JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

A COMPARATIVE STUDY ON RETROFITTING OF REINFORCED CONCRETE BUILDING USING STEEL BRACING AND INFILL WALL

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial

Fulfillment of the Requirements for the Degree of Master of Science in Structural Engineering

BY

BILISTU ABDO HAJI

APRIL, 2021

JIMMA, ETHIOPIA

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Advisor: - Eng. Elmer C.Agon (Asso. Prof)

Co-advisor:- Eng. Goshu Kenea (M.Sc.)

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DECLARATION

I hereby declare that this research entitled "A Comparative Study on Retrofitting of Reinforced Concrete Building Using Steel Bracing and Infill Wall" is my original work, and has not been submitted by any other person for an award of a degree in this or any other University.

Name: Bilistu Abdo Haji

Signature Date

__________ ________________

This thesis approved by:

Signature Date

Advisor: Eng. Elmer C.Agon (Asso. Prof) ____________ ___________

Co- Advisor: Eng. Goshu Kenea (M.Sc.) ___________ __________

ABSTRACT

For different reasons, such as changes in service conditions, environmental conditions, seismic code, or low quality of operation, it may become necessary to upgrade or strengthen an existing building. Many techniques may used to retrofit concrete structures on a local or global basis. The main objective of this study is to compare steel bracing and infill wall intervention techniques for retrofitting RC buildings.

Four, seven, and ten storey moment-resisting frames of medium ductility class were designed according to ES-EN 2015 for high seismicity region (Zone V), to study the effect of infill wall and steel bracing on seismic response of RC buildings by comparing with bare frame, masonry infill wall and steel bracing (concentric X-bracing) were introduced at the corner bays of the ground floor separately. The modeling and design of the building were done by using ETABSv18.1.1 structural design and analysis software. Linear dynamic and nonlinear static (pushover) analyses were conducted to evaluate the seismic response of the building.

From the linear dynamic analysis it was found that, adding infill wall to the bare frame reduces top floor displacement by 8%, 6%, and 2%, the maximum drift at the critical storey of the building by 6%, 6% ,and 2%, and increases the average lateral stiffness of the building by 13%, 10%, and 5 %,adding steel bracing to the bare frame reduces the top floor displacement by 21%, 20% ,and 12%, the maximum drift at the critical storey of the building by 13%, 24%, and 11% ,and increases the average lateral stiffness by 64%,58 %, and 57% ,for G+3, G+6, and G+9 buildings respectively. From the conducted nonlinear analysis, the pushover curve shows that both retrofitting techniques increases the lateral load-carrying capacity and they change the pattern and order of plastic hinge formation in the building, by preventing plastic hinges from developing in the columns at lower stories. The lateral load carrying capacity was increased by 6.76% for infilled frame, and by 21.09% for braced frame. As the height of the building increases infill wall is found to be not as effective as steel bracing.

Keywords: *Seismic retrofitting, infill wall, Steel bracing, Response Spectrum analysis, Pushover analysis, ETABS*

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ACRONYMS

CQC	Complete Quadratic Combination
DCM	Ductility Class Medium
EN	European Norms
ES	Ethiopian Standards
ETABS	Extended Three-dimensional Analysis of Buildings
FEMA	Federal Emergency Management Agency
HCB	Hollow Concrete Block
IS	Indian Standard
RC	Reinforced Concrete
SRSS	Square root of the sum of squares

LIST OF NOTATIONS

$S_d(T)$ design spectrum (for elastic analysis)

S soil factor

T_B lower limit of the period of the constant spectral acceleration branch;

T_C upper limit of the period of the constant spectral acceleration branch;

T_D the value defining the beginning of the constant displacement response range of the spectrum;

β lower bound factor for the horizontal design spectrum.

a_g design ground acceleration on type A ground

q behaviour factor

q_0 basic value of the behaviour factor

k_w factor associated with the prevailing failure mode in structural system with walls.

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

$\psi_{E,I}$ combination coefficient for a variable action i, to be used when determining the effects of the design seismic action

e_{ai} accidental eccentricity

h_{col} Column height between centerlines of beams

h_{inf} Height of infill panel

E_{fe} Expected modulus of elasticity of frame material

E_{me} Expected modulus of elasticity of infill material

I_{col} Moment of inertia of column

L_{inf} Length of infill panel

r_{inf} Diagonal length of infill panel

t_{inf} Thickness of infill panel and equivalent strut

CHAPTER ONE
INTRODUCTION

1.1 Background of the study

A reinforced concrete building should be designed to possess a capacity to carry combined loads (dead, live, and seismic loads) at a certain safety level and at a certain degree of reliability. However, this ideal condition is not always realized, Performance of structural building could be below the expected criteria in terms of safety level and service life by various causes.

Retrofitting is technical interventions in the structural system of a building that improves the resistance to the earthquakes by optimizing the strength, ductility, and earthquake loads. It also proves to be a better option catering to the economic considerations and immediate shelter problems rather than replacement of seismic deficient buildings. Strengthening schemes are implemented as part of either post-earthquake restoration of buildings or as pre-earthquake preparedness. There are many different techniques to retrofit an existing RC building. They can be classified as local and global retrofitting.

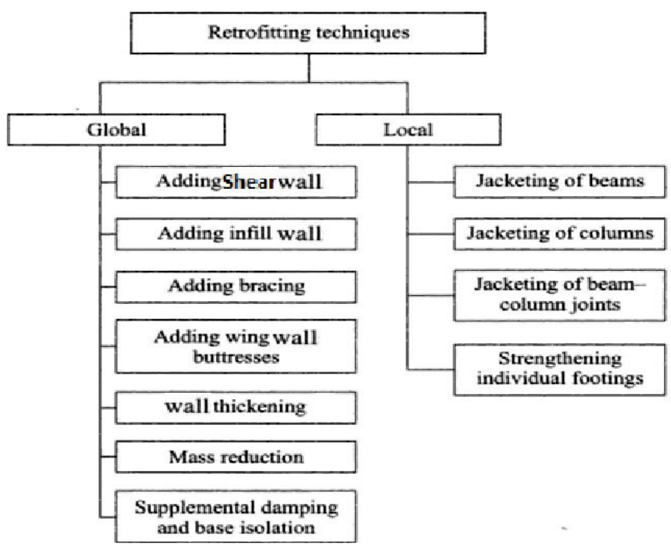


Figure 1.1 Retrofitting techniques

In the present study, analytical investigations were carried out to evaluate the effectiveness of the adding steel bracing and infill wall strengthening schemes in improving the strength and ductility of RC buildings.

1.2 Statement of the Problem

The assessment of seismic vulnerability and strengthening of existing buildings is a topic of relevant importance and priority. Performance of structural building could be below the expected criteria in terms of safety level and service life due to faulty design and low-quality construction. Also, Changes in service conditions, often made arbitrarily, may lead to substantial changes in the structural behaviour resulting in a degradation of the structural response to the expected loading conditions [1]. As well as cause of vulnerability is connected with the maintenance of constructions, it is obvious that if a construction is not regularly maintained, the mechanical properties of the materials may undergo local and global degradation with a significant loss of resistance of the structural members and of the entire construction. In addition, other situations could impair the performance of structural building such as changes of seismic codes, change of environmental conditions, etc.

Based on what has been presented so far, in areas long known to be subject to seismic hazard, constructions vulnerable to earthquakes may found. These constructions need to be retrofitted to allow them to withstand the effects of the earthquake ground motion expected at the site considered.

Many techniques can be used to retrofit concrete structures, such as adding steel braces, post-tensioned cables, infill walls, shear walls, and base isolators to the structure which is carried out on a global basis or it can be performed on a local basis by retrofitting the existing structural elements.

To develop an appropriate retrofitting strategy, experimental and analytical studies should be conducted. In this study, the response of reinforced concrete buildings retrofitted with HCB masonry infill wall and steel bracing (X-bracing) were studied to investigate the better seismic retrofitting intervention, by using ETABSv18.1.1 structural analysis software.

1.3 Objective of the Study

1.3.1 General objective

The general objective of this study is to compare steel bracing and infill wall retrofitting techniques in improving seismic response of RC buildings.

1.3.2 Specific objective

1) To model an imaginary G+3,G+6,and G+9 reinforced concrete buildings according to (ES-EN 2015) with structural analysis software.
2) To examine the response such as lateral storey displacement, storey drift, base shear, overturning moment and storey stiffness for the bare frame, infilled frame, and braced frame by using response spectrum analysis.
3) To evaluate the lateral load-carrying capacity and plastic hinge formation pattern of the bare frame, infilled frame, and braced frames by using pushover analysis.
4) To identify the retrofit option that would improve more the capacity of RC buildings.

1.4 Significance of the Study

In this study, four, seven, and ten storey frames have been analyzed with and without retrofit techniques to understand the structural behavior under seismic action. These can serve as a guideline for designers to consider and analyze the possible seismic strengthening schemes during the assessment and analysis phase, and improves understanding of students and academic researchers interested in this area by filling the gap of knowledge regarding retrofitting of RC building with infill wall and steel bracing. Moreover, the result of the study can aid to understand more the concept of seismic retrofitting of structures and which method can be applicable or repeatable for future purposes.

1.5 Scope and Limitations of the Study

This study aims at evaluating the response and performance of multi storey open ground RC building retrofitted by using infill wall and steel bracing retrofitting techniques against seismic loads and suggesting better retrofitting technique that decrease lateral displacement and storey drift, and increase lateral stiffness and load carrying capacity of the building using the structural engineering software ETABSv18.1.1. Here Response spectrum and pushover analysis have been experienced.

Although retrofitting is adopted after the performance assessment of the existing buildings, due to lack of full existing building data for the selected seismic zone, an imaginary building is modeled and designed to compare the two intervention methods.

1.6 Content of the thesis

The thesis is organized in different sections which are arranged as follows:

a) Section one deals with an introductory part which include background of the study, statement of the problem, objective of the study, significance of the study, scope and limitations of the study and contents of the thesis.
b) Section two briefly reviews theoretical background of retrofittinng,infill walls and bracing systems,classifications, principles and behaviors are considered.
c) Section three discusses about the methods of analysis and modeling of structural systems.
d) Section four presents, result and discussion comparison made on lateral displacement, storey drift, storey stiffness and load carrying capacity for bare,infilled,and braced frame, and were made with the help of figures and tables.
e) Finally, conclusions and recommendation were drawn and forwarded respectively to show research areas for the next researchers.

CHAPTER TWO
REVIEW OF RELATED LITERATURE

2.1 Earthquake Resistant Design

Most earthquakes occur through the sudden movement of earth crust in faults zones. The sudden movement releases strain energy and causes seismic waves through the crust around the fault. These seismic waves cause the ground surface to shake and this ground shaking is the principal concern of structural engineering to resist earthquakes among many other effects [2].

(a) (b)

Figure 2.1 Building failures during earthquake (a) L'Aquila 2009, (b) Izmit 1999 [3]

The primary objective of earthquake resistant design is to prevent building collapse during earthquakes to minimize the risk of death or injury to people.

ES-EN1998:2015 asks for a two level seismic design establishing explicitly the two following requirements [4].

No-collapse requirement: The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events

Damage limitation requirement: The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of itself.

In order to satisfy the fundamental requirements in ultimate limit states and damage limitation states shall be checked. Ultimate limit states are those associated with collapse or with other forms of structural failure which might endanger the safety of people. Damage limitation states are those associated with damage beyond which specified service requirements are no longer met.

2.2 Structural Types

As per ES-EN 2015 Concrete buildings shall be classified into one of the following structural types according to their behavior under horizontal seismic actions [4].

a) **frame system;** structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system

b) **dual system (frame or wall equivalent);**structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls coupled or uncoupled

- **wall equivalent;** dual system in which the shear resistance of the walls at the building base is higher than 50% of the total seismic resistance of the whole structural system
- **frame equivalent;** dual system in which the shear resistance of the frame system at the building base is greater than 50% of the total shear resistance of the whole structural system

c) **ductile wall system (coupled or uncoupled);**wall fixed at the base so that the relative rotation of the base with respect to the rest of the structural system is prevented, and that is designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large perforations, just above its base.

d) **system of large lightly reinforced walls**; wall with large cross-sectional dimensions, that is, a horizontal dimension *l*w at least equal to 4.0 m or two-thirds of the height *h*w of the wall, whichever is less, which is expected to develop limited cracking and inelastic behavior under the seismic design situation

e)) **inverted pendulum system;** system in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element

f) **torsionally flexible system**; dual or wall system not having a minimum torsional rigidity.

$(rx \geq ls)$

where rx is the square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction ("torsional radius"); and ls is the radius of gyration of the floor mass in plan (square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the centre of mass of the floor to (b) the floor mass).

2.3 Seismic Retrofitting of Concrete Structures

Retrofitting is the basic overall approach adopted to improve the probable seismic performance of the building or to otherwise reduce the existing risk to an acceptable level [7].Retrofitting ensures the safety and security of a building, employees, structure functionality, machinery and inventory. Retrofitting also proves to be a better option catering to the economic considerations and immediate shelter problems rather than replacement of seismic deficient buildings [8].

The primary objective of introducing retrofit measures was to minimize structural irregularities, correct discontinuities, complete the load path, uniformize inter-storey drift and obtain a regular structure for improving the seismic performance [9].

Retrofitting can be applied for both Earthquake vulnerable and Earthquake damaged structures.

2.3.1 Seismic retrofitting techniques

There are two approaches to enhance the seismic capacity of existing structures. The first approach is a structure-level retrofit, which involves global modifications to the structural system. Common global modifications include the addition of infill walls, steel braces, or base isolators [10]. Global retrofit strategy improves the performance of the entire building under lateral loads, by increasing the strength and ductility of structure. Global stiffening of the structure may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive lateral deflection of the building, and critical components do not have adequate ductility to resist the resulting deformations [5].

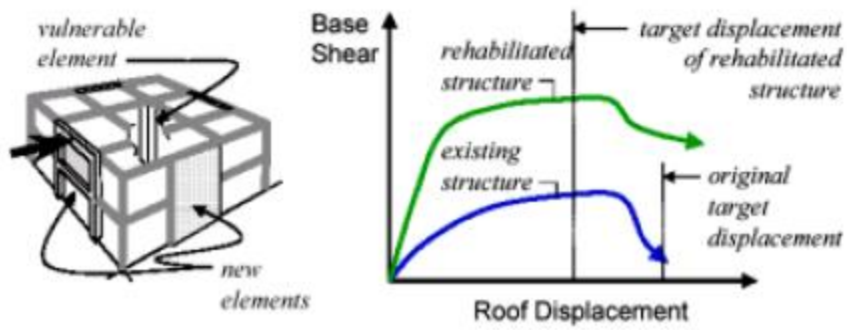


Figure 2.2 Global modification of the structural system [6]

The second approach is a member-level retrofit, the maintenance of local deficiencies in a building such as crushing of columns, flexure and shear failure of beams, columns, and shear walls. In this approach, the ductility of components with inadequate capacities is increased to satisfy their specific limit states. The member-level retrofit includes methods such as the addition of concrete, steel, or fiber reinforced polymer (FRP) jackets to columns for confinement [5].

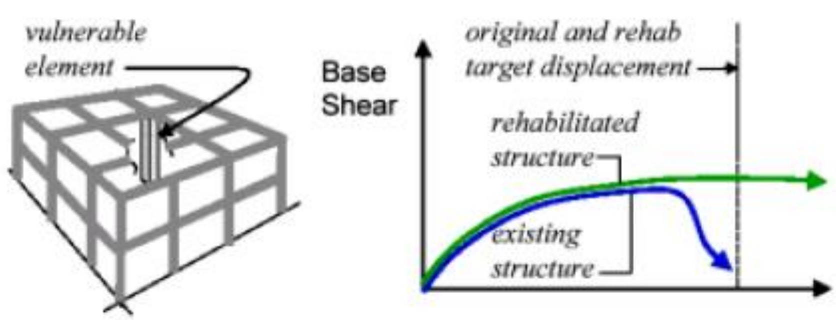


Figure 2.3 Local modification of the structural components [6]

2.4 Steel Bracing

The addition of steel bracing systems for seismic retrofitting of RC buildings is a widespread successfully applied technique all over the World since the late ‘70s.Steel bracing has been used to stabilize laterally the majority of the world’s tallest building structures as well as one of the major retrofit measures. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear. A bracing system improves the seismic performance of the frame by increasing its lateral stiffness and capacity. Through the addition of the bracing system, load would be transferred out of the frame and into the braces, bypassing the weak columns while increasing strength. Steel braced frames are efficient structural systems for buildings subjected to seismic or wind lateral loadings. Therefore, the use of steel bracing systems for retrofitting reinforced concrete frames with inadequate lateral resistance is attractive [7].

2.4.1 Types of Bracing

The general classification of bracing based on their geometrical arrangements or connection styles were grouped in to two (i.e. concentrically bracing and eccentrically bracing).

Concentric bracings: Concentrically Braced Frames (CBFs) are a class of structures resisting lateral loads through a vertical concentric truss system, the axes of the members aligning concentrically at the joints [8]. Concentric bracings increase the lateral stiffness of the frame thus increases the natural frequency and also usually decreases the lateral storey drift.

Eccentric Bracings: Eccentrically braced frames (EBFs) are a relatively new lateral force resisting system developed to resist seismic events in a predictable manner. Properly designed and detailed EBFs behave in a ductile manner through shear or flexural yielding of a link element [8]. Eccentric Bracings reduce the lateral stiffness of the system and improve the energy dissipation capacity. The lateral stiffness of the system depends upon the flexural stiffness property of the beams and columns, thus reducing the lateral stiffness of the frame. The vertical component of the bracing forces due to earthquake causes lateral concentrated load on the beams at the point of connection of the eccentric bracings [7].

Figure 2.4 Application of Concentric bracings for retrofitting *[9]*

2.4.2 Steel Brace Connection to RC frame

Different bracing methods fall into two main categories, namely (1) external bracing; and (2) internal bracing. In the external bracing method, existing buildings are retrofitted by attaching a local or global steel bracing system to the exterior (and occasionally interior) frames. Architectural concerns and difficulties encountered when connecting the steel bracing to the RC frames are two of the main shortcomings of this method. In the internal bracing method, the buildings are retrofitted by positioning a bracing system inside the individual bays of the RC frames. The bracing may be attached to the RC frame either indirectly or directly. In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is carried out indirectly through the steel frame. This method of internal bracing can be costly and technical difficulties in fixing the steel frame to the RC frame can be inhibiting. The direct internal bracing method, first proposed by Maheri and Sahebi, overcomes the aforementioned shortcomings of the indirect internal bracing system. In this method the steel braces are directly connected to the RC frames without the use of an intermediary steel frame .The direct internal bracing method was proposed not only as a retrofit measure for existing buildings, but also as a shear-resisting element to be used in the seismic design of new buildings [10].

To connect the plate to an existing concrete member, holes may be drilled into the concrete and anchor bolts fixed in position inside the holes using appropriate adhesive resins. Alternatively, if the concrete member dimensions or the strength of adhesive inhibit the use of this method on the grounds of insufficient development length, holes may be drilled through the concrete member so that the bolts can be anchored at the opposite face with an appropriate back plate and nuts [10].

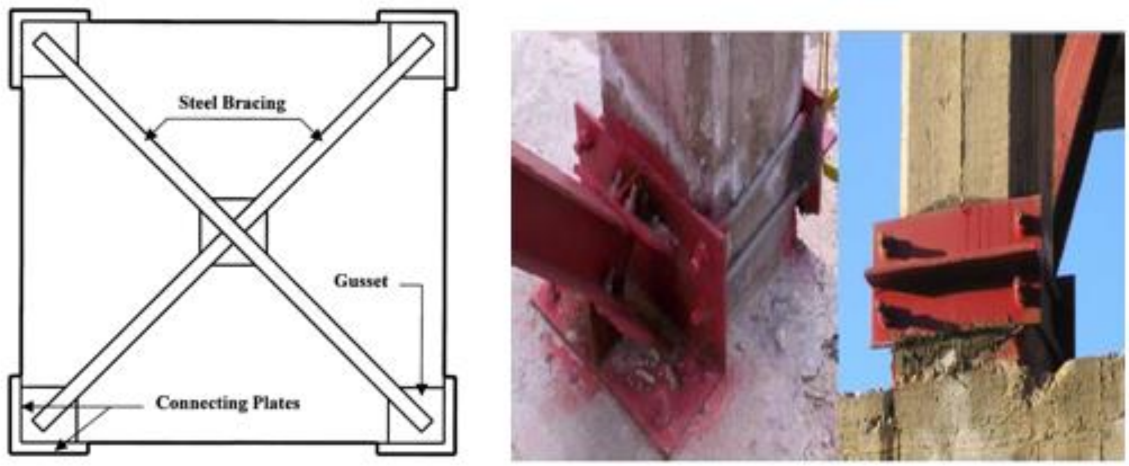


Figure 2.5 Direct internal bracing of RC frames*[10]*

2.5 Infill walls

The addition of infill walls significantly helped to increase the rigidity and capacity of the structure. Field evidence has shown that continuous infill masonry walls can significantly reduce the vulnerability of a reinforced concrete structure [11]. Earlier studies concluded that, the presence of infills leads to a decrease in shear force in the columns, as “nonstructural” infills takes part in resistance to seismic forces.

2.5.1 Masonry Infill Wall Modelling Approaches

Several methods have been developed on modelling infills, and they are grouped in two main categories: macro-models, based on the equivalent strut method, and micro-models, based on the finite element method [12].

Micro-modelling

The micro-modelling approach considers the effect of the mortar joints as discrete element in the model. Considering the fact of mortar joints are the weakest plane in a masonry infill wall, this approach can be considered the most exact [13].

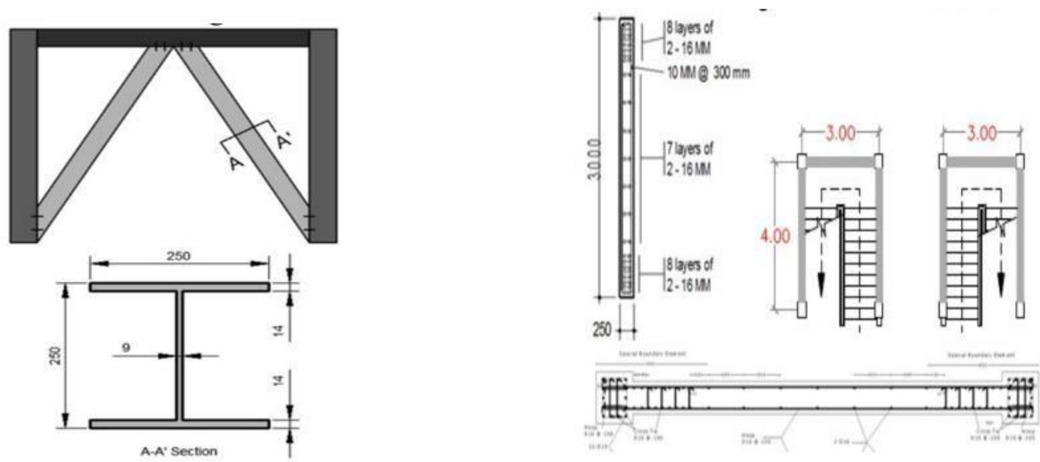
Macro-modelling

It refers to analyses that use frame elements and typically takes the infill presence into consideration through equivalent strut models. This approach is faster and easier to apply with today's computational tools and speeds is of greater interest for designers and engineers [14].

2.6 Related Previous works

Research done on the use of steel bracing to improve seismic performance of reinforced concrete building. Three methods of seismic evaluation were employed for the purpose of the study i.e. Nonlinear Static Pushover Displacement Coefficient Method as described in FEMA 356, Improvement of Nonlinear Static Pushover Displacement Coefficient Method as described in FEMA 440 and dynamic time history analysis following the Indonesian Code of Seismic Resistance Building (SNI 03-1726-2002) criteria. The results show that the target displacement determined from nonlinear pushover analysis of the existing building in X direction is 0.188 m and in Y direction is 0.132 m. The performance of the building could be categorized in between Life Safety (LS) - Collapse Prevention (CP) and plastic hinges occur in columns. Also for the non-strengthened building the story drifts in Y direction exceed the serviceability limit criterion when the recorded El Centro accelerogram was used for dynamic time history analysis. The authors concluded that the addition of steel bracings improves the seismic performance of the existing building. As show on their study from the nonlinear pushover analysis that target displacements in both directions are reduced by 16%-55% when steel bracings are used. Furthermore, dynamic time history analysis points out that the story drifts of the retrofitted building are within the limit criteria. Meanwhile, the size of steel bracing elements do not significantly affect the seismic performance of retrofitted building [15].

The researchers examined the performance and structural strength of a school public building in West Sumatera retrofiton with shear wall and steel bracing. In their study, the performance and structural strength of a two-story RC school building (SMAN3 Batusangkar) designed using previous seismic code (SNI03-1726-2002) and constructed before 2009, was evaluated based on the new seismic code. Calculation and structural analysis are applied by a three-dimensional structure made of a computer program, ETABS 9.7.1. The result of evaluation on the SMAN3 Batusangkar building shows that the building cannot resist the working loads applied to the structure, especially the earthquake loads. They propose two retrofitting systems, shear wall and steel bracing to increase the resistant capacity of structure, which installed on the building frame with different locations.



a) b)

Figure 2.6 Specification detail of a) steel bracing and b) shear wall[16]

Structural analysis using ETABS v9.7.1 was carried out after strengthening the building and they concluded that, retrofitting of the building by adding steel bracing and shear wall systems are very effective for reducing the displacement by 60-99% and the internal force by 10-95% compared to the existing structure [16].

Researchers analyzed 3 and 5 story RC buildings on pilotis, having brick infill walls in all stories except the ground story. They are space frame structures with two different plan layouts: one symmetric and the other nonsymmetric, the latter with an elevator shaft located in a corner of the building and causing bidirectional eccentricity, with ex=0.15 and ey=0.19. Dimensioning of the original buildings was done according to the old Greek codes for reinforced concrete and for earthquake resistant design. All four buildings were strengthened using diagonal steel braces in corner bays of their open ground stories. Their goal was to limit the interstory drift of the ground story to a level comparable to the interstory drift of the story above, and then compare the response of the original and the strengthened building for the selected earthquake action. Seismic capacity of the buildings before and after strengthening was investigated using nonlinear time history dynamic analyses, based on Part-3 of Eurocode 8 (EC8-Part 3, 2005) for assessment and retrofitting of buildings by using the computer program Ruaumoko 3D.8

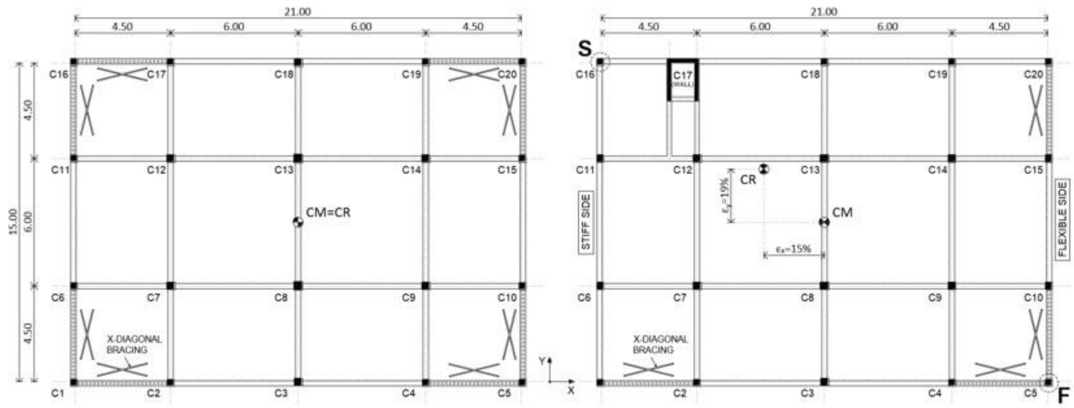


Figure 2.7 Typical layouts for the 3-story and 5-story buildings[17]

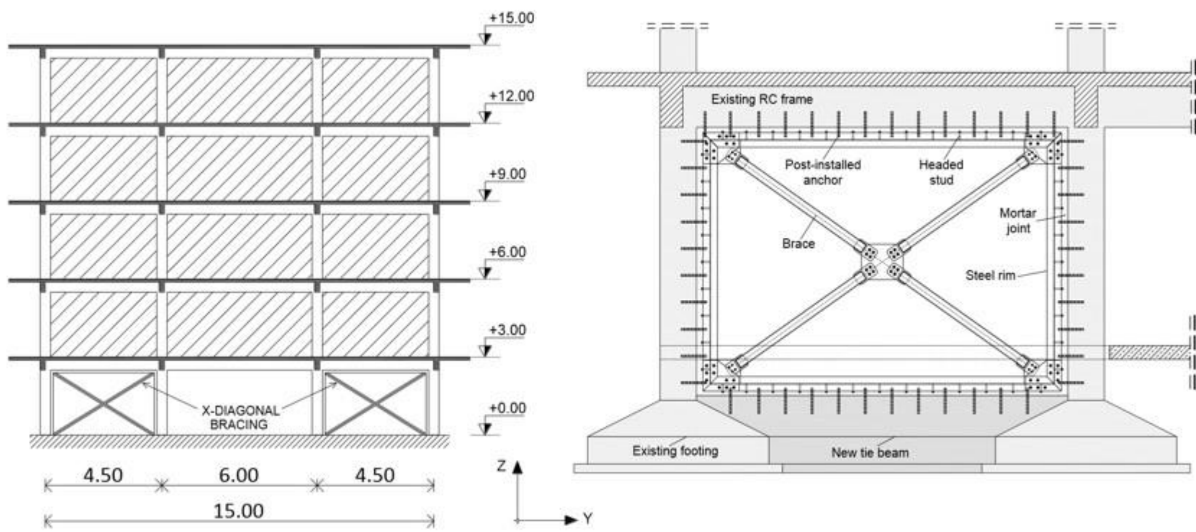


Figure 2.8 Elevation of the 5-story buildings and typical detail of X-bracing for strengthening the ground story[17]

The researchers concluded that, in both the symmetric and non-symmetric cases showed greatly improved response, which met the set objective of removing the ground story weakness without moving the problem to higher stories. Note also that with the selected bracing locations in the case of non-symmetric buildings, it was possible to drastically reduce the ground story eccentricity, and through that, the undesirable torsional response of the building. It is believed that the proposed retrofitting scheme, which is perhaps the only feasible way of strengthening a building with open ground story that would be acceptable to its owners, could indeed save such a building from collapse or very heavy damage in a future catastrophic earthquake [17].

Researchers highlights the principles of assessing and retrofitting of structure against seismic events. . The different retrofitting methods such as steel and concrete jacketing and application of fibre reinforced polymer (FRP) composites which were used to improve the load bearing capacity of individual structure elements were highlighted and methods such as shear walls and shear cores which can be used to improve overall stability of buildings. A three dimensional R.C. frame designed with linear elastic dynamic analysis using response spectrum method. The computer software package STAAD Pro was used for dynamics analysis technique was used to assess the performance of a reinforced concrete building. From their analysis most retrofitting techniques results an increase in stiffness and slightly increase in mass which causes in return a shorter period. Shortening in period of vibration often results an increase in strength and ductility of retrofitted structure. The authors summarized that a proposed retrofit scheme can be said to be successful if it results an increase in strength and ductility capacity of the structure which is greater than the demands imposed by earthquakes [2].

Research done on the effectiveness of seismic retrofitting of multi-storey multi-bay RC-frame buildings by converting selected bays into new walls through infilling with RC walls experimentally on a full scale specimen at the ELSA facility of the Joint Research Centre at Ispra. The authors demonstrated that the method was a viable method for retrofitting existing deficient structures [18].

Researchers evaluated the seismic vulnerability of the existing open ground storeyed buildings by introducing masonry infills of varying thickness (5 inches and 10 inches) at strategic locations in the ground level, as a retrofitting measure by using SAP2000. The authors concluded that, placing masonry infills at strategic locations at the ground level would decrease the seismic vulnerability to a greater extent, by reducing the hinge formation to a lower level (IO and Linear), thereby preventing collapse and irrepairable damages [19].

Research done on the effect of infill masonry panels on the seismic response of RC frame buildings. Two different types of infill panels confined in frame have been tested. Panels were made of hollow clay tile blocks, and hollow concrete blocks. The tests have been made under cyclic loading. The author concluded that infill panels increases the lateral stiffness and strength of the bare frame. The equivalent diagonal representing the confined panels transform the rigid frame into trussed frame, and there is a definite change in the form in which the frame will resist lateral loads; flexural effects will decrease substantially [20].

CHAPTER THREE
RESEARCH METHODOLOGY

3.1 Research Design

The data used in this study was analyzed and interpreted in terms of quantitative data, by using, ETABSv18.1.1 structural analysis software.

The methodology was designed as shown in chart below.

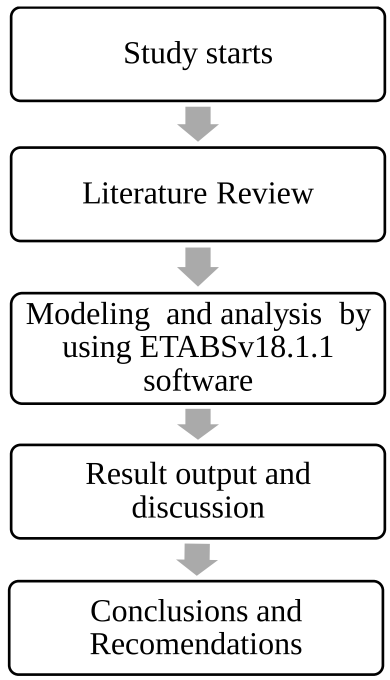


Figure 3.1 Flow chart of the methodology used

3.2 Study Variables

3.2.1 Dependent variables

In this study the dependent variables were performance of infill wall and steel bracing as a retrofitting method.

3.2.2 Independent variables

The independent variables/parameters that used in this study to compare infill wall and steel bracing retrofitting techniques were lateral displacement, storey drift, storey stiffness base shear, overturning moment, pushover curve, and plastic hinge formation pattern of the building.

3.3 Materials

The materials used in the design were selected based on ES-EN 1998-2015, for the design of DCM (ductility class medium) building, Concrete of a class lower than C 16/20 shall not be used in primary seismic elements. According to (ES-EN 1998-1/4.3.1 (7)) unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the un cracked elements. Torsional stiffness of the cracked section shall be equal to 10% of the torsional stiffness of the uncracked section. In critical regions of primary seismic elements reinforcing steel of class B or C in ES-EN 1992-1-1:2015, Table C.1 shall be used.

Table 3-1 Material Properties

Grade of concrete	C25/30 For column E=31000 MPa Shear Modulus =12916.67 Poisson ratio $v=0.2$
	C20/25 for beam and slab E=30000 MPa Shear Modulus =12500MPa Poisson ratio $v=0.2$
Grade of steel	Rebar S-400 Structural steel S 355 Yield strength f_y= 355 MPa Modulus of elasticity E =210000 MPa Poisson's ratio $v= 0.3$ Shear modulus G $G = E / [2 \cdot (1 + v)] \approx 81000$ MPa
HCB compressive strength[21]	6.08Mpa
Masonry compressive strength	4.243 Mpa
Modulus of Elasticity of masonry (Em)	2333.65 Mpa

3.4 Methods of analysis

Earthquake loads are to be carefully modelled so as to assess the real behavior of structure with a clear understanding. In this study modal response spectrum (linear dynamic) and pushover (nonlinear static) analyses were used.

3.4.1 Modal response spectrum analysis

Linear Dynamic analysis is applied to buildings whose response is significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. In Response spectrum method, square root of the sum of squares (SSRS) and complete quadratic combination (CQC) is used as the directional combination and modal combination respectively. In this study a sufficient number of modes are used in the analysis in order of achieving the sum of the modal mass of all modes equals to 90% of the total seismic mass as explained by ES- EN 1998 section 4.3.3.3

The inertial effects of the design seismic action were evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\sum G_{k,j} \text{"+"} \sum \psi_{E,i} \cdot Q_{k,I} \tag{3.1}$$

The combination coefficients ψ_{Ei} for the calculation of the effects of the seismic actions are computed from the following expression:

$$\psi_{Ei} = \varphi . \psi_{2i} \tag{3.2}$$

According to ES EN 1998-1-2015 section 3.2.2.5 For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions:

$$0 < T \leq T_B : S_d(T) = a_g . S . \left[\frac{2}{3} + \frac{T}{T_B} . \left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{3.3}$$

$$T_B < T \leq T_C : S_d(T) = a_g . S . \eta . \frac{2.5}{q} \tag{3.4}$$

$$T_B < T \leq T_C : S_d(T) = \begin{cases} a_g . S . \eta . \frac{2.5}{q} . \left[\frac{T_C}{T}\right] \\ \geq \beta . a_g \end{cases} \tag{3.5}$$

$$T_D \leq T_C : S_d(T) = \begin{cases} a_g . S . \eta . \frac{2.5}{q} . \left[\frac{T_C . T_D}{T^2}\right] \\ \geq \beta . a_g \end{cases} \tag{3.6}$$

The values of the period T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

Table 3-2 Values of the parameters describing the recommended Type 1 elastic response spectra [4]

Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

3.4.2 Non-linear static (pushover) analysis

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The analysis involves applying horizontal loads, in a prescribed pattern, to a computer model of the structure, incrementally; i.e.•- "pushing." the - structure; and plotting the total applied shear force and associated lateral displacement at each increment, until the structure reaches a limit state or collapse condition [22]. It can help demonstrate how progressive failure in buildings really occurs, and identify the mode of final failure [23].

Gravity loads were applied as initial conditions prior to the earthquake loadings. For present study, the total dead load and 48% of the live load (1.0 DL + 0.48 LL) were considered. The lateral (push) load was defined as acceleration load pattern in negative X-direction. Plastic hinges were assigned at the ends of beams and columns, in case of columns PMM hinges were provided while in case of beams M3 hinges were provided according to EC8, 2005 part 3 acceptance criteria which is integrated within ETABSv18.1.1 software.

Performance level of a structure

The structural and non- structural components of the buildings together comprise the building performance. The performance levels are the discrete damage states identified from a continuous spectrum of possible damage states.

Table 3-3 Performance Level of Building

Level	Description
Operational	Very light damage, no permanent drift, structure retains original strength and stiffness, all systems are normal
Immediate Occupancy	Light damage, no permanent drift, structure retains original strength and stiffness, elevator can berestarted, Fire protection operable
Life Safety	Moderate damage, some permanent drift, some residual strength and stiffness left in all stories, damage to partition, building may be beyond economical repair
Collapse Prevention	Severe damage, large displacement, little residualstiffness and strength but loading bearing column and wall function, building is near collapse

The performance levels (IO, LS, and CP) of a structural element are represented in the load versus deformation curve as shown in figure 3.2

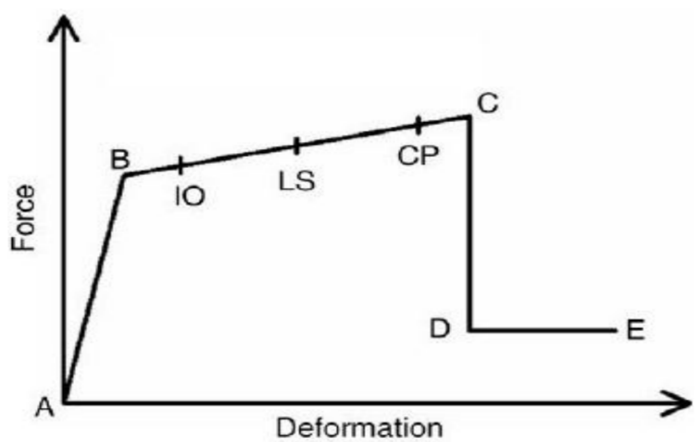


Figure 3.2 Force Vs deformation curve [24]

Point A corresponds to the unloaded condition. Load deformation relation shall be described by the linear response from A to an effective yield B. Then the stiffness reduces from point B to C. Point C has a resistance equal to the nominal strength then a sudden decrease in lateral load resistance to point D, the response at reduced resistance to E, final loss of resistance. The CD line corresponds to an initial failure of the member. The DE Line represents the residual strength of the member. These points are specified according to FEMA to determine hinge rotation behavior of RC members. The points between B and C represent acceptance criteria for the hinge, which is Immediate Occupancy (IO), LS (Life Safety), and CP (Collapse Prevention).

Capacity curve

Pushover curve (in the base shear vs roof displacement domain) transformed to capacity curve (in the modal acceleration vs modal displacement domain), by using the equations in ES-EN 1998-2015 Annex B .The procedures are integrated within ETABSv18.1.1, so that the capacity curve is obtained from the software based on the selected spectra type, ground acceleration, and behavior factor of the building.

3.5 Ground conditions and seismic action

3.5.1 Ground conditions

The earthquake vibration at the surface is strongly influenced by the underlying ground conditions and correspondingly the ground characteristics very much influence the seismic response of structures. In order to account for the influence of local ground conditions on the seismic action.ES EN 1998-1-2015 provides five ground profiles, denoted Ground types A, B, C, D, and E. For this study ground type B was selected.

3.5.2 Seismic Zones

The seismic action to be considered for design purposes should be based on the estimation of the ground motion expected at each location in the future, i.e. it should be based on the hazard assessment. For the purpose of ES EN 1998, national territories shall be subdivided into seismic zones, depending on the local hazard. For the Purpose of this study it is assumed that the building was located in a high seismic region, Zone 5 (Afar Semera which is seismic Zone 5)

Table 3-4 Bedrock Acceleration Ratio αo [4]

Zone	**5**	**4**	**3**	**2**	**1**	**0**
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

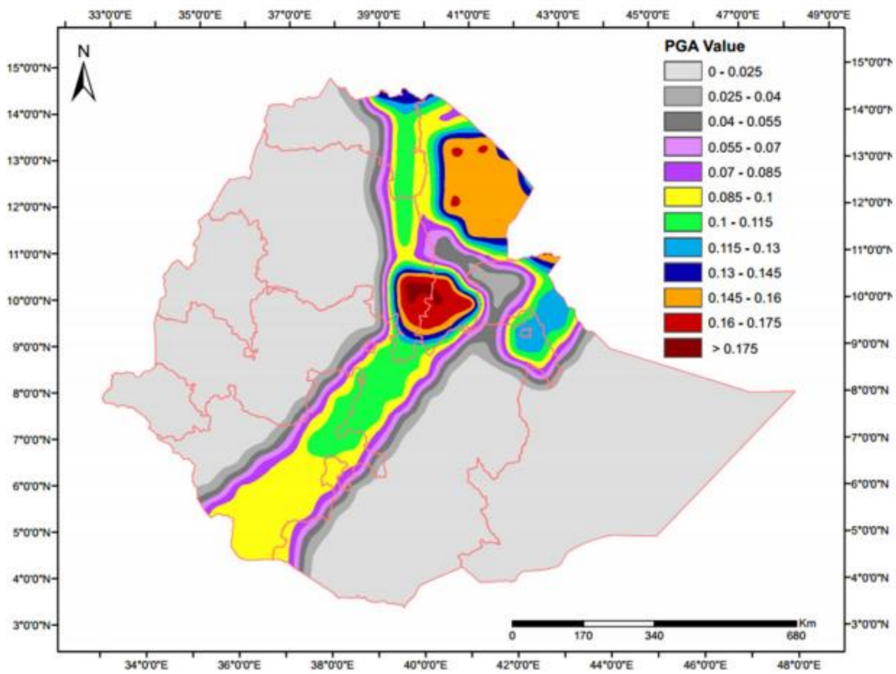


Figure 3.3 Ethiopia's Seismic hazard map in terms of peak ground acceleration [4]

3.5.3 Importance classes and importance factors

For the different importance classes, the design ground acceleration, a_g is equal to a_{gR} times the importance factor γ_I:

$$a_g = \cdot \gamma_I \cdot a_{gR} \tag{3.7}$$

Buildings are classified in 4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse.[4] For the purpose of this study first the building was designed as importance class II building by using importance factor 1, then it was changed to importance class III building and the ground acceleration was multiplied by importance factor (1.2)

3.6 Structural type of the building and behavior factor

Structural type and ductility class of the structure influences the value of the behaviour factor *q used* in the analysis.

The behaviour factor *q* for each horizontal direction is calculated by equation

$$q = q_0 \cdot k_w \tag{3.8}$$

Where q_0 is the basic value of the behaviour factor and k_w is the factor associated with the prevailing failure mode in structural system with walls.

For purposes of defining the value of behaviour coefficient is necessary classify the structural system and define their regularity in plan and height. If the structure is classified irregular in elevation, the reference values of behaviour coefficient must be reduced in 20%.

The modeled structure was considered as an open ground building (irregular in elevation), classified as a frame system and will be designed as a DCM (Ductility Class Medium) structure.

Basic value of the behaviour factor (qo), for Frame system, DCM,and vertically regular building taken as

$$q_0 = 3.0\alpha_u/\alpha_1 \tag{3.9}$$

The value of multiplication factor α_u/α_1 = 1.3, for multi storey multi bay frames (ES-EN 1998-1-2015, 5.2.2.2(5a)

The factor *k*w reflecting the prevailing failure mode in structural systems with walls shall be taken as follows: frames (ES EN 1998-1-2015, 5.2.2.2(11)

$$k_w = \left\{ \begin{array}{l} 1.00,\ \text{for frame and frame-equivalentdual systems} \\ (1+\alpha_0)/3 \le 1,\ \text{but not less than } 0.5,\ \text{for wall-equivalent and torsionally} \\ \ \text{flexible systems} \end{array} \right\}$$

For the selected building

q=0.8x3x1.3x1=3.12 (behavior factor)

3.7 Description of the Modelling

To have a better comparative view of the two retrofitting interventions, a G+3, G+6, and G+9 frames, with a plan dimension of 24mx20m were modeled. ETABSv18.1.1 was used for modeling and analysis, the design of concrete frames is seamlessly integrated within the program [25]. The models were first designed as category II building then changed to category III building by increasing the live load intensity on the building and by multiplying the ground acceleration by importance factor (γ_I).

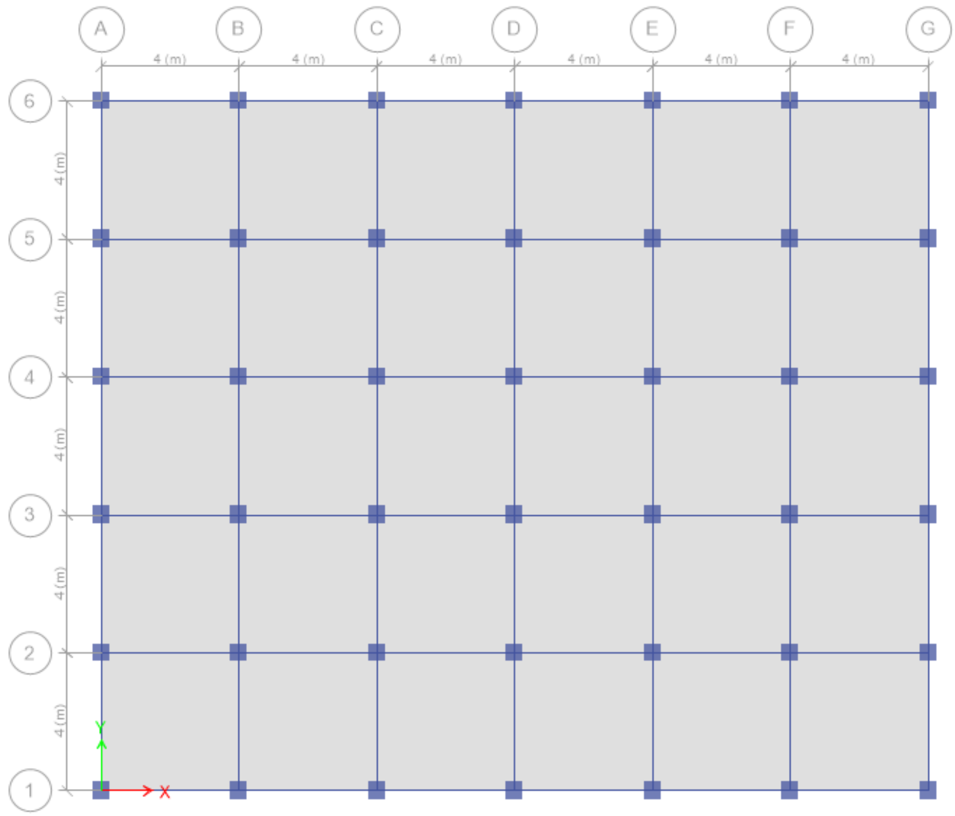


Figure 3.4 Building Plan layout

Table 3-5 Geometric properties of building

Building size	24m x 20m
Span length	4m both in X and Y direction
Storey height	3.0 m
Assumed footing depth	1.5m
Slab thickness (mm)	150
Beam size (mm)	300x400
Column size (mm) for 10 storey building	400 x 400 ,for 10th storey 500x500,for 6th-9th storey 600x600,for 2nd - 5th storey

	700x700,for footing column and 1st storey
Column size (mm) for 7 storey building	400x400,for 6th and 7th storey 500x500,for 1st-5th storey 600x600, footing column
Column size (mm) for 4 storey building	400x400,for 4th storey 500x500, for footing column –3rd storey
Bracing steel (mm)	Tube section 180x180x16
Size of HCB (mm)	200x200x400
tinf (mm)	200mm
Width of strut (α)	618mm

3.7.1 Infill wall Modelling

According to **FEMA 273** behavior of cracked concrete frames with masonry infills may be represented by a diagonally braced frame model in which the columns act as vertical chords, the beams act as horizontal ties, and the infill is modeled using the equivalent compression strut analogy.

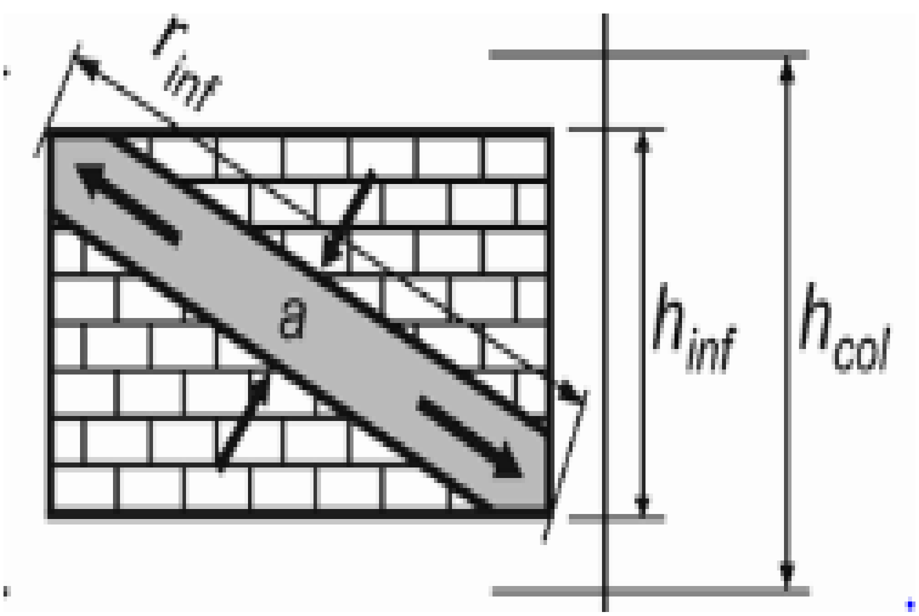


Figure 3.5 Compression Strut Analogy [5]

The equivalent strut shall have the same thickness and modulus of elasticity as the infill panel it represents [25].

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{3.10}$$

where

$$\lambda 1 = \left[\frac{Emetinfsin2\theta}{4EfeIcoltinfsin2\theta}\right]^{\frac{1}{4}} \tag{3.11}$$

h_{col} Column height between centerlines of beams, in

h_{inf} Height of infill panel, in

E_{fe} Expected modulus of elasticity of frame material, ksi

E_{me} Expected modulus of elasticity of infill material, ksi

I_{col} Moment of inertia of column, in^4

L_{inf} Length of infill panel, in

r_{inf} Diagonal length of infill panel, in

t_{inf} Thickness of infill panel and equivalent strut, in

For the modeled frame

h_{col}= 3m (118.02in)

h_{inf} = hcol-beam depth =(3-0.4)m=2.6m(102.362in)

E_{fe} = 33Gpa (4496.16ksi)

E_{me} = 2333.65Mpa (338.4673ksi)

I_{col} = 12513.07in^4

L_{inf} = (4-0.7) m= 3.3m (129.92in)

t_{inf} =200mm (7.874in)

r_{inf} = hinf /sinϴ

$\Theta=\tan^{-1} h_{inf}/L_{inf} = \tan^{-1}(102.362/129.92) = 38.23^\circ$

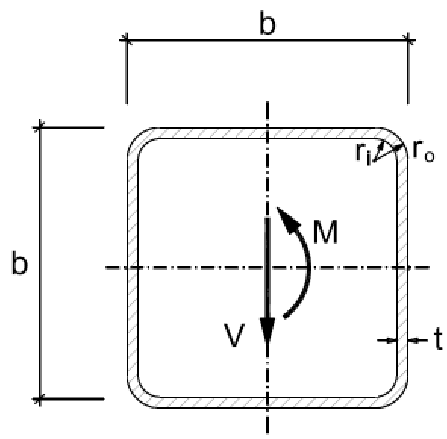
rinf= 118.02in/sin38.23°=191in

$$\lambda 1 = \left[\frac{338.4673x7.874sin2\Theta}{44496.16x12513.07x102.362sin2\Theta}\right]^{\frac{1}{4}} = 0.0186$$

$a = 0.175(\lambda\, h_{col})^{-0.4} r_{inf=0.175(0.0186x118.02)}{}^{-0.4} x191=24.4in\ (618mm)$

width of strurt=618mm, as recommended by FEMA273 thickness of strut should be equal with thickness of infill wall.

Steel section (180x180x16)mm was selected based on EN-1993, source (https://eurocodeapplied.com/design/en1993/shs-design-properties)



Notation for Square Hollow Sections (SHS) according to EN1993-1-1

Table 3-6 Gravity loadings on building

Live Load intensity for Residential	2 kN/m^2
Live Load intensity for Commercial	4 kN/m^2
Live load intensity for roof	1 kN/m^2
Finishing load on floors	1.5kN/m^2
Line Loads on beam(wall load)	For Internal wal1 =7.852 KN/m
	For External wall = 9.672 KN/m

Table 3-7 Seismic parameters and building category

Building type	Moment resisting RC frame (MRF)
Ductility class	DCM
Importance factor	1 for residential 1.2 for commercial
Seismic zone	V
Design ground acceleration (**a_g/g)**	0.2
Soil type	B
S	1.35
T_B	0.05
T_C	0.25
T_D	1.2
q (behavior factor)	3.12
Damping ratio	5%
β	0.2 (recommended value)
Mass source coefficients,	1 ,for dead/permanent loads 0.3, for roof live load 0.24 ,for residential live load 0.48, for commercial live load

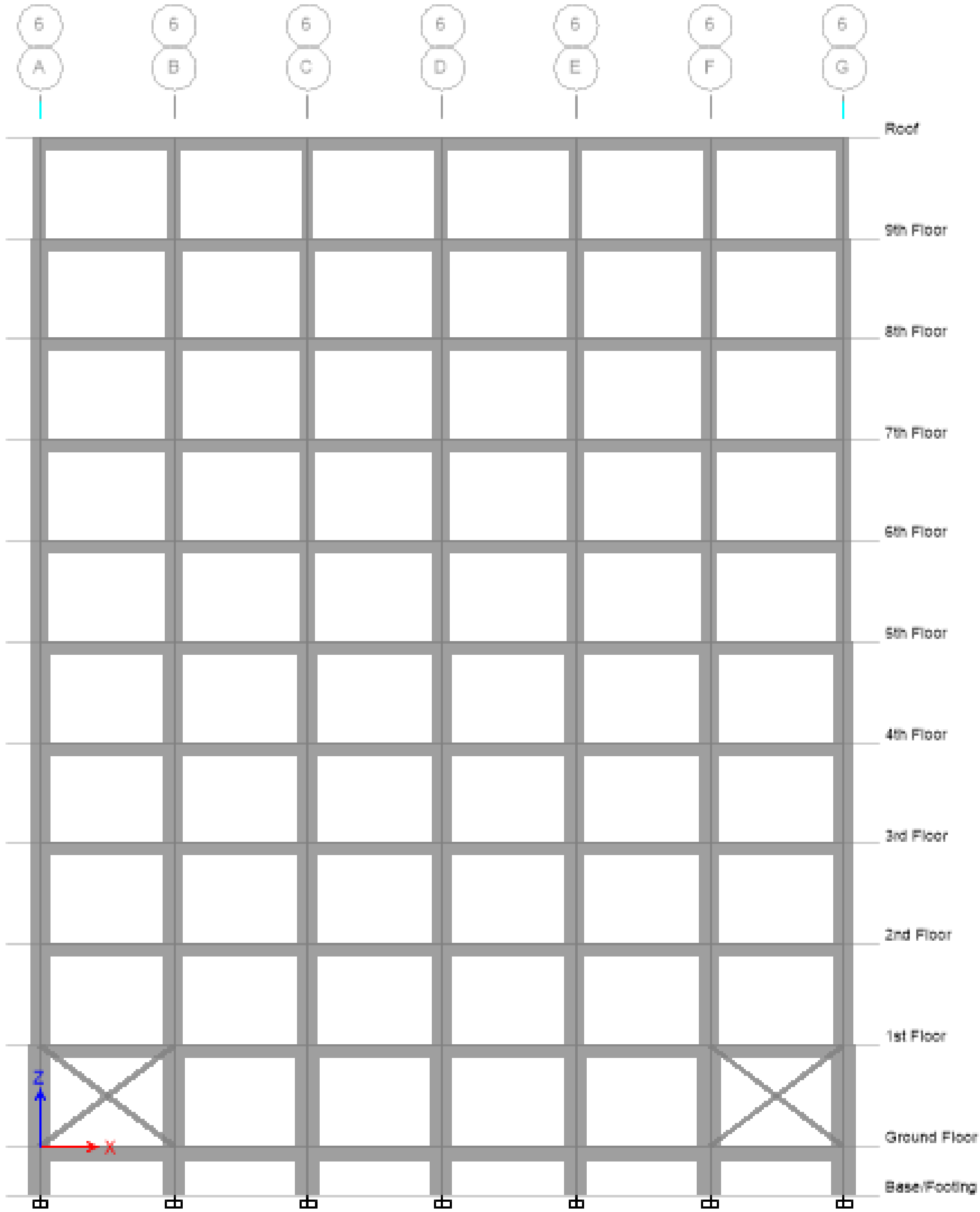


Figure 3.6 Steel bracing arrangement in the modeled G+9 frame

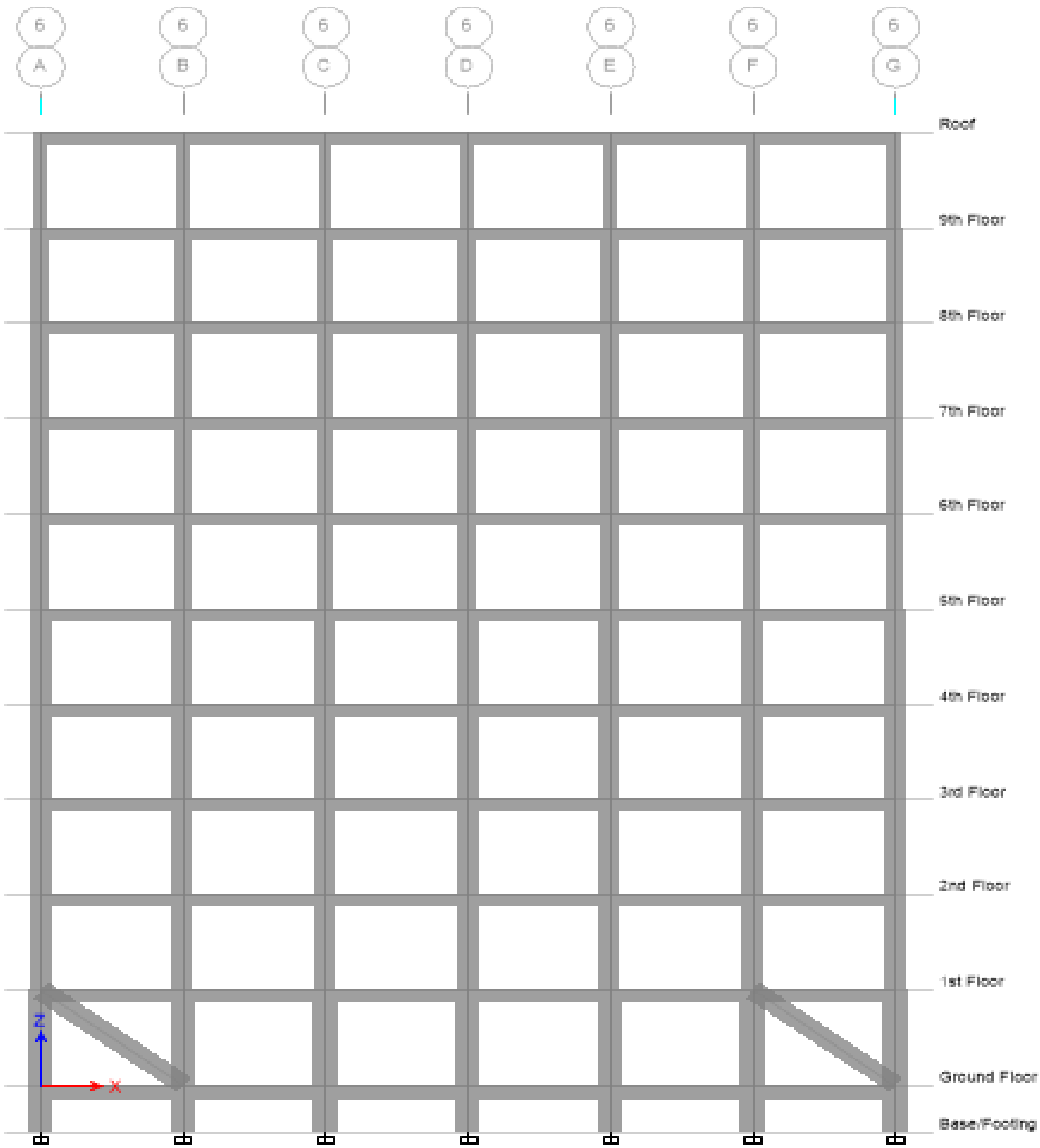
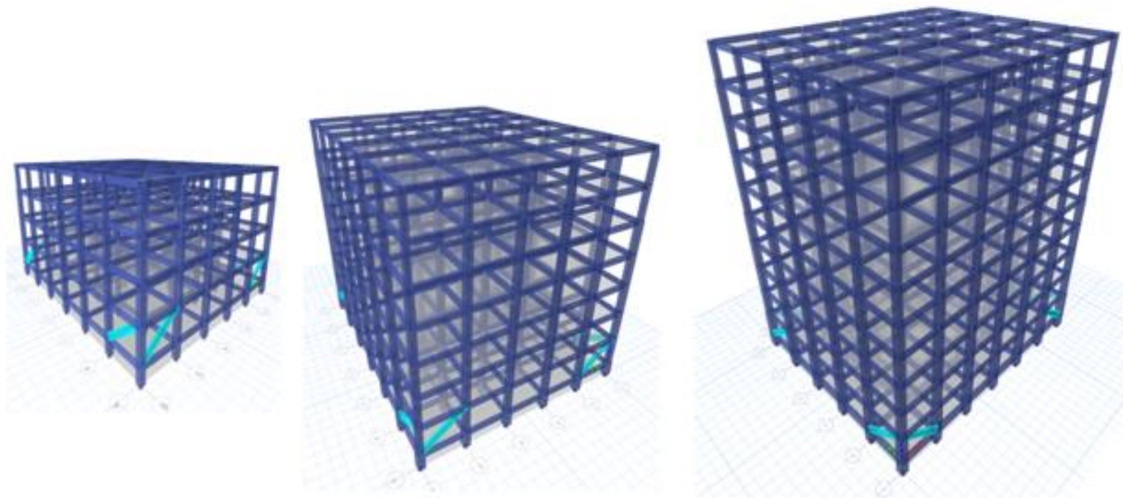
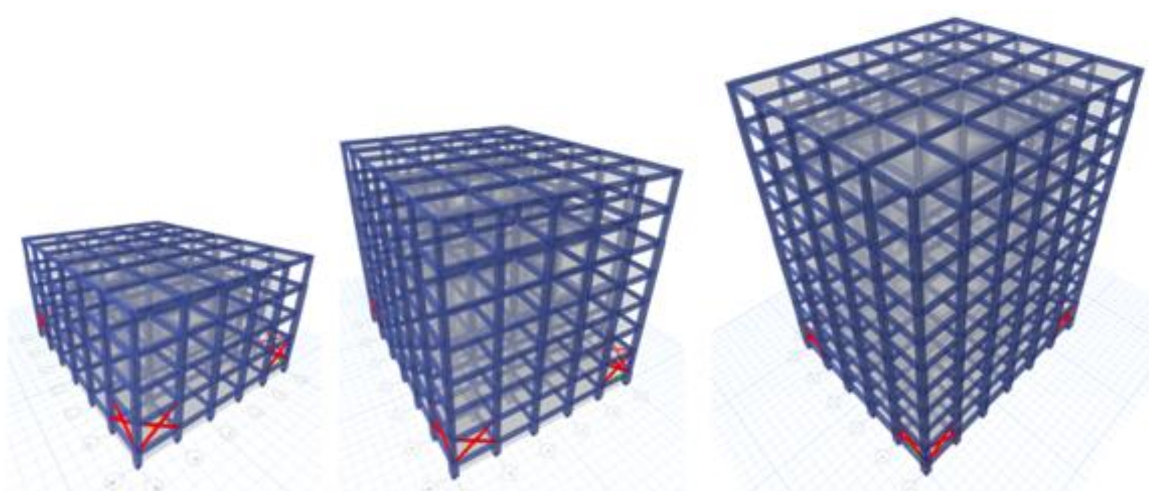


Figure 3.7 Infill wall arrangement in the modeled G+9 frame



(a) (b) (c)

Figure 3.8 3D view of infilled (a) 4, (b) 7, and (c) 10 storey buildings



(a) (b) (c)

Figure 3.9 3D view of braced (a) 4, (b) 7, and (c) 10 storey building

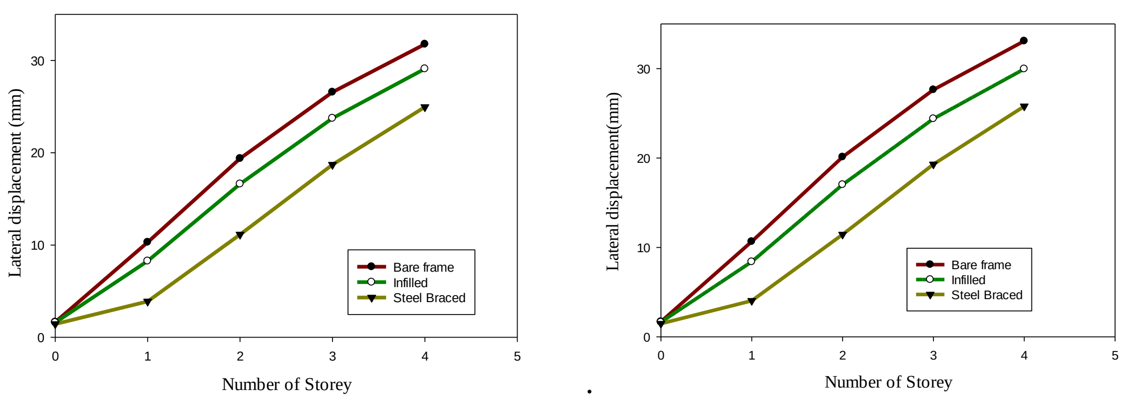
CHAPTER FOUR

RESULT AND DISCUSSION

The response of reinforced concrete buildings strengthened with steel bracing and infill wall have been evaluated. To study the effect of infill wall and steel bracing on seismic response of RC buildings, masonry infill wall and steel bracing were introduced at the corner bays of the ground floor separately. The building frames have been designed by using ES- EN 2015 as a category II building then it was changed to category III building. The design and analysis was done by using ETABSv18.1.1 software. Modal response spectrum analysis was performed to compare parameters like lateral storey displacement, story drift, storey stiffness base shear, and overturning moment. Pushover analysis was conducted to obtain the pushover curve of the bare and retrofitted building. The comparison of results obtained from the analyses are given in terms of tables & graphs in the coming paragraphs.

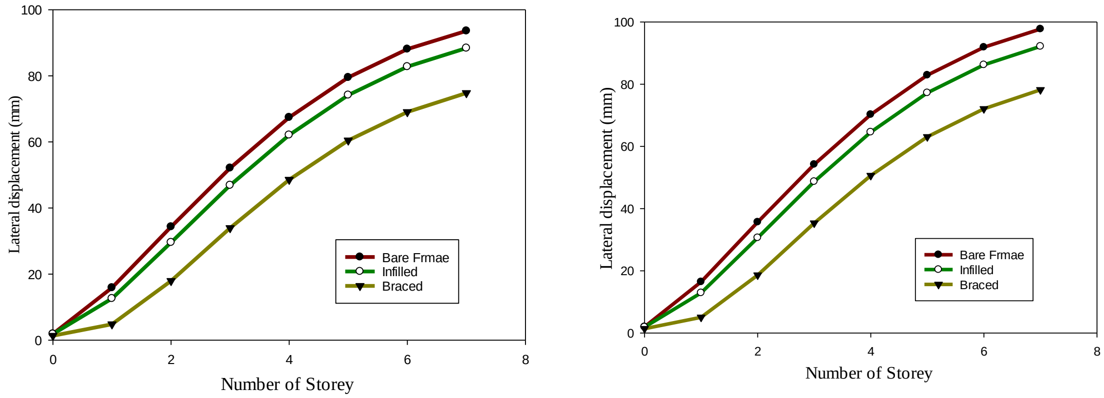
4.1 Lateral displacement comparison

The lateral displacement of the storey relative to the base of the building are plotted for the buildings modeled with and without the retrofit scheme.



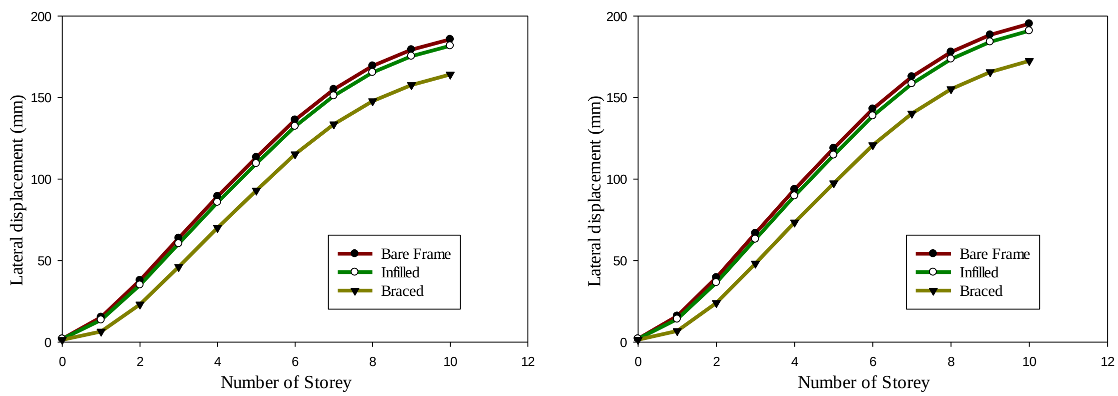
(a) (b)

Figure 4.1 Lateral displacement of 4 storey building in (a) X and (b) Y direction



(a) (b)

Figure 4.2 Lateral displacement of 7 storey building (a) in X, (b) in Y direction



(a) (b)

Figure 4.3 Lateral displacement of 10 storey building (a) in X, (b) in Y direction

From Figure 4.1 to Figure 4.3, it is observed that the lateral displacement is reduced for retrofitted buildings. As tabulated in Table 4-1, top roof displacement of G+3, G+6 and G+9 for the infilled frame reduced by 8%, 6%, and 2% respectively. For the braced frame the reduction percentage is higher than the infilled frame, the displacement reduced by 21%, 20% and 12% respectively. The study conducted by providing infill wall in RC buildings reduces the top floor displacement by 10% [20], also the researchers concluded that, retrofitting of the building by adding steel bracing very effective for reducing the displacement up to 60% [16].Based on the result of this study and previous studies, adding both infill wall and steel bracing to RC frames reduces the storey displacement, the reduction percentage is higher for steel bracing.

Table 4-1 Top storey displacement comparison of buildings with and without retrofitting

Building	**Bare Frame**		**Infilled**				**Braced**			
	Displacement (mm)		Displacement (mm)		% reduced		Displacement (mm)		% reduced	
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
G+3	31.771	31.771	29.103	29.103	8%	8%	24.969	24.969	21%	21%
G+6	93.62	97.747	88.402	92.195	6%	6%	74.828	78.222	20%	20%
G+9	185.766	195.305	181.799	191.006	2%	2%	164.192	172.546	12%	12%

4.2 Storey drift Comparison

Storey drift is the relative displacement of one storey relative to the other. The storey drift is very important parameter in the analysis and design of buildings.

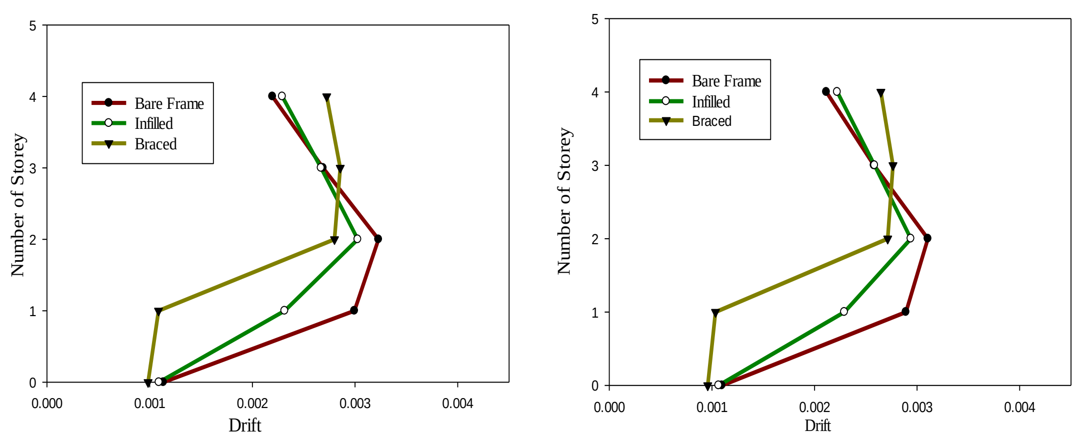
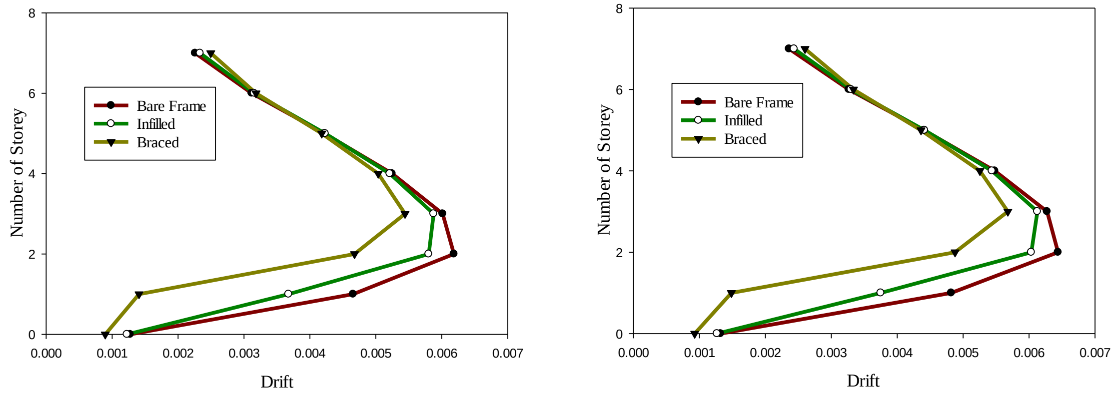
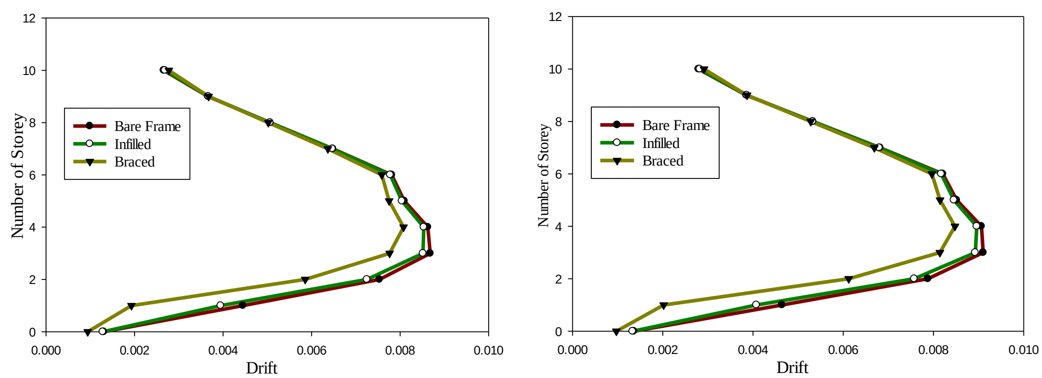


Figure 4.4 Storey drift of 4 storey building (a) in X, (b) in Y direction



(a) (b)

Figure 4.5 Storey drift of 7 storey building (a) in X, (b) in Y direction



(a) (b)

Figure 4.6 Storey drift of 10 storey building (a) in X, (b) in Y direction

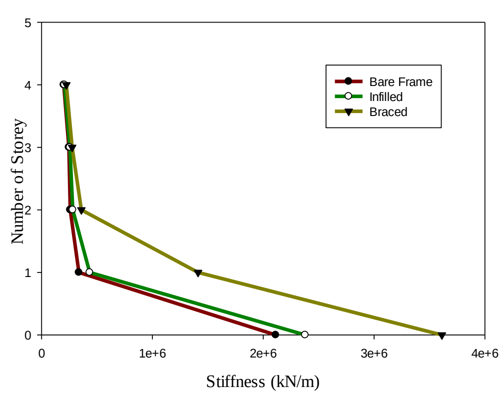
From Figure 4.4 to Figure 4.6, it can be seen that storey drift in retrofitted frames in both X and Y direction are reduced in comparison with the bare frames. Although the drift for retrofitted frames increases in the upper stories, the storey drift around the intermediate level of the building is more critical than that at the top. For G+3 building the maximum drift at the critical storey (2nd storey) reduced by 6% for infilled and by 13% for braced frame. For G+6 building the maximum drift at the critical storey (2nd storey) reduced by 6% for infilled and by 24% for braced frame.For G+9 building the maximum drift at the critical storey (3rd storey) reduced by 2 % for infilled and by 11% for braced frame. Previous researches also proves that, the intervention of steel bracing found to be more effective in reducing the storey drift to lower values compared to jacketing and adding of shear wall techniques. The maximum drift was reduced by almost 5–10 times of the bare frame [26]. In this study the percentage reduction pattern is not similar, other researchers also observes that the pattern of reduction in storey drifts are differ in different frame [27].

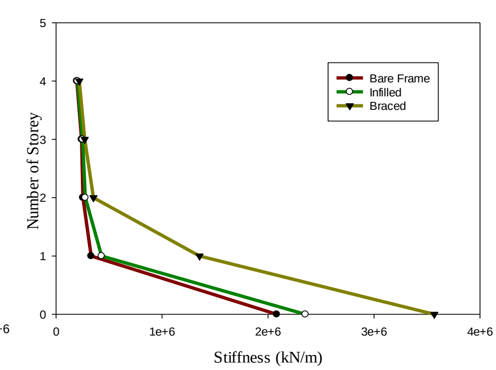
Table 4-2 storey drift comparison of buildings with and without retrofitting

Building	**Bare Frame**		**Infilled**				**Braced**			
	Maximum drift		Maximum drift		%reduced		Maximum drift		% reduced	
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
G+3 At 2^{nd} floor	0.003105	0.003229	0.00294	0.003 027	5%	6%	0.0027 14	0.0028	13%	13%
G+6 At 2^{nd} floor	0.006185	0.00644	0.005802	0.006 034	6%	6%	0.0046 77	0.004878	24%	24%
G+9 At 3^{rd} floor	0.00868	0.009103	0.008518	0.008 929	2%	2%	0.0077 62	0.008143	11%	11%

4.3 Storey stiffness comparison

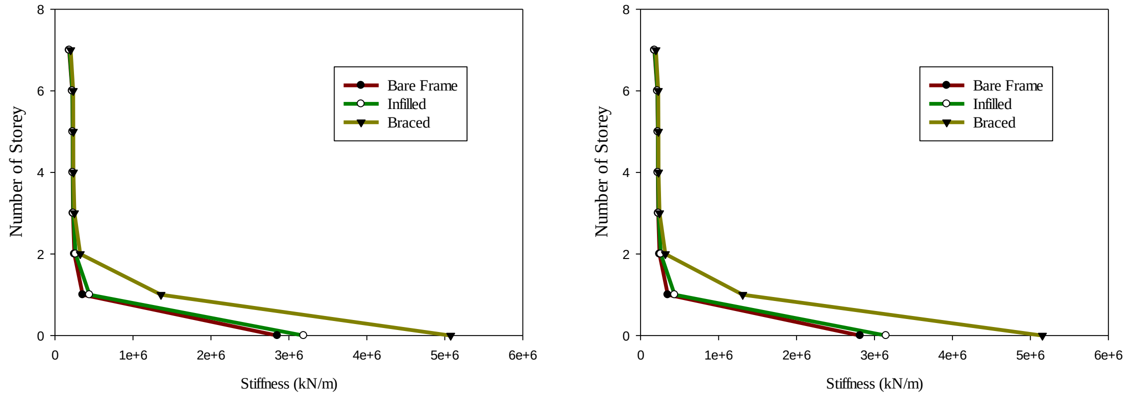
Stiffness of the structure is one of the fundamental parameter governing the structural response of the building when subjected to seismic actions.





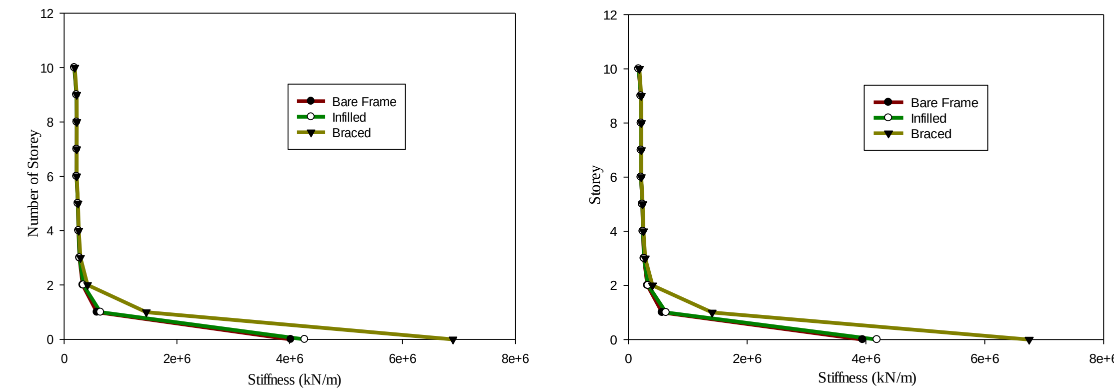
(a) (b)

Figure 4.7 Storey stiffness of 4 storey building (a) in X, (b) in Y direction



(a) (b)

Figure 4.8 Storey stiffness of 7 storey building (a) in X, (b) in Y direction



(a) (b)

Figure 4.9 Storey stiffness of 10 storey building (a) in X, (b) in Y direction

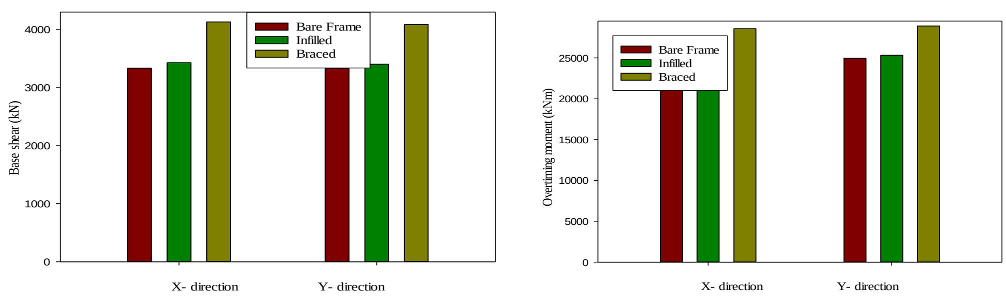
From Figure 4.7 to Figure 4.9, it can be seen that the stiffness of retrofitted building is higher than the bare frame at the lower stories. As shown in Table 4-3, for all G+3,G+6 and G+9 buildings the average building stiffness increased by 13%, 10% and 5 % for infilled and increased by 64%, 58 % and 57% for braced frames respectively. Other researchers also shows that the retrofit measure improves the global building stiffness 4-7 times[26].

Table 4-3 Average building stiffness comparison of buildings with and without retrofitting

Building	Bare Frame		Infilled				Braced			
	Stiffness (kN/m))		Stiffness (kN/m)		%increased		Stiffness (kN/m)		% increased	
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
G+3	630692.8	620599.4	710765.8	701658.2	13%	13%	1176214	1152707	68%	64%
G+6	566993	557708.1	624286.7	614514.2	10%	10%	988841.2	988143.5	74%	58%
G+9	613550.6	600297.2	643930.5	630452.2	5%	5%	968438	945165.2	58%	57%

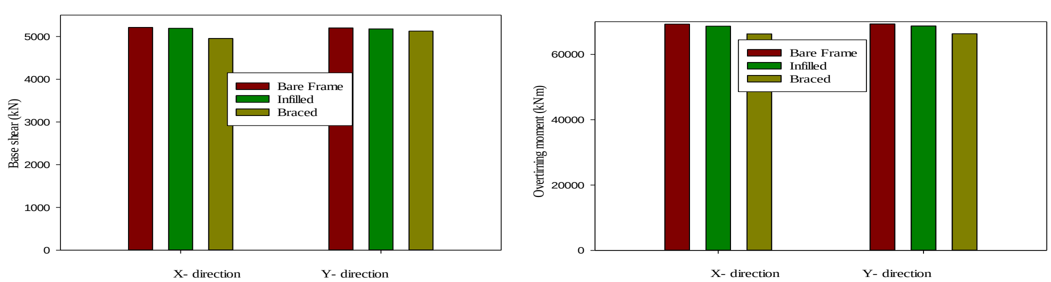
4.4 Base Shear and Base Overturning moment comparison

Base shear varies from structure to structure and depends on stiffness and weight of the structure and also on lateral force applied to the structure.



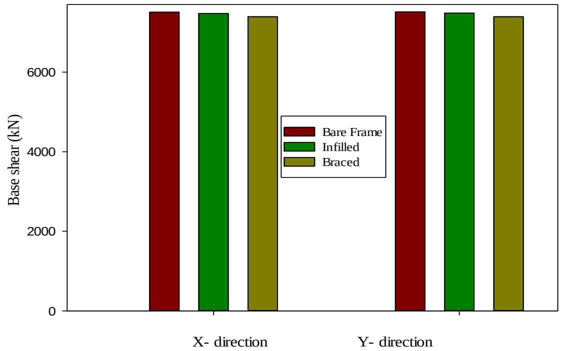
(a) (b)

Figure 4.10 (a) Base shear graph,(b) Base Overturning moment graph of 4 storey building

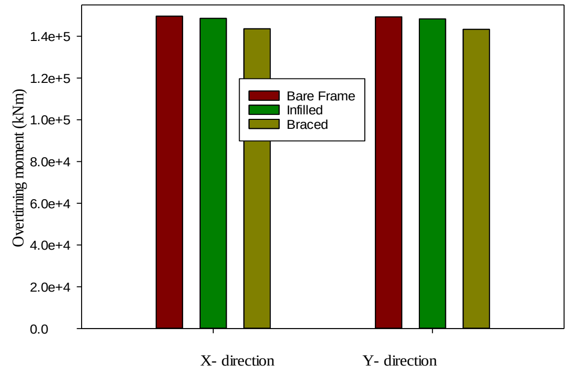


(a) (b)

Figure 4.11 (a) Base shear graph, (b) Base Overturning moment graph of 7 storey building



(a)



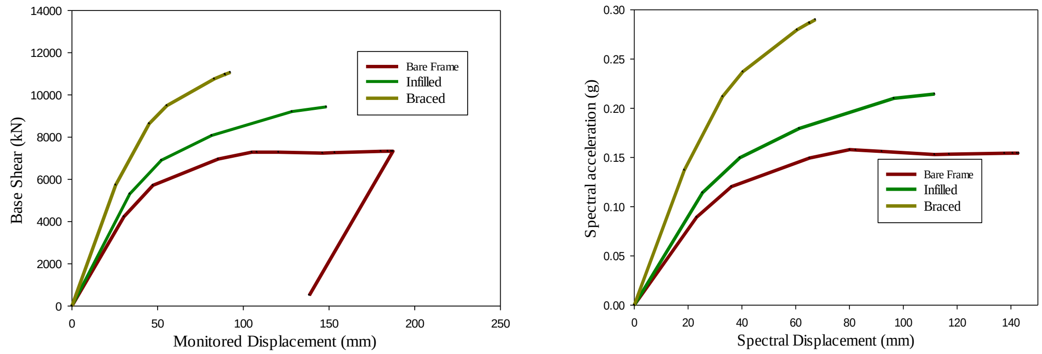
(b)

Figure 4.12 (a) Base shear graph, (b) Base Overturning moment graph of 10 storey building

From figure 4.10 it can be observed that for the four storey building the base shear in infilled frame increased by 3%, and increases by 24% for X bracing system compared to bare frame due to the addition of weight in the building and results the increasing of overturning moment in the building. For the seven and ten storey buildings the addition of infill wall and steel bracing does not result significant change in base shear and overturning moment. Researchers also shows that, the rate of increase in base shear due to the addition strengthening interventions is reduced as the height of frame increases [27].

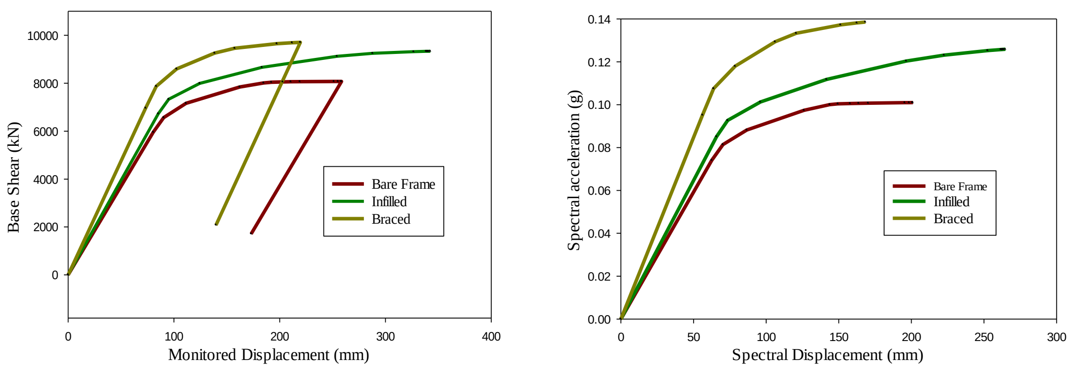
4.5 Pushover and capacity curve

Pushover analysis has been conducted for the 9 building models, the material nonlinearities are assigned as hinges; M3 flexural hinges for beams and PM2M3 flexural hinges for columns. Carrying out the pushover analysis on typical structure gives a curve having seismic base shear and monitored roof displacement at various performance levels. Seismic base shear is an estimate of the maximum expected lateral forces that will occur due to seismic ground motion at the base of a structure. The roof displacement is the measured top floor displacement of the structures subjected to the incremental load (push load).



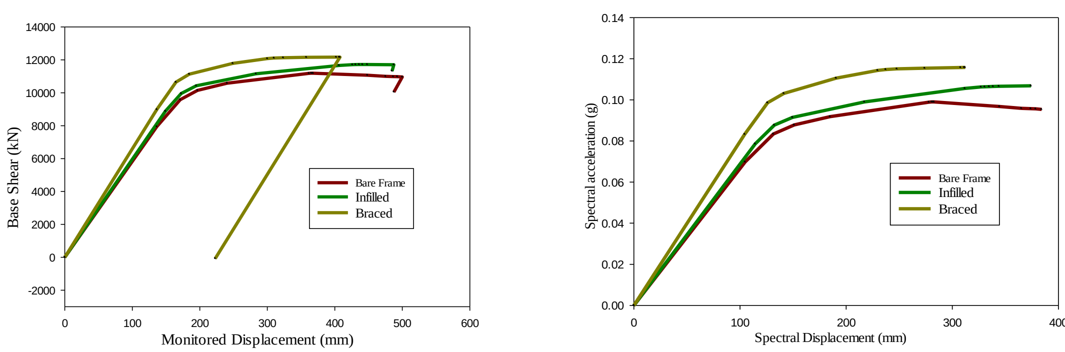
(a) (b)

Figure 4.13 Comparative (a) Pushover, (b) capacity curve of G+3 building



(a) (b)

Figure 4.14 Comparative (a) Pushover, (b) capacity curve of G+6 building



(a) (b)

Figure 4.15 Comparative (a) Pushover, (b) capacity curve of G+9 building

Figure 4.13 to Figure 4.15 presents the pushover (Base shear vs roof displacement) and capacity (spectral acceleration vs spectral displacement) curves for all the modeled buildings with and without retrofit measures. The plots demonstrates that there is a significant increase in lateral load capacity of the building after introducing retrofit measures. As it is observed in the curves, the lateral load carrying capacity of braced frame is more than that of infilled frame. From the analysis result, tabulated through Table 4-4, the bare frame reaches its maximum load carrying capacity when the lateral load is 10963.46 kN, From Table 4-5 and Table 4-6 the lateral load carrying capacity is increased by 6.76% for infilled frame, and by 21.09% for braced frame. Previous studys done for the six storey building with the 140 Tube section X bracing also proves that the capacity of RC frames can be greatly enhanced through the addition of steel braces [28].

4.6 Plastic Hinge Formation

Hinges are points on a structure where one expects cracking and yielding to occur in relatively higher intensity so that they show high flexural (or shear) displacement, as it approaches its ultimate strength under cyclic loading. These are locations where one expects to see cross diagonal cracks in an actual building structure after a seismic action.

Table 4-4: Plastic hinge pattern of G+9 bare model

Step	**Monitored Displacement**	**Base Force**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total**
	mm	kN					
0	0	0	2486	0	0	0	2486
1	136.274	7947.38	2486	0	0	0	2486
2	170.955	9581.14	2486	0	0	0	2486
3	196.517	10143.97	2486	0	0	0	2486
4	240.406	10584.60	2449	36	0	1	2486
5	361.77	11179.87	2149	336	0	1	2486
6	365.092	11189.77	2149	336	0	1	2486
7	366.93	11192.21	2149	336	0	1	2486
8	368.005	11191.65	2148	337	0	1	2486
9	448.228	11066.95	2065	420	0	1	2486
10	475.166	11002.98	2009	452	14	11	2486
11	486.581	10987.93	1980	475	16	15	2486
12	492.658	10982.65	1968	487	13	18	2486

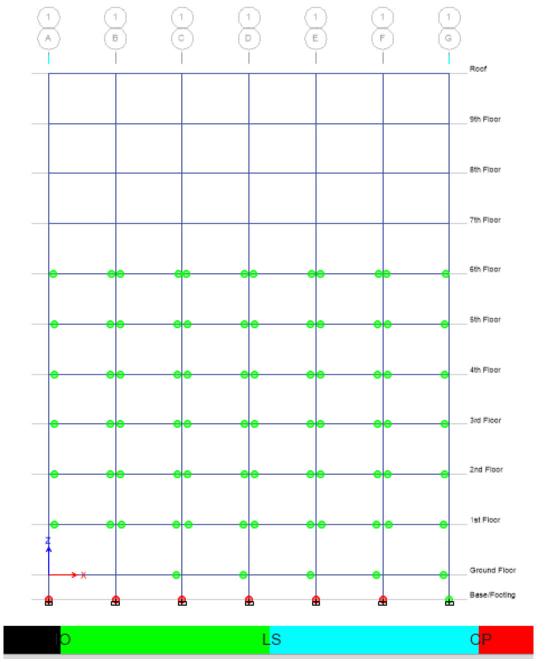
13	499.946	10963.46	1949	504	14	19	2486
14	487.668	10047.21	1949	498	16	23	2486

Table 4-5 Plastic hinge pattern of G+9 infilled model

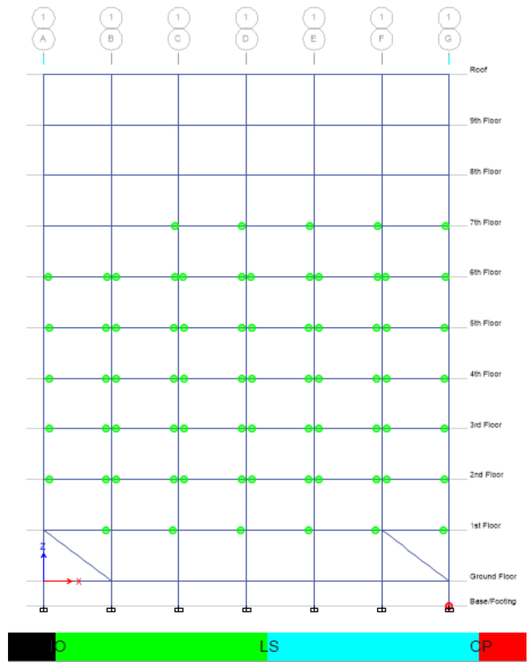
Step	**Monitored Displacement**	**Base Force**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total**
	mm	kN					
0	0	0	2486	0	0	0	2486
1	148.928	8883.67	2486	0	0	0	2486
2	172.326	9957.33	2486	0	0	0	2486
3	194.707	10425.62	2486	0	0	0	2486
4	283.087	11157.79	2358	128	0	0	2486
5	406.321	11662.36	2136	348	0	2	2486
6	426.237	11708.18	2096	384	0	6	2486
7	430.805	11713.56	2087	393	0	6	2486
8	436.158	11717.57	2067	412	0	7	2486
9	441.065	11719.67	2061	416	0	9	2486
10	448.129	11718.44	2059	418	0	9	2486
11	487.516	11704.60	2042	433	1	10	2486
12	484.86	11320.51	2041	429	0	16	2486

Table 4-6 Plastic hinge pattern of G+9 braced model

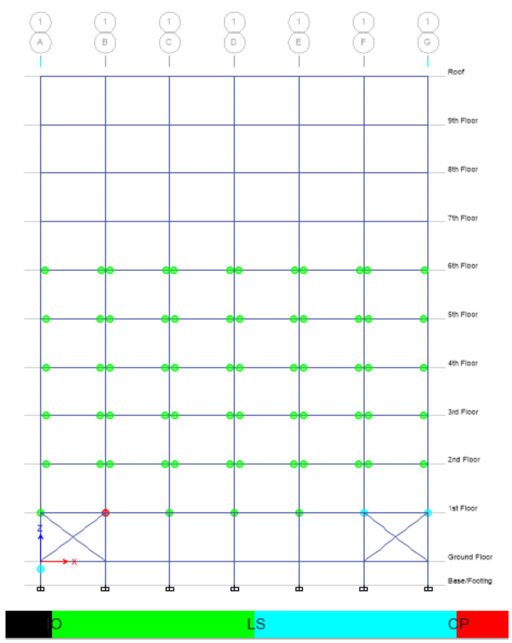
Step	**Monitored Displacement**	**Base Force**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total**
	mm	kN					
0	0	0	2486	0	0	0	2486
1	136.264	8983.92	2486	0	0	0	2486
2	164.534	10647.21	2486	0	0	0	2486
3	184.424	11131.34	2486	0	0	0	2486
4	248.808	11789.18	2417	68	0	1	2486
5	299.998	12079.78	2312	173	0	1	2486
6	309.98	12112.72	2271	214	0	1	2486
7	323.709	12134.85	2217	268	0	1	2486
8	357.942	12154.59	2163	321	0	2	2486
9	402.137	12165.77	2060	399	23	4	2486
10	406.355	12166.38	2060	397	25	4	2486
11	407.706	12166.50	2060	395	25	6	2486
12	222.759	-79.05	2060	395	23	8	2486



(a)



(b)



(c)

Figure 4.16 Plastic hinge distribution at the location of strengthening methods adopted (a) bare, (b) infilled, (c) braced

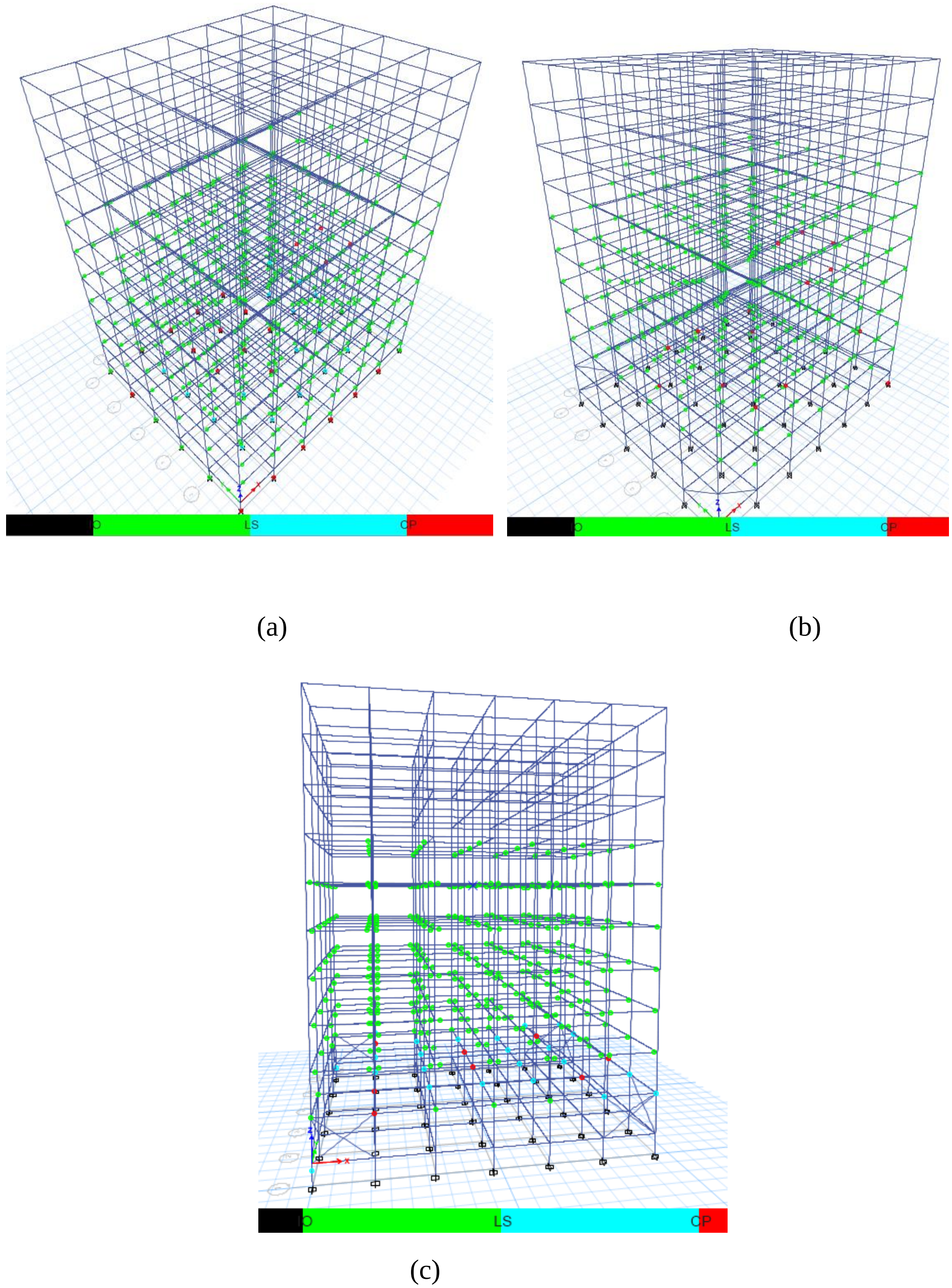


Figure 4.17 Plastic hinge distribution of G+9 building in 3D view for (a) bare, (b) infilled and (c) braced frames

Pushover analysis can identify weak elements by predicting the failure mechanism and account for the redistribution of forces during progressive yielding. With the increase in the magnitude of the lateral loads plastic hinge formation of the building are found. During seismic shakings, the formation of hinges starts accordingly from Immediate Occupancy (IO) level then rise up to Life Safety (LS), Collapse Prevention (CP) and finally Collapse (C), leading to catastrophe.

The effectiveness adding of steel bracing and infill wall to improve seismic performance can be quantified from the number of plastic hinges occurred due to nonlinear static pushover analysis. On observing hinge formation patterns as shown in Figure 4-13 to Figure 4-14, infill wall and steel bracing changes the order of plastic hinge formation mechanism of building, for the braced frame no plastic hinge is observed in the column, for the infilled frame the plastic hinge occurred in the column is less, while for the bare frame, the columns at the lower storey develops plastic hinges. (Figure 4.13).

Table 4-4 to 4-7 presents the failure mechanism of the modeled buildings in terms of the plastic hinges. As shown in tables in bare frame hinges starts yielding at the base shear of 10584.6KN, in infilled frame at 11157.79kN, in braced frame at 11789.18kN, this indicates that masonry infill wall and steel bracing (X-bracing) reduces the number of plastic hinge formation in the building. The strengthening methods prevents the building from catastrophic failure by preventing the formation CP hinges at the columns (as indicated in Figure 4.16).Based on the results discussed above, both methods decreases the seismic vulnerability, by reducing the hinge formation to a lower level ,thereby preventing collapse and drastic damages. Compared to masonry infill, steel bracing (X-bracing) reduces the damage to a greater extent.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusions

In the present work effects of infill wall and steel bracing in seismic response of RC buildings has been examined, via numerical simulations by using ETABSv18.1.1 software.

The various conclusions obtained are summarized below.

- Retrofitting the RC frame by steel X-bracing and HCB masonry infill wall, reduced the top floor displacement in all three model frames, but in varying degrees, the most reductions were observed in braced frames for all models. The reduction percentage was by, 21%,20%,12% for braced frame, and by 8%, 6%, and 2% for infilled frame for G+3, G+6, and G+9 buildings.
- Adding steel bracing (X-bracing) and masonry infill wall to the bare frame decreased the storey drift and increased stiffness capacity of the building. From the adopted intervention methods steel bracing is more effective.
- The retrofitting methods increases the lateral load carrying capacity of the building, the lateral load carrying capacity is increased by 6.76% for infilled frame, and by 21.09%for braced frame.
- Both retrofitting interventions change the pattern and order of plastic hinge formation in the building, by preventing plastic hinges from developing in the columns at lower stories, thus can reduce collapse and drastic damages.
- The results of this study shows that both X- steel bracing and masonry infill wall can improve the seismic resistance of RC buildings compared to bare frame.
- From the comparison made for the steel bracing and masonry infill wall retrofitting techniques based on the analyses results, steel bracing is more effective than masonry infill wall. Both linear dynamic analysis and nonlinear static pushover analysis based on ES EN 1998-1-2015 and EN 1998-3 confirm this.

5.2 Recommendations for Future Study

In this thesis, infill wall and steel bracing retrofitting techniques are compared. This gives rise for future investigations and improvements in seismic retrofitting techniques and seismic damage control; which are of high importance since, they have an extensive contribution to the present state of development. The scope and recommendations future researchers are:

- Here only the global response of structure is considered. But, it constitutes a reasonable base for further investigation of local analysis of the structure, the local effect of adding bracing on the joint, beams and individual columns they attached and the interaction of existing columns and infill wall should be studied.
- In this study the retrofitting strategies are compared based on the response of structures, further research should be conducted to improve the selection of appropriate retrofit techniques using criteria based on economy and constructability.
- In the future new innovative methods can be used, by using a combination of seismic retrofitting techniques to optimize the functionality of the structure under seismic hazards.

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APPENDICES

Appendix "A"“Sample analysis result outputs of structure in Tabular form”

Table A- 1 Lateral displacement of 4 storey building in X and Y direction

Maximum displacement								
			Bare Frame		**Infilled**		**Braced**	
Storey	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		mm	mm	mm	mm	mm	mm
Roof	12	Top	31.771	33.09	29.103	29.978	24.969	25.797
3rd Floor	9	Top	26.578	27.647	23.74	24.406	18.714	19.305
2nd Floor	6	Top	19.387	20.125	16.623	17.033	11.137	11.448
1st Floor	3	Top	10.293	10.656	8.262	8.393	3.887	4.034
Ground Floor	0	Top	1.647	1.7	1.598	1.635	1.438	1.476
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 2 Lateral displacement of 7 storey building in X and Y direction

Maximum displacement								
			Bare Frame		**Infilled**		**Braced**	
Storey	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		mm	mm	mm	mm	mm	mm
Roof	21	Top	93.62	97.747	88.402	92.195	74.828	78.222
6th Floor	18	Top	88.101	91.888	82.799	86.247	69.075	72.112
5th Floor	15	Top	79.536	82.927	74.187	77.239	60.462	63.096
4th Floor	12	Top	67.448	70.276	62.109	64.601	48.572	50.653
3rd Floor	9	Top	52.099	54.223	46.899	48.704	33.921	35.34
2nd Floor	6	Top	34.306	35.642	29.558	30.602	17.929	18.623
1st Floor	3	Top	15.864	16.43	12.595	12.909	4.807	5.043
Ground Floor	0	Top	1.911	1.979	1.839	1.899	1.342	1.392
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 3 Lateral displacement of 10 storey building in X and Y direction

Maximum displacement								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		mm	mm	mm	mm	mm	mm
Roof	30	Top	185.766	195.305	181.799	191.006	164.192	172.546
9th Floor	27	Top	179.375	188.502	175.388	184.183	157.744	165.682
8th Floor	24	Top	169.429	177.972	165.44	173.653	147.85	155.21
7th Floor	21	Top	155.078	162.83	151.107	158.534	133.684	140.27
6th Floor	18	Top	136.298	143.051	132.378	138.812	115.286	120.903
5th Floor	15	Top	113.363	118.918	109.544	114.794	93.037	97.509
4th Floor	12	Top	89.393	93.684	85.741	89.746	70.103	73.389
3rd Floor	9	Top	63.732	66.708	60.369	63.088	46.118	48.206
2nd Floor	6	Top	37.85	39.554	35.009	36.489	23.111	24.032
1st Floor	3	Top	15.292	15.962	13.58	14.068	6.506	6.812
Ground Floor	0	Top	1.954	2.039	1.921	1.997	1.413	1.457
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 4 Storey drift of 4 storey building in X and Y direction

Storey drift								
			Bare Frame		Infilled		Braced	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m							
Roof	12	Top	0.002115	0.002196	0.002222	0.002292	0.002647	0.002725
3rd Floor	9	Top	0.002581	0.002689	0.002586	0.00267	0.002765	0.002856
2nd Floor	6	Top	0.003105	0.003229	0.00294	0.003027	0.002714	0.0028
1st Floor	3	Top	0.002892	0.002995	0.002292	0.002315	0.001035	0.001086
Ground Floor	0	Top	0.001098	0.001133	0.001065	0.00109	0.000959	0.000984
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 5 Storey drift of 7 storey building in X and Y direction

Storey drift								
			Bare Frame		Infilled		Braced	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m							
Roof	21	Top	0.002259	0.00236	0.002333	0.002435	0.002495	0.0026
6th Floor	18	Top	0.003117	0.003266	0.003153	0.003298	0.003182	0.003336
5th Floor	15	Top	0.004222	0.004408	0.004231	0.004415	0.004179	0.004361
4th Floor	12	Top	0.005248	0.005481	0.005211	0.005438	0.005037	0.005255
3rd Floor	9	Top	0.006013	0.006274	0.005876	0.006127	0.005442	0.005679
2nd Floor	6	Top	0.006185	0.00644	0.005802	0.006034	0.004677	0.004878
1st Floor	3	Top	0.004656	0.004821	0.003677	0.003751	0.001408	0.001485
Ground Floor	0	Top	0.001274	0.00132	0.001226	0.001266	0.000895	0.000928
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 6 Storey drift of 10 storey building in X and Y direction

Storey drift								
			Bare Frame		Infilled		Braced	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m							
Roof	30	Top	0.002658	0.002793	0.002686	0.002822	0.002776	0.002913
9th Floor	27	Top	0.003659	0.003854	0.003668	0.003863	0.003674	0.003867
8th Floor	24	Top	0.005065	0.005328	0.005065	0.005327	0.005022	0.00528
7th Floor	21	Top	0.006483	0.006816	0.00647	0.0068	0.006368	0.006691
6th Floor	18	Top	0.007811	0.008208	0.007779	0.008172	0.007587	0.007967
5th Floor	15	Top	0.008094	0.008515	0.00804	0.008455	0.007753	0.008147
4th Floor	12	Top	0.008627	0.009065	0.008535	0.008962	0.008079	0.008476
3rd Floor	9	Top	0.00868	0.009103	0.008518	0.008929	0.007762	0.008143

2nd Floor	6	Top	0.007535	0.007879	0.00725	0.007575	0.005857	0.006124
1st Floor	3	Top	0.004449	0.004644	0.003946	0.004076	0.001929	0.002019
Ground Floor	0	Top	0.001303	0.001359	0.001281	0.001332	0.000942	0.000971
Base/Footing	-1.5	Top	0	0	0	0	0	0

Table A- 7 Storey Shear of 4 storey building in X and Y direction

Storey Shear								
			Bare Frame		Infilled		Braced	
Storey	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN	kN	kN	kN	kN	kN
Roof	12	Top	1213.8	1209.59	1312.37	1303.82	1694.89	1679.44
		Bottom	1213.8	1209.59	1312.37	1303.82	1694.89	1679.44
3rd Floor	9	Top	1820.81	1817.21	1886.91	1869.34	2181.08	2157.12
		Bottom	1820.81	1817.21	1886.91	1869.34	2181.08	2157.12
2nd Floor	6	Top	2300.82	2299.69	2345.65	2322.42	2709.89	2679.84
		Bottom	2300.82	2299.69	2345.65	2322.42	2709.89	2679.84
1st Floor	3	Top	2792.2	2788.43	2894.91	2867.27	3584.76	3541.46
		Bottom	2792.2	2788.43	2894.91	2867.27	3584.76	3541.46
Ground Floor	0	Top	3333.51	3327.16	3428.8	3402.44	4132.14	4088.38
		Bottom	3333.51	3327.16	3428.8	3402.44	4132.14	4088.38
Base/Footing	1.5	Top	0	0	0	0	0	0
		Bottom	0	0	0	0	0	0

Table A- 8 Storey Shear of 7 storey building in X and Y direction

Storey Shear								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN	kN	kN	kN	kN	kN
Roof	21	Top	1174.618	1170.793	1241.177	1237.817	1439.14	1433.626
		Bottom	1174.618	1170.793	1241.177	1237.817	1439.14	1433.626

6th Floor	18	Top	1969.322	1963.979	2011.322	2005.821	2123.838	2115.835
		Bottom	1969.322	1963.979	2011.322	2005.821	2123.838	2115.835
5th Floor	15	Top	2739.774	2735.148	2764.118	2759.779	2807.744	2802.955
		Bottom	2739.774	2735.148	2764.118	2759.779	2807.744	2802.955
4th Floor	12	Top	3410.745	3406.236	3416.629	3411.838	3413.154	3407.942
		Bottom	3410.745	3406.236	3416.629	3411.838	3413.154	3407.942
3rd Floor	9	Top	3977.101	3972.464	3960.531	3955.743	3877.265	3873.244
		Bottom	3977.101	3972.464	3960.531	3955.743	3877.265	3873.244
2nd Floor	6	Top	4392.88	4387.282	4352.849	4346.61	4228.183	4223.052
		Bottom	4392.88	4387.282	4352.849	4346.61	4228.183	4223.052
1st Floor	3	Top	4743.54	4735.85	4721.988	4713.43	4730.17	4731.805
		Bottom	4743.54	4735.85	4721.988	4713.43	4730.17	4731.805
Ground Floor	0	Top	5212.81	5201.249	5193.134	5178.198	4953.865	5125.585
		Bottom	5212.81	5201.249	5193.134	5178.198	4953.865	5125.585
Base/Footing	-1.5	Top	0	0	0	0	0	0
		Bottom	0	0	0	0	0	0

Table A- 9 Storey Shear of 10 storey building in X and Y direction

Storey Shear								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN	kN	kN	kN	kN	kN
Roof	30	Top	1365.68	1362.517	1394.933	1392.41	1512.07	1508.787
		Bottom	1365.68	1362.517	1394.933	1392.41	1512.07	1508.787
9th Floor	27	Top	2281.347	2279.172	2297.047	2294.795	2357.913	2354.244
		Bottom	2281.347	2279.172	2297.047	2294.795	2357.913	2354.244
8th Floor	24	Top	3221.791	3222.997	3229.016	3230.366	3247.061	3247.394
		Bottom	3221.791	3222.997	3229.016	3230.366	3247.061	3247.394
7th Floor	21	Top	4105.497	4109.712	4105.239	4109.169	4089.147	4092.145

		Bottom	4105.497	4109.712	4105.239	4109.169	4089.147	4092.145
6th Floor	18	Top	4911.252	4918.249	4902.691	4909.468	4845.853	4851.806
		Bottom	4911.252	4918.249	4902.691	4909.468	4845.853	4851.806
5th Floor	15	Top	5623	5632.207	5604.29	5612.91	5501.367	5508.822
		Bottom	5623	5632.207	5604.29	5612.91	5501.367	5508.822
4th Floor	12	Top	6242.983	6253.31	6212.844	6222.406	6052.809	6061.56
		Bottom	6242.983	6253.31	6212.844	6222.406	6052.809	6061.56
3rd Floor	9	Top	6726.907	6737.944	6683.134	6693.274	6455.286	6464.69
		Bottom	6726.907	6737.944	6683.134	6693.274	6455.286	6464.69
2nd Floor	6	Top	7070.707	7081.107	7018.298	7027.438	6775.303	6782.824
		Bottom	7070.707	7081.107	7018.298	7027.438	6775.303	6782.824
1st Floor	3	Top	7395.458	7404.515	7355.736	7363.684	7199.73	7203.558
		Bottom	7395.458	7404.515	7355.736	7363.684	7199.73	7203.558
Ground Floor	0	Top	7506.984	7515.926	7475.834	7483.631	7393.891	7393.301
		Bottom	7506.984	7515.926	7475.834	7483.631	7393.891	7393.301
Base/Footing	-1.5	Top	0	0	0	0	0	0
		Bottom	0	0	0	0	0	0

Table A- 10 Overturning Moment of 4 storey building in X and Y direction

Overturning Moment								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN-m	kN-m	kN-m	kN-m	kN-m	kN-m
Roof	12	Top	0	0	0	0	0	0
3rd Floor	9	Top	3628.78	3641.388	3911.464	3937.104	5038.317	5084.661
2nd Floor	6	Top	8293.526	8321.669	8639.247	8724.927	10029.2	10129.72
1st Floor	3	Top	14122.04	14145.52	14317.26	14473.43	15916.72	16094.85
Ground Floor	0	Top	21043.15	21060.42	21130.28	21366.77	23744.89	24028.19
Base/Footing	-1.5	Top	24923.83	24941.64	25038.32	25314.81	28564.77	28903.98

Table A- 11 Overturning Moment of 7 storey building in X and Y direction

Overturning Moment								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN-m	kN-m	kN-m	kN-m	kN-m	kN-m
Roof	21	Top	0	0	0	0	0	0
6th Floor	18	Top	3512.38	3523.855	3713.451	3723.531	4300.879	4317.419
5th Floor	15	Top	8785.915	8816.526	9026.342	9057.444	9438.171	9467.196
4th Floor	12	Top	16224.63	16265.44	16416.65	16458.15	16545.89	16578.76
3rd Floor	9	Top	25679.96	25725.54	25773.6	25819.83	25547.93	25581.59
2nd Floor	6	Top	36823.71	36873.43	36766.47	36817.54	36038.34	36071.25
1st Floor	3	Top	49216.63	49272.38	48937.31	48995.85	47488.66	47521.47
Ground Floor	0	Top	62430.52	62497.37	61904.49	61976.37	59776.82	59812.57
Base/Footing	-1.5	Top	69251.72	69326.02	68624.48	68704.6	66295.55	66338.53

Table A- 12 Overturning Moment of 10 storey building in X and Y direction

Overturning Moment								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN-m	kN-m	kN-m	kN-m	kN-m	kN-m
Roof	30	Top	0	0	0	0	0	0
9th Floor	27	Top	4087.552	4097.04	4177.229	4184.8	4526.36	4536.211
8th Floor	24	Top	10132.85	10150.83	10218.99	10236.79	10440.83	10459.96
7th Floor	21	Top	18853.08	18861.86	18900.66	18910.25	18908.13	18918.69
6th Floor	18	Top	30276.99	30262.85	30271.91	30259.37	30023.05	30012
5th Floor	15	Top	44174.56	44127.93	44100.83	44056.98	43514.52	43471.98
4th Floor	12	Top	60250.22	60163.78	60082.08	60000.14	59025.97	58944.76
3rd Floor	9	Top	78177.16	78046.76	77880.6	77757.45	76193.07	76069.88

2nd Floor	6	Top	97542.74	97366.42	97077.68	96911.82	94558.82	94391.27
1st Floor	3	Top	117891.4	117669.3	117217.7	117009.8	113690.5	113479.5
Ground Floor	0	Top	138909.1	138643.9	138015.8	137769.1	133462.2	133212.8
Base/Footing	-1.5	Top	149603.7	149318.3	148610.5	148345.7	143610.3	143345.7

Table A- 13 Storey stiffness of 4 storey building in X and Y direction

Storey Stiffness								
			Bare Frame		Infilled		Braced	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN/m	kN/m	kN/m	kN/m	kN/m	kN/m
Roof	12	Top	199910.9	195828	205708.7	202116.3	223042.1	219060
3rd Floor	9	Top	245957.9	240429.9	254562.2	249307.9	275716.9	269977.6
2nd Floor	6	Top	258246.4	253249	280840.2	275696.9	358835.1	352105
1st Floor	3	Top	336178.9	330789.3	433929.4	428958.4	1411285	1353392
Ground Floor	0	Top	2113170	2082701	2378789	2352211	3612190	3569002

Table A- 14 Storey stiffness of 7 storey building in X and Y direction

Storey Stiffness								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN/m	kN/m	kN/m	kN/m	kN/m	kN/m
Roof	21	Top	181123.8	176381.7	185395.7	180737.1	201097	196259.4
6th Floor	18	Top	220419.4	214214.2	222721	216642.3	233169.1	226442.6
5th Floor	15	Top	225904.5	220323.8	227584.8	222031.2	234062.6	228378.2
4th Floor	12	Top	226192.1	220725.9	228269.7	222793.1	235825.8	230231.5
3rd Floor	9	Top	230191.2	224896.6	234790.6	229490.4	248449.4	242990.9

2nd Floor	6	Top	247300.3	242168.3	263591.9	258284.1	323621.3	317054.9
1st Floor	3	Top	354777.4	348932	442045.7	436260.9	1359478	1309751
Ground Floor	0	Top	2850036	2814023	3189894	3149874	5075026	5154040

Table A- 15 Storey stiffness of 10 storey building in X and Y direction

Storey Stiffness								
			Bare Frame		**Infilled**		**Braced**	
Story	**Elevation**	**Location**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
	m		kN/m	kN/m	kN/m	kN/m	kN/m	kN/m
Roof	30	Top	179113.2	173716.4	181053.7	175717.1	190024.5	184637.3
9th Floor	27	Top	217398.3	210623.6	218368.3	211586.9	223894.4	216972.5
8th Floor	24	Top	221613.2	215284.2	222182.3	215867.9	225328.7	218968.7
7th Floor	21	Top	220631.5	214580.1	221092.1	215040.5	223702.3	217624.8
6th Floor	18	Top	219095.7	213289	219636.5	213839.5	222514.1	216699.3
5th Floor	15	Top	242100.2	235467.4	242932.8	236304.6	247212	240539.5
4th Floor	12	Top	252107.7	245555.4	253602.4	247058.8	260758.1	254162.7
3rd Floor	9	Top	270085.1	263603.1	273631.8	267149.2	290785.5	284042.2
2nd Floor	6	Top	327034.6	320139.6	339374.3	332317.2	414410.2	405615.8
1st Floor	3	Top	579297.8	568018.9	644881.2	634243.8	1456978	1409632
Ground Floor	0	Top	4020579	3942991	4266480	4185849	6897210	6747922

Table A- 16 Base shear Comparison in percentage increase

Building	**Bare Frame**	**Infilled**		**Braced**	
	Base shear (kN)	Base shear (kN)	% increase compared	Base shear (kN)	% increase compared to bare

					to bare frame (mm)				frame (mm)	
	X-Dir	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**	**X-Dir**	**Y-Dir**
G+3	3333.5058	3327.1635	3428.7959	3402.4388	3%	2%	4132.1416	4088.3809	24%	23%
G+6	5212.8102	5201.2485	5193.1341	5178.1978	0%	0%	4953.8652	5125.5853	-5%	-1%
G+9	7506.9835	7515.9259	7475.8337	7483.6314	0%	0%	7393.8911	7393.3007	-2%	-2%

Appendix "B" "Design outputs"

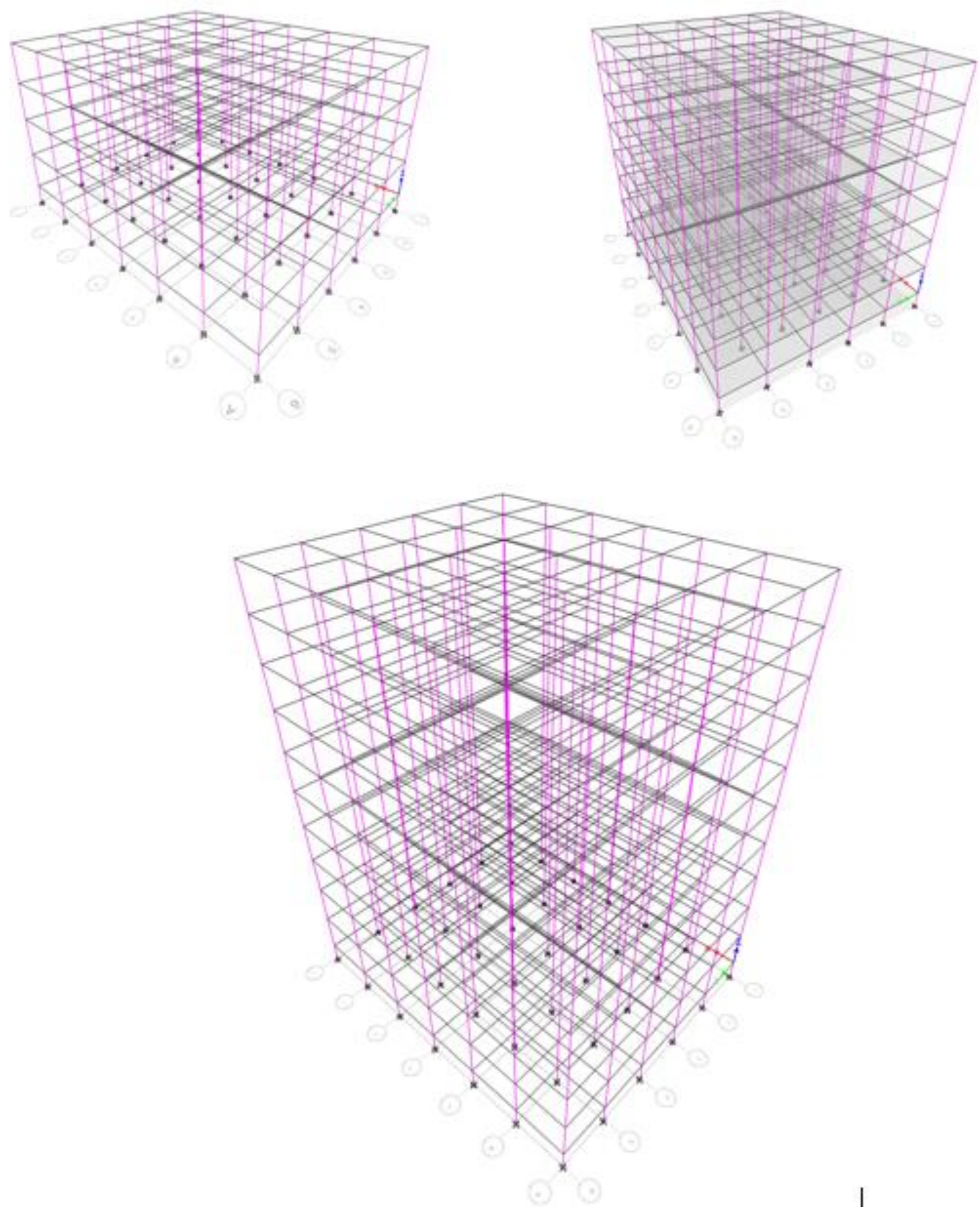

Figure B- 1 member design verifications for G+3, G+6 and G+9 building

Table B- 1 Modal Participating Mass Ratios

Case	**Mode**	**Period**	**UX**	**UY**	**UZ**	**SumUX**	**SumUY**
		sec					
Mode eigen	1	3.002	0	0.6895	0	0	0.6895
Mode eigen	2	2.962	0.6903	0	0	0.6903	0.6895
Mode eigen	3	2.677	0.000001264	0.00003441	0	0.6903	0.6896
Mode eigen	4	0.947	0	0.0978	0	0.6903	0.7874
Mode eigen	5	0.937	0.0976	0	0	0.7879	0.7874
Mode eigen	6	0.856	0	0.000004979	0	0.7879	0.7874
Mode eigen	7	0.526	0	0.0426	0	0.7879	0.83
Mode eigen	8	0.522	0.0423	0	0	0.8301	0.83
Mode eigen	9	0.477	0	0.000002317	0	0.8301	0.83
Mode eigen	10	0.349	0	0.0255	0	0.8301	0.8554
Mode eigen	11	0.347	0.0254	0	0	0.8555	0.8554
Mode eigen	12	0.317	0	0.000001424	0	0.8555	0.8554
Mode eigen	13	0.251	0	0.0172	0	0.8555	0.8726
Mode eigen	14	0.25	0.0171	0	0	0.8726	0.8726
Mode eigen	15	0.229	0	9.568E-07	0	0.8726	0.8726
Mode eigen	16	0.19	0	0.0132	0	0.8726	0.8858
Mode eigen	17	0.189	0.0132	0	0	0.8858	0.8858
Mode eigen	18	0.174	0	9.049E-07	0	0.8858	0.8858
Mode eigen	19	0.149	0	0.0099	0	0.8858	0.8958
Mode eigen	20	0.148	0.0099	0	0	0.8958	0.8958
Mode eigen	21	0.136	0	9.133E-07	0	0.8958	0.8958
Mode eigen	22	0.12	0	0.0061	0	0.8958	0.9019
Mode eigen	23	0.12	0.0061	0	0	0.9019	0.9019
Mode eigen	24	0.11	0	6.861E-07	0	0.9019	0.9019
Mode eigen	25	0.103	5.251E-07	0.0079	0	0.9019	0.9098
Mode eigen	26	0.103	0.0079	5.211E-07	0	0.9098	0.9098
Mode eigen	27	0.095	0	9.263E-07	0	0.9098	0.9098
Mode eigen	28	0.082	0.000001167	0.0053	0	0.9098	0.9151
Mode eigen	29	0.082	0.0053	0.000001167	0	0.915	0.9151

Mode eigen	30	0.076	0	6.056E-07	0	0.915	0.9151

Appendix "C" "Data used in the analysis"

Table C- 1 Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores

Source ES EN 1991-2015 table 6.1[29]

Table C- 2 Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	***q*k [kN/m²]**	***Q*k [kN]**
Category A		
(1) **Floors**	1.5 to 2.0	2.0 to 3.0
(2) **Stairs**	2.0 to 4.0	2.0 to 4.0
(3) **Balconies**	2.5 to 4.0	2.0 to 3.0
Category B	2.0 to 3.0	1.5 to 4.5
Category C		
- C1	2.0 to 3.0	3.0 to 4.0
- C2	3.0 to 4.0	2.5 to 7.0 (4.0)
- C3	3.0 to 5.0	4.0 to 7.0
- C4	4.5 to 5.0	3.5 to 7.0
- C5	5.0 to 7.5	3.5 to 4.5
Category D		
- D1	4.0 to 5.0	3.5 to 7.0 (4.0)
- D2	4.0 to 5.0	3.5 to 7.0

Source ES EN 1991-2015,Table 6.2 [29]

Table C- 3 Combination coefficients for variable actions

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see ES EN 1991-1-1:2015)			
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 area, vehicle weight ≤ 30kN	0.7	0.7	0.6
Category G : traffic area 30kN < vehicle weight ≤ 160kN	0.7	0.5	0.3
Category H : roofs	0	0	0
Snow loads on buildings	0	0	0
Wind loads on buildings (see ES EN 1991-1-4:2015)	0.6	0.2	0
Temperature (non-fire) in buildings (see ES EN 1991-1-5:2015)	0.6	0.5	0

Source ES EN 1900:2015, Annex A1.[30]

Table C- 4 Values of ϕ for calculating $\psi 2i$

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

Source ES EN 1998:2015 Table 4.2[4]

Table C- 5 Ground Types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30 cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S 1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
*S*2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

Source ES EN 1998-1-2015, Table 3.1

Table C- 6 Importance classes for buildings

Importance class	Buildings	Importance factor γI (recommended value)
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging in the other categories.	1
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4

Source ES EN 1998-1-2015, Table 4.3 [4]

Table C- 7 Basic value of the behaviour factor, q0, for systems regular in elevation

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3.0\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\alpha_u/\alpha_1$
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

Source ES EN 1998-1-2015, Table 5.1 [4]