP	X	0.001735	0.002776	0.005	0.5	0.01	Ok!
14	RS ENVELOP	Y	0.001271	0.0020336	0.005	0.5	0.01	Ok!
14	RS ENVELOP	X	0.001215	0.001944	0.005	0.5	0.01	Ok!
14	RS ENVELOP	Y	0.001278	0.0020448	0.005	0.5	0.01	Ok!
13	EQ ENVELOP	X	0.001457	0.0023312	0.005	0.5	0.01	Ok!
13	EQ ENVELOP	Y	0.001785	0.002856	0.005	0.5	0.01	Ok!
13	EQ ENVELOP	X	0.001616	0.0025856	0.005	0.5	0.01	Ok!
13	EQ ENVELOP	Y	0.001645	0.002632	0.005	0.5	0.01	Ok!
13	RS ENVELOP	X	0.001766	0.0028256	0.005	0.5	0.01	Ok!
13	RS ENVELOP	Y	0.001288	0.0020608	0.005	0.5	0.01	Ok!
13	RS ENVELOP	X	0.001231	0.0019696	0.005	0.5	0.01	Ok!
13	RS ENVELOP	Y	0.001295	0.002072	0.005	0.5	0.01	Ok!
12	EQ ENVELOP	X	0.00147	0.002352	0.005	0.5	0.01	Ok!
12	EQ	Y	0.001802	0.0028832	0.005	0.5	0.01	Ok!

	ENVELOP							
12	EQ ENVELOP	X	0.001634	0.0026144	0.005	0.5	0.01	Ok!
12	EQ ENVELOP	Y	0.001659	0.0026544	0.005	0.5	0.01	Ok!
12	RS ENVELOP	X	0.001787	0.0028592	0.005	0.5	0.01	Ok!
12	RS ENVELOP	Y	0.001297	0.0020752	0.005	0.5	0.01	Ok!
12	RS ENVELOP	X	0.001241	0.0019856	0.005	0.5	0.01	Ok!
12	RS ENVELOP	Y	0.001304	0.0020864	0.005	0.5	0.01	Ok!
11	EQ ENVELOP	X	0.001475	0.00236	0.005	0.5	0.01	Ok!
11	EQ ENVELOP	Y	0.00181	0.002896	0.005	0.5	0.01	Ok!
11	EQ ENVELOP	X	0.001644	0.0026304	0.005	0.5	0.01	Ok!
11	EQ ENVELOP	Y	0.001663	0.0026608	0.005	0.5	0.01	Ok!
11	RS ENVELOP	X	0.0018	0.00288	0.005	0.5	0.01	Ok!
11	RS ENVELOP	Y	0.001301	0.0020816	0.005	0.5	0.01	Ok!
11	RS ENVELOP	X	0.001248	0.0019968	0.005	0.5	0.01	Ok!
11	RS ENVELOP	Y	0.001308	0.0020928	0.005	0.5	0.01	Ok!
10	EQ ENVELOP	X	0.001476	0.0023616	0.005	0.5	0.01	Ok!
10	EQ	Y	0.001809	0.0028944	0.005	0.5	0.01	Ok!

	ENVELOP							
10	EQ ENVELOP	X	0.001651	0.0026416	0.005	0.5	0.01	Ok!
10	EQ ENVELOP	Y	0.001659	0.0026544	0.005	0.5	0.01	Ok!
10	RS ENVELOP	X	0.001809	0.0028944	0.005	0.5	0.01	Ok!
10	RS ENVELOP	Y	0.0013	0.00208	0.005	0.5	0.01	Ok!
10	RS ENVELOP	X	0.001253	0.0020048	0.005	0.5	0.01	Ok!
10	RS ENVELOP	Y	0.001306	0.0020896	0.005	0.5	0.01	Ok!
9	EQ ENVELOP	X	0.001448	0.0023168	0.005	0.5	0.01	Ok!
9	EQ ENVELOP	Y	0.001778	0.0028448	0.005	0.5	0.01	Ok!
9	EQ ENVELOP	X	0.001625	0.0026	0.005	0.5	0.01	Ok!
9	EQ ENVELOP	Y	0.001628	0.0026048	0.005	0.5	0.01	Ok!
9	RS ENVELOP	X	0.001783	0.0028528	0.005	0.5	0.01	Ok!
9	RS ENVELOP	Y	0.00128	0.002048	0.005	0.5	0.01	Ok!
9	RS ENVELOP	X	0.001238	0.0019808	0.005	0.5	0.01	Ok!
9	RS ENVELOP	Y	0.001285	0.002056	0.005	0.5	0.01	Ok!
8	EQ ENVELOP	X	0.00143	0.002288	0.005	0.5	0.01	Ok!
8	EQ	Y	0.001748	0.0027968	0.005	0.5	0.01	Ok!

	ENVELOP							
8	EQ ENVELOP	X	0.00161	0.002576	0.005	0.5	0.01	Ok!
8	EQ ENVELOP	Y	0.001598	0.0025568	0.005	0.5	0.01	Ok!
8	RS ENVELOP	X	0.001768	0.0028288	0.005	0.5	0.01	Ok!
8	RS ENVELOP	Y	0.001262	0.0020192	0.005	0.5	0.01	Ok!
8	RS ENVELOP	X	0.00123	0.001968	0.005	0.5	0.01	Ok!
8	RS ENVELOP	Y	0.001266	0.0020256	0.005	0.5	0.01	Ok!
7	EQ ENVELOP	X	0.001397	0.0022352	0.005	0.5	0.01	Ok!
7	EQ ENVELOP	Y	0.001697	0.0027152	0.005	0.5	0.01	Ok!
7	EQ ENVELOP	X	0.001577	0.0025232	0.005	0.5	0.01	Ok!
7	EQ ENVELOP	Y	0.001549	0.0024784	0.005	0.5	0.01	Ok!
7	RS ENVELOP	X	0.001734	0.0027744	0.005	0.5	0.01	Ok!
7	RS ENVELOP	Y	0.001229	0.0019664	0.005	0.5	0.01	Ok!
7	RS ENVELOP	X	0.00121	0.001936	0.005	0.5	0.01	Ok!
7	RS ENVELOP	Y	0.001232	0.0019712	0.005	0.5	0.01	Ok!
6	EQ ENVELOP	X	0.001347	0.0021552	0.005	0.5	0.01	Ok!
6	EQ	Y	0.001624	0.0025984	0.005	0.5	0.01	Ok!

	ENVELOP							
6	EQ ENVELOP	X	0.001527	0.0024432	0.005	0.5	0.01	Ok!
6	EQ ENVELOP	Y	0.00148	0.002368	0.005	0.5	0.01	Ok!
6	RS ENVELOP	X	0.00168	0.002688	0.005	0.5	0.01	Ok!
6	RS ENVELOP	Y	0.00118	0.001888	0.005	0.5	0.01	Ok!
6	RS ENVELOP	X	0.001177	0.0018832	0.005	0.5	0.01	Ok!
6	RS ENVELOP	Y	0.001183	0.0018928	0.005	0.5	0.01	Ok!
5	EQ ENVELOP	X	0.001278	0.0020448	0.005	0.5	0.01	Ok!
5	EQ ENVELOP	Y	0.001523	0.0024368	0.005	0.5	0.01	Ok!
5	EQ ENVELOP	X	0.001454	0.0023264	0.005	0.5	0.01	Ok!
5	EQ ENVELOP	Y	0.001387	0.0022192	0.005	0.5	0.01	Ok!
5	RS ENVELOP	X	0.001602	0.0025632	0.005	0.5	0.01	Ok!
5	RS ENVELOP	Y	0.001111	0.0017776	0.005	0.5	0.01	Ok!
5	RS ENVELOP	X	0.001127	0.0018032	0.005	0.5	0.01	Ok!
5	RS ENVELOP	Y	0.001114	0.0017824	0.005	0.5	0.01	Ok!
4	EQ ENVELOP	X	0.001173	0.0018768	0.005	0.5	0.01	Ok!
4	EQ	Y	0.001383	0.0022128	0.005	0.5	0.01	Ok!

	ENVELOP							
4	EQ ENVELOP	X	0.001339	0.0021424	0.005	0.5	0.01	Ok!
4	EQ ENVELOP	Y	0.001258	0.0020128	0.005	0.5	0.01	Ok!
4	RS ENVELOP	X	0.001478	0.0023648	0.005	0.5	0.01	Ok!
4	RS ENVELOP	Y	0.001013	0.0016208	0.005	0.5	0.01	Ok!
4	RS ENVELOP	X	0.001043	0.0016688	0.005	0.5	0.01	Ok!
4	RS ENVELOP	Y	0.001015	0.001624	0.005	0.5	0.01	Ok!
3	EQ ENVELOP	X	0.001055	0.001688	0.005	0.5	0.01	Ok!
3	EQ ENVELOP	Y	0.001222	0.0019552	0.005	0.5	0.01	Ok!
3	EQ ENVELOP	X	0.001209	0.0019344	0.005	0.5	0.01	Ok!
3	EQ ENVELOP	Y	0.001111	0.0017776	0.005	0.5	0.01	Ok!
3	RS ENVELOP	X	0.001335	0.002136	0.005	0.5	0.01	Ok!
3	RS ENVELOP	Y	0.000896	0.0014336	0.005	0.5	0.01	Ok!
3	RS ENVELOP	X	0.000943	0.0015088	0.005	0.5	0.01	Ok!
3	RS ENVELOP	Y	0.000897	0.0014352	0.005	0.5	0.01	Ok!
2	EQ ENVELOP	X	0.000298	0.0004768	0.005	0.5	0.01	Ok!
2	EQ	Y	0.000691	0.0011056	0.005	0.5	0.01	Ok!

	ENVELOP							
2	EQ ENVELOP	X	3.17E-07	5.0672E-07	0.005	0.5	0.01	Ok!
2	EQ ENVELOP	Y	0.000347	0.0005552	0.005	0.5	0.01	Ok!
2	RS ENVELOP	X	0.00089	0.001424	0.005	0.5	0.01	Ok!
2	RS ENVELOP	Y	0.00101	0.001616	0.005	0.5	0.01	Ok!
2	RS ENVELOP	X	0.001024	0.0016384	0.005	0.5	0.01	Ok!
2	RS ENVELOP	Y	0.000918	0.0014688	0.005	0.5	0.01	Ok!
1	EQ ENVELOP	X	0.000092	0.0001472	0.005	0.5	0.01	Ok!
1	EQ ENVELOP	Y	0.000092	0.0001472	0.005	0.5	0.01	Ok!
1	EQ ENVELOP	X	0.000076	0.0001216	0.005	0.5	0.01	Ok!
1	EQ ENVELOP	Y	0.000511	0.0008176	0.005	0.5	0.01	Ok!
1	RS ENVELOP	X	0.000217	0.0003472	0.005	0.5	0.01	Ok!
1	RS ENVELOP	Y	0.000512	0.0008192	0.005	0.5	0.01	Ok!
1	RS ENVELOP	X	1.9E-07	3.0448E-07	0.005	0.5	0.01	Ok!
1	RS ENVELOP	Y	0.000258	0.0004128	0.005	0.5	0.01	Ok!
Ground	EQ ENVELOP	X	1.01E-07	1.6208E-07	0.005	0.5	0.01	Ok!
Ground	EQ	Y	8.95E-08	1.43152E-	0.005	0.5	0.01	Ok!

	ENVELOP			07				
Ground	EQ ENVELOP	X	0.000036	0.0000576	0.005	0.5	0.01	Ok!
Ground	EQ ENVELOP	Y	0.000036	0.0000576	0.005	0.5	0.01	Ok!
Ground	RS ENVELOP	X	0.000043	0.0000688	0.005	0.5	0.01	Ok!
Ground	RS ENVELOP	Y	0.000043	0.0000688	0.005	0.5	0.01	Ok!
Ground	RS ENVELOP	X	0.000034	0.0000544	0.005	0.5	0.01	Ok!
Ground	RS ENVELOP	Y	0.000233	0.0003728	0.005	0.5	0.01	Ok!

Table C. 5 Torsion check input value

Story	Mass X	Mass Y	Mass Moment of Inertia
	ton	Ton	ton-m2
ROOF	893.6824	893.6824	99227.7122
15	966.7745	966.7745	111925.3086
14	977.6663	977.6663	113164.9932
13	1090.3588	1090.3588	124723.9008
12	1090.8557	1090.8557	124835.7999
11	1090.8557	1090.8557	124835.7999
10	1090.8557	1090.8557	124835.7999
9	1120.7192	1120.7192	128068.5135
8	1139.0687	1139.0687	130190.1713
7	1139.0687	1139.0687	130190.1713
6	1139.0687	1139.0687	130190.1713
5	1139.0687	1139.0687	130190.1713
4	1155.2264	1155.2264	132081.3935

3	1176.0225	1176.0225	134485.939
2	1176.0225	1176.0225	134485.939
1	1176.0225	1176.0225	134485.939
Ground	1176.0225	1176.0225	134485.939

Story	Mass X	Mass Y	Mass Moment of Inertia	Isx,y	KX	KY	RZ	MZ	RZ new	KM	rx	ry
	ton	kg	ton-m²		kN/m	kN/m						
ROOF	893.6824	893682.4	99227.71	111.0324	333841.2	347228.5	0.000451	1000	0.000017	58823529	13.27412	13.01571
15	966.7745	966774.5	111925.3	115.7719	588924.2	605952.7	0.000434	2000	0.000017	1.18E+08	14.13386	13.93385
14	977.6663	977666.3	113165	115.7501	793436.9	820879	0.000417	3000	0.00002	1.5E+08	13.74958	13.5178
13	1090.359	1090359	124723.9	114.3879	923574.6	956567.9	0.000397	4000	0.000021	1.9E+08	14.36099	14.11115
12	1090.856	1090856	124835.8	114.4384	1017007	1054352	0.000376	5000	0.000024	2.08E+08	14.31257	14.05681
11	1090.856	1090856	124835.8	114.4384	1089647	1132064	0.000352	6000	0.000026	2.31E+08	14.55279	14.27754
10	1090.856	1090856	124835.8	114.4384	1149582	1202001	0.000326	7000	0.000028	2.5E+08	14.74688	14.42174
9	1120.719	1120719	128068.5	114.2735	1238902	1324343	0.000298	8000	0.000029	2.76E+08	14.92202	14.43264
8	1139.069	1139069	130190.2	114.2953	1292308	1389231	0.000269	9000	0.000031	2.9E+08	14.98847	14.45617
7	1139.	1139	1301	114.2	1345	1448	0.000	100	0.000	3.03E	15.00	14.46

	069	069	90.2	953	108	991	238	00	033	+08	944	139
6	1139.069	1139069	130190.2	114.2953	1402340	1510403	0.000205	110000	0.000034	3.24E+09	48.03196	46.28182
5	1139.069	1139069	130190.2	114.2953	1469869	1580194	0.000171	120000	0.000035	3.43E+09	48.29669	46.58021
4	1155.226	1155226	132081.4	114.3338	1600105	1728145	0.000136	13000	0.000034	3.82E+08	15.45817	14.87449
3	1176.023	1176023	134485.9	114.3566	1746206	1851977	0.000102	14000	0.000033	4.24E+08	15.58688	15.13524
2	1176.023	1176023	134485.9	114.3566	2012807	2061244	0.000069	15000	0.000031	4.84E+08	15.50471	15.32146
1	1176.023	1176023	134485.9	114.3566	2628874	2553205	0.000038	160000	0.000025	6.4E+09	49.34068	50.06649
Ground	1176.023	1176023	134485.9	114.3566	5596142	5153708	0.000013	170000	0.000013	1.31E+10	48.3402	50.37243

Table C. 6 Torsion check

	Direction X			Direction Y		
Story	r_x (m)	l_s (m)	Status	r_y (m)	l_s (m)	Status
ROOF	13.27412	111.03	Not Ok!	13.02	111.03	Not Ok!
15	14.13386	115.77	Not Ok!	13.93	115.77	Not Ok!
14	13.74958	115.75	Not Ok!	13.52	115.75	Not Ok!
13	14.36099	114.39	Not Ok!	14.11	114.39	Not Ok!
12	14.31257	114.44	Not Ok!	14.06	114.44	Not Ok!
11	14.55279	114.44	Not Ok!	14.28	114.44	Not Ok!
10	14.74688	114.44	Not Ok!	14.42	114.44	Not Ok!
9	14.92202	114.27	Not Ok!	14.43	114.27	Not Ok!
8	14.98847	114.30	Not Ok!	14.46	114.30	Not Ok!
7	15.00944	114.30	Not Ok!	14.46	114.30	Not Ok!
6	48.03196	114.30	Not Ok!	46.28	114.30	Not Ok!
5	48.29669	114.30	Not Ok!	46.58	114.30	Not Ok!

4	15.46	114.33	Not Ok!	14.87	114.33	Not Ok!
3	15.59	114.36	Not Ok!	15.14	114.36	Not Ok!
2	15.50	114.36	Not Ok!	15.32	114.36	Not Ok!
1	49.34	114.36	Not Ok!	50.07	114.36	Not Ok!
Ground	48.34	114.36	Not Ok!	50.37	114.36	Not Ok!