An-Najah National University

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EFFECT OF SETBACK ON THE ELASTIC DYNAMIC RESPONSE OF REINFORCED CONCRETE FRAMED STRUCTURES

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Dedication

To my parents

To my brothers and sisters

To my brother and sisters

To my friends and colleagues

To my teachers

To everyone who left a fingerprint during my journey

Acknowledgements

Praise be to Allah, first of all, for helping me to complete this study.

Then, I would like to express my sincere thanks and gratitude to my thesis supervisor:

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For his guidance, suggestions, and assistance while doing this thesis.

Special thanks to my parents, brothers and sisters for their support and encouragement throughout my life.

Finally, I thank myself for arriving here and getting my wish, thank Allah.

Declaration

I, the undersigned, declare that I submitted the thesis entitled:

EFFECT OF SETBACK ON THE ELASTIC DYNAMIC RESPONSE OF REINFORCED CONCRETE FRAMED STRUCTURES

I declare that the work provided in this thesis, unless otherwise referenced, is the researcher's own work and has not been submitted elsewhere for any other degree or qualification.

Student's name Aseel Jamal Fareed Badran

Signature:

Date: 03/01/2023

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EFFECT OF SETBACK ON THE ELASTIC DYNAMIC RESPONSE OF REINFORCED CONCRETE FRAMED STRUCTURES

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Abstract

Background: The collapse due to a seismic load typically starts at locations of the weakness of the structural system in the building. The weaknesses often occur due to a discontinuity in mass, strength, or stiffness between two adjacent floors. A well-known type of vertical geometric irregularity in structures is a setback, defined as a sudden decrease in the building's lateral dimension at a certain height.

Problem Statement: The ASCE 7:16 code sets limitations on the Linear Equivalent Static method (LES), where it does not permit the use of this method for some types of irregularity, including setback irregularity, which are located in seismic design category (SDC) D, E and F. On the other hand, the LES method can be used in SDC B and C without restrictions. The code permits using the Modal Response Spectrum (MRS) and the Time History Analysis (THA) for all buildings without limitations on the SDC.

Objectives: This study examines the effect of a setback on the elastic seismic response parameters of reinforced concrete framed structures, such as the fundamental period, the seismic base shear, the inter-story distribution of shear forces, and story drift. Finally, this study examines the solution to the problem of the code restrictions on the use of the LES method.

Methodology: Buildings with Perimeter masonry walls with different setback ratios at different levels were analyzed using the commercial software ETABS 2016 according to the provisions of ACI318-14 and ASCE 7:16. The analysis was done using the LES method, MRS method, and THA method. The outcomes of the elastic response of the setback buildings were compared to similar buildings but without setbacks, in addition to comparing the results of the analysis methods used in this study.

Conclusion: Based on the results, the vertical distribution of shear force from LES was modified depending on the THA method results. This modification allows the use of the LES method in various SDCs.

Keywords: Linear Equivalent Static Analysis (LES); Modal Response Spectrum (MRS); Seismic Response; Setback structures; Time History Analysis (THA).

Chapter One
Introduction and Theoretical Background

1.1 General

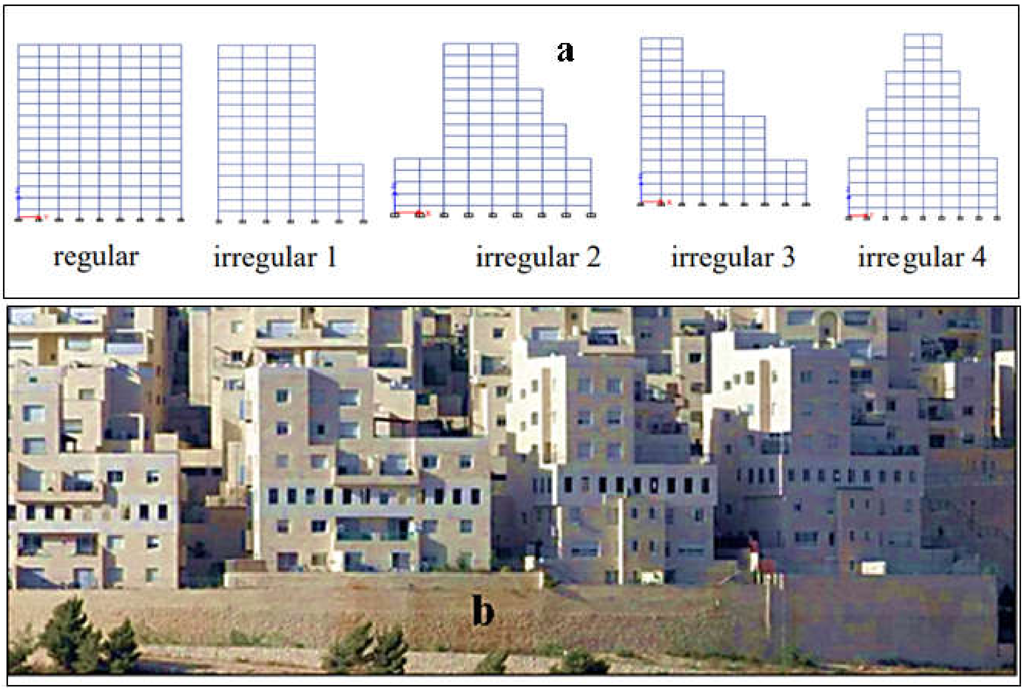
It is widely accepted that both horizontal and vertical stiffness, mass, and strength distributions affect the structural behaviour during earthquakes. Build failure due to earthquake loading typically occurs when a weak structural point exists. The main cause of this weakness is the existence of irregularities in a building's mass, stiffness, and strength. Having this in mind, there are two types of irregularities provided by the codes, which are horizontal irregularities and vertical irregularities.

According to table 12.3-1 in ASCE 7:16, the structure is classified as irregular in the vertical direction if one of these cases exists: stiffness irregularity, mass irregularity, vertical geometric irregularity, and discontinuity in a capacity-weak story. The structure is classified as irregular on the horizontal plan according to table 12.3-2 if at least one of the following cases exists: torsional irregularity, re-entrant corners, diaphragm discontinuity, out-of-plan offsets, and nonparallel systems.

A common type of vertical irregularity with a geometric discontinuity is called a setback. In the case of setback buildings, the length of the building is reduced over its height. Figure 1-1 (a) shows different types of vertical geometric irregularities, and figure 1-1 (b) shows an example of these buildings. The main reasons for using these buildings are the esthetic and functional criteria. The closeness of the buildings in urban environments makes these structures particularly useful. These structures offer enough light and ventilation for the lower stories in such areas.

Figure 1.1

(a) some of the vertical geometric irregular shapes and (b) setback buildings at Palestine.



Correct design of regular and irregular buildings' seismic resistance is important to maintain structural integrity and human safety. The seismic resistance of the structure is an important aspect of the design process to ensure that the building construction has a high earthquake resistance performance and reduces the damage caused by seismic activity, which has become a problem that engineers and researchers give considerable attention. Basically, seismic engineering state that there should be one or more reliable, strong, and stiff path to transfer loads from floors to the ground.

Most buildings are analyzed against earthquakes by applying the LES or linear dynamic methods like the MRS and the THA. Lateral forces are determined per the LES approach based on the mass and the fundamental period of buildings. The MRS approach, in theory, uses modal analysis to determine the building's natural periods and their participation to estimate the base shear. However, some international codes, like the American Society of Civil Engineers (ASCE7, 2016) and Indian Standard (IS1893, 2016), recommend scaling up the response quantities related to the fundamental period, the most important of which is the base shear. Thus, the base shear from the MRS method is equivalent to that from the LES method. As a result, determining the base shear using the equation of the code is unavoidable for the seismic design of structures.

The LES method can be used for regular structures with limited height, as lateral forces are computed according to the structure's fundamental time period based on code. The question remains: is the LES method for analyzing setback buildings located in the permissible seismic design categories effective?

Based on the brief introduction above, there is a necessity to investigate the impact of setbacks on the elastic dynamic response of structures.

1.2 The design code point view

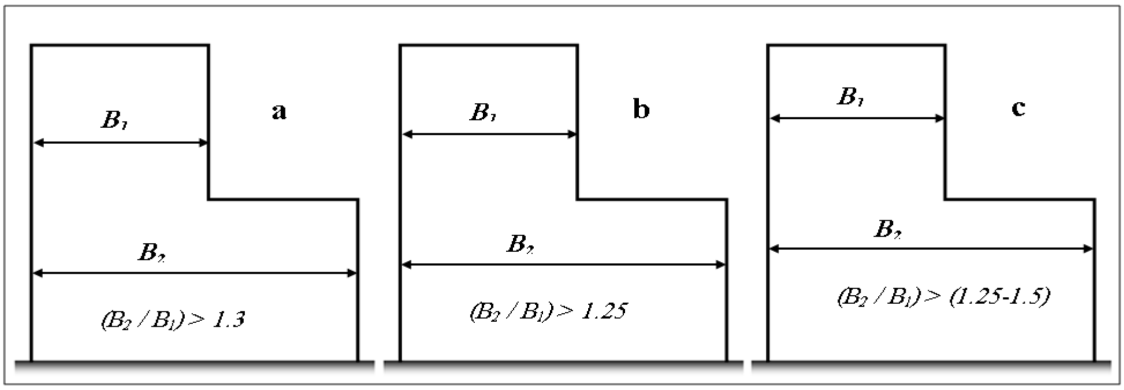
Almost all design codes, including ASCE7:16, IS1893:16, European Code (Eurocode8, 2004), etc., recommend an empirical equation for estimating the fundamental period of a structure. Design codes give different building types irregularities, each with a different definition than other codes. The following sections discuss the notion of setback irregularity and the methodologies for determining fundamental time periods in light of current design codes.

1.2.1 Vertical Geometric Irregularity

Vertical geometric irregularity is a subgroup of vertically uneven buildings called "setback buildings" with discontinuities in their geometry. According to ASCE 7:16, a setback is defined as existing where the horizontal dimension of the lateral force-resisting system in any story is greater than 130 per cent of that in the adjacent story, as shown in figure 1-2(a). However, according to IS 1893:2016, the percentage is greater than 125 per cent, as shown in figure 1-2(b). Regarding Euro code 8, the percentage is from 125 per cent to 150 per cent, as shown in figure 1-2(c). Design codes use the lateral dimension ratio of two adjacent floors to define setback irregularity. Design codes do not specify the degree and the level of irregularity in setback buildings; it is simply a guideline for identifying setback buildings from regular buildings.

Figure 1.2

Vertical geometric irregularity according to (a) ASCE 7:16, (b) IS 1893:16 and (c) EC8:2004



1.2.2 Fundamental Time Period

The natural period is the time required for a single degree of freedom to complete one cycle of free vibration. The fundamental time period is an important parameter in determining the dynamic behaviour of the building.

The natural period of the building is determined using three methods: theoretical modelling, numerical modelling, and empirical code equation. An empirical equation is used initially in the conceptual design phase because it is impossible to calculate the properties of a building that has not yet been designed. The simple properties known at the conceptual design phase are the construction material and the lateral resisting systems like shear walls, resisting moment frames, and dual systems, as well as the overall height and plans of the building. Theoretical and numerical approaches are usually used after preliminary design since they need more information in the calculations, such as the building's mass and stiffness.

The approximate fundamental period Ta (in seconds) of a structure with an overall height h_n (in meters) for reinforced concrete (RC) special moment resisting frame building (SMRF), as per ASCE 7:16, is given by:

$$T_{a,ASCE1} = 0.0466 \times {h_n}^{0.9} \qquad 1.1$$

Alternatively, ASCE 7:16 permits determining the fundamental time period from the following equation for buildings up to 12 floors above the base and where the average story height is at least 3 m, where N is the number of floors:

$$T_{a,ASCE2} = 0.1\, N \qquad 1.2$$

IS1893:16 and Eurocode 8 give the following formula to estimate the fundamental period of a RC moment resisting frame (MRF) building without brick infill:

$$T_{a,IS\ and\ EC8} = 0.075 \times {h_n}^{0.75} \qquad 1.3$$

Similarly, as Uniform Building Code (UBC, 1997) recommends the formula as shown:

$$T_{a,UBC} = 0.0731 \times {h_n}^{0.75} \qquad 1.4$$

These empirical equations for moment-resisting frame structures, prescribed in most building codes, were developed using an actual database of recorded periods of real buildings in earthquakes (Goel & Chopra, 1997). Particularly, Eq 1.1 was produced by regression analysis on the fundamental periods of 27 RC building frames, mainly those of 16-story or higher, located in southern California, measured during eight California earthquakes, starting with the 1971 San Fernando earthquake (M = 6.6) and ending with the 1994 Northridge earthquake (M = 6.7). It is noted that this formula was derived by subtracting one standard deviation from the best-fit curve, resulting in slightly lower period values; therefore, the base shear force is conservatively over-predicted, which is acceptable from a force-based design standpoint.

According to the codes, the fundamental period can be calculated using an alternative substantiated analysis such as normal mode analysis or Rayleigh's method. These theory-based techniques for obtaining the fundamental time period are cumbersome for most practising engineers because they require a computer program and long manual calculations.

The Rayleigh method is based on the properties of the structure and deformation. It is the most well-known approach for computing the fundamental period in theoretical models. Also, in Rayleigh's approach, the lumped mass distribution model is used to estimate the system's natural period quickly. This approach applies to systems with multiple degrees of freedom. The following is Rayleigh's equation for computing the fundamental period (in seconds):

$$T_{a,\mathrm{Rayleigh}} = 2\pi \sqrt{\frac{\sum_{i=1}^{n}(wi\ \Delta i^2)}{g\sum_{i=1}^{n}(fi\Delta i)}} \qquad 1.5$$

Where:

wi: Weight at floor i.

Δi : Elastic deflection due to lateral force at floor i.

g = Acceleration due to gravity.

fi = The lateral force at level i of the floor.

All of those mentioned above empirical fundamental period equations specified in codes (Eq. 1.1, Eq. 1.3, and Eq. 1.4) are a function of total building height and do not account for setback buildings, which are common in setback buildings. Rayleigh's formula, on the other hand, is a more rational method based on the features of the structures and the deformation properties.

1.3 Literature Review

1.3.1 General

In the late 1970s, researchers began focusing on vertically uneven structures' seismic behaviour. The following sections present previous studies and research on setback buildings. Part of these studies focused on improving the code equation of the fundamental period to make it applicable for setback buildings. These studies found that the problem of incorrect estimation of the dynamic response for setback buildings begins from the fundamental period. Since the fundamental period is an important factor in determining the behaviour of the building.

The other part of the previous studies focused on studying the effects of setback buildings on the seismic response factors, represented in the fundamental period, base shear, story shear distribution, story drift, and so forth.

The previous studies used static or dynamic analysis, according to the code followed and its limitations. The following section explains some of the limitations considered by ASCE7:16, IS 1893 and Euro-code 8 for the static analysis method.

1.3.2 Restrictions of design codes on the LES method

Most design codes, including ASCE 7:16, IS 1893 and Euro-code 8, set limitations on seismic analysis methods used for setback buildings, as these buildings are classified as vertically irregular. According to ASCE 7:16, the LES method can only be used for buildings in the seismic design category (SDC) B and C with no restrictions.

Additionally, it is allowed to use this method in SDC D, E and F for buildings with risk category I or II, which do not exceed two floors in height. In addition to the buildings that do not exceed 49 meters in height and do not have irregularities of certain types, these are setback irregularities.

IS 1893 code also contains provisions on the use of the LES method. The static analysis method is known as the seismic coefficient. This method is limited to regular buildings with a height not exceeding 40 m and to irregular buildings with less than 12 m located in the seismic zone V.

For Euro-code 8, static analysis is used for buildings with $T_{EC8} < 2$ s and $T_{EC8} \leq 4T_C$, where T_C is the controlled period. Also, it is used for structures regularly in elevations. The static analysis method cannot be used for irregular buildings, including setback buildings. Therefore, Euro-code 8 allows alternative methods for irregular buildings, such as the MRS and THA methods.

1.3.3 Previous studies on the setback buildings

(Bekele, 2022) conducted a study to assess the elastic seismic demand of RC structures with the setback. The MRS method was used to analyze the regular and irregular buildings. This study compared the seismic response results of setback buildings with regular buildings. This investigation's results show that the setback structures' seismic response parameters differed significantly from the regular buildings. For the setback buildings, the base shear, force distribution, and story drift are less compared to the regular building. Moreover, the research results show that the variance in seismic response of setback structures depends on the size and location of the irregularity.

A recent study (Thejaswini et al., 2020) conducted a numerical study to investigate the dynamic response of 5-story buildings, symmetric setback and asymmetric setback buildings. THA was done, and the dynamic response was assessed. Symmetrical and asymmetrical buildings with different irregularities were compared according to codes and papers. The result from the displacement and story drift comparison shows that the symmetrical setback buildings behave similarly to asymmetrical setback buildings, although having different irregularity indices according to IS:1893-2016.

Reviewing the literature review (Satish Paudel et al., 2019) shows that the LES and the MRS methods were used to investigate the seismic performance of RC buildings with various setback levels. The study shows that the change in the time period does not correspond to the methods used to define the number of setbacks. The comparison was made with the methods of codes and previous studies. Furthermore, the LES analysis gives higher response quantities values than the MRS method.

In addition, another study (Patel & Takkalaki, 2019) also assessed the efficacy of the current code formula for estimating the fundamental period of setback structures. A set of setback buildings with different irregularities and different heights were modelled. This study concluded that IS 1893:2002 code is very conservative in estimating the fundamental period. Also, the fundamental period of the building depends on the number of setbacks that occurred, so it is impossible to neglect the number of bays, the width of the bays, and the buildings' area in estimating the time period.

Another work (Kumar & Manjunath, 2018) studied the seismic performance of RC buildings with vertical geometric irregularities, considering the provisions of IS 1893:2016 code. The analysis was done using the LES, MRS, and THA methods. Compared with regular buildings, setback structures have higher values for story displacement and drift. The LES approach over estimates the values of seismic response parameters compared to the MRS technique. Finally, THA is more accurate than other analysis methods in predicting how the structure will react.

Seminal contributions have been made recently (Thakur & Rai, 2018) aimed to compare the fundamental period from modal analysis with the empirical equations and Rayleigh methods. Free vibration analysis was done on 90 setback building models. The study found that the relationship between 3D structures and the design code used to specify the number of setbacks from the original dimension is very weak. Therefore, it has been suggested that there could be other ways to define setback irregularity than using the design code.

A series of studies (Sayyed et al., 2017) conducted a study on the behaviour and performance of two types of irregularities, stiffness and setback. The comparison of different seismic responses between regular and irregular buildings was carried out using the MRS method. The result indicates that the setbacks affect the performance of the buildings and cause instability during earthquake loading. Therefore, irregularities should be prevented as much as possible, but if they are introduced, they must be carefully designed.

There have been numerous studies investigating the same topic. For example, researchers (Khaleeluddin & Azeem, 2017) studied setback structures with infill walls. The study aimed to verify the accuracy of the current code equation for estimating the fundamental period. Based on analytical studies, 90 setback frames with varying geometric irregularity and height have been modelled. An equation has been proposed to modify the empirical formula of expression of the time period.

The proposed equation for estimating the time period of setback buildings with brick infill is given:

$$T_{\text{Khaleeluddin and Azeem}} = 0.0038{h_n}^{1.34} \qquad 1.6$$

For $0.25 < B_2/B_1 \leq 0.66$

Where h_n is the overall building height in meter and B_2/B_1 is the setback ratio.

Previous studies (Alam & Prakash, 2017) studied the parameters affecting the period of setback buildings with and without shear walls. This is to develop a more reliable empirical equation for estimating the fundamental period based on the IS code. Sixteen setbacks building was selected with different irregularities. Based on the analysis, the formula for the fundamental time period is proposed as the following, where h_n is the building height, and A is the plan area of the building:

$$T_{\text{Alam and Parakash}} = 0.05143h_n + 0.00117\text{A} - 0.3707 \qquad 1.7$$

The above equation has an error range of (30% to 36%), which the researchers consider to be better compared to (the 45% to 47%) error given by the current code formula. Nevertheless, the error per cent is still high and needs further revision.

The study concluded that the key factor related to the fundamental period is height and the configuration of the structure. However, other factors, like the existence of non-structural elements, can have a substantial impact on the structures' dynamic behaviour.

In addition, (Uday Neelam Smt & Bajaj, 2017) conducted a parametric investigation on the fundamental period of several types of RCMRF with different floors, bays, and irregularities. Free vibration analysis was performed on 90 setback building models varying in height from 6 to 30 stories. This study aims to identify the parameters that describe the irregularity of setback buildings. However, the results of this study showed that it is challenging to determine the amount of setback irregularity using a single variable.

This has also been explored in prior studies (Rana & Raheem, 2015). Rand and Raheem studied the behaviour and performance of regular and setback buildings under earthquake motion. One of the goals of this research was to suggest the ideal building configuration based on the existing condition. The study concluded that the performance of regular and irregular buildings improves as the number of bays increases for tall buildings. Therefore, eight bays for high buildings (12 and 16 stories) have the best seismic performance, and four bays for lower building heights (up to 8 stories) is the best.

Prior research (Saraswathy et al., 2014) investigated the effect of geometric irregularity on the seismic performance of RC-framed buildings. Nonlinear analysis was used according to IS code. Twelve-story RC frame building with masonry infill walls with different setback irregularities was considered. The study concluded that the fundamental period of a setback structure is always shorter than a similar building without a setback. The fundamental period depends on the setback ratio and irregularity level. Also, with an increase in setback ratio, the top story drift increases and the base shear decreases.

For instance, another study (Varadharajan et al., 2014) developed an irregularity index (η_{ir}) for assessing setback irregularity based on the dynamic properties of structures. It takes into account the properties of the frame's mass and stiffness distribution. The study suggests a correction coefficient to the code equation for the fundamental period. This modification makes the formula more realistic for building with setbacks. The proposed irregularity index is as follows:

$$\eta_{ir} = \frac{\omega_i}{\omega_r} \qquad 1.8$$

Where ω_i and ω_r are the modal frequencies of vibration of irregular and regular structures. The correction factor $\acute{\lambda}$ was defined as the ratio of Ti/ T_r for the empirical formula of IS 1893:2002 as shown:

$$\acute{\lambda} = \frac{T_i}{T_r} = 4.4032\ \eta_{ir}^{\ 2} - 10.582\eta_{ir} + 7.2936 \qquad 1.9$$

$$T_{\text{Varadharajan}} = \acute{\lambda} \times 0.075 {h_n}^{0.75} \quad 1.10$$

Where h_n is the overall building height.

In the same way, an early study (Sarkar et al., 2010) offered a new method for assessing irregularities in setback building frames. This method takes into consideration dynamic properties such as mass and stiffness. A free vibration investigation of 78 setback frames with variable irregularity and different heights was conducted. Therefore, a correction factor is used to make the empirical code equation of the fundamental period applicable for setback buildings.

Sarkar et al. proposed the regularity index η as the following:

$$\eta = \frac{\Gamma_1}{\Gamma_{1,\text{ref}}} \quad 1.11$$

Where, Γ_1 is the first mode participation factor for the stepped frame and $\Gamma_{1,ref}$ is the first mode participation factor for the same regular building frame.

This regularity index measures vertical irregularity that accounts for changes in stiffness and mass along the building's height. The empirical formula of IS 1893:2002 was adjusted by defining a correction factor k, as shown, where h_n is the total building height:

$$T_{\text{Sarkar}} = 0.075 {h_n}^{0.75} \times k \quad 1.12$$

$$k = \frac{T}{T_{ref}} = [1 - 2\,(1 - \eta)(2\eta - 1)] \quad \text{For } 0.6 \leq \eta \leq 1.0 \quad 1.13$$

However, the previously proposed formulas (Varadharajan et al. and Sarkar et al.) are a function of the regularity index and irregularity index, which are a method of calculating the changes in stiffness and mass caused by the setback, and the proposed equations require the use of computer programs as well as complex manual calculations.

1.4 Problem Statement

The above studies discussed the effect of setback buildings on dynamic response factors. Each study used different analysis methods depending on the code's methods for setback buildings in the study area. None of these studies solved the problem of using the LES method for setback buildings. As previously explained, the design codes, including ASCE 7:16, IS1893:16, Euro-code 8:2004, UBC 97, and so forth., contain restrictions on the use of the LES method. Some researchers tried to solve the problem of estimating the fundamental period for setback buildings since the mentioned codes also give the same formula for all buildings.

Predicting a new equation to calculate the fundamental period of the structure is not feasible to solve the problem of designing and analyzing setback buildings. Because of the limitations of using the static analysis method for irregular buildings, the fundamental period based on the equations becomes unnecessary. The most important thing is that previous studies also did not reach a correct definition of setback buildings or find a factor describing this irregularity.

As for the SDC, where the static analysis method is allowed, the fundamental period equation becomes important. Setback buildings have different dynamic characteristics than regular buildings due to the variations in geometrical and structural properties. Therefore, measuring the fundamental period of setback structures using the code equation while neglecting the height difference results in an incorrect natural period. As a result, the design against seismic excitation is not representative. Consequently, the resulting values of seismic parameters (base shear, distribution of inter-story shear, story displacement, story drift, and so on) will be incorrect.

Making the static analysis method available in all cases without restrictions is one of the most important reasons for this study. Almost all design codes still use the static analysis method since it relies on manual calculation and theoretical formulae.

1.5 Research Objective

A detailed examination of the literature established the goals of the thesis. Most studies were limited to clarifying the effects of setback buildings on dynamic response. The efficacy of the static analysis approach for setback buildings has not been investigated, and none of these studies addresses making the LES method available for use in all SDCs. The following are the main goals of the current research:

- Studying the effect of a setback on the seismic response of SMRF structures and comparing it to the regular building.
- Comparing the base shear results, inter-story distribution of shear, and story drift from the three methods of seismic analysis (LES, MRS, and THA) for setback structures.

- Verify the LES's effectiveness for setback buildings and the possibility of applying it to different SDCs, then modify the formula of the vertical distribution of inter-story shear forces from the LES method based on the results of the THA method.
- Comparing finite element results of fundamental period of setback structures to ASCE7:16 code formula and Rayleigh method.

1.6 Importance of the study

Setback irregularity in the buildings cannot be avoided or prevented, so it is necessary to study the behaviour of the setback buildings under seismic loading. Knowing the effects of a setback on dynamic response factors is important so that the building is designed correctly and carefully.

In general, dynamic analysis is accepted as the best approach to obtain the distribution of seismic load for setback buildings. However, most building codes, including ASCE 7:16, recommend scaling up the seismic base shear determined by the dynamic analysis approach using the base shear determined by the LES method. As indicated in the previous sections, ASCE 7:16 put restrictions on using the LES method for setback buildings. This method is the easiest and most conservative, so it is necessary to solve the problem of the distribution of inter-story forces to make this method available and applicable for setback buildings unconditionally.

1.7 Limitations of the study

The following limitations are made in this thesis:

1. The current research is limited to RC buildings of SMRF according to ASCE 7:16, ACI 318:14 and IBC:18.
2. Linear elastic analysis was performed on the structures.
3. The impacts of soil-structure interactions are ignored in this study.
4. It is assumed that the column ends are pinned at the foundation.
5. The setbacks of the buildings are assumed to be in one direction only.
6. The columns are supposed to be square.
7. The compressive strength for all members is 24 MPa.

1.8 Methodology

The following are the steps used in this study to reach the objectives mentioned earlier:

First, a literature review was undertaken to understand the parameters that affect the dynamic response of the setback building frames and to study the method of analysis of the structures. This also helps to benefit from the analytical and experimental results and compare them with the current study results.

Different buildings were modelled with different heights and setback ratios, and then the results of the setback buildings were compared with the regular buildings using the three methods of analysis: LES, MRS, and THA, as well as the comparison between the results of the used analysis methods. The fundamental period resulting from the modal analysis was compared to ASCE 7:16 formula.

The finite element method-based commercial program (ETABS, 2016) was selected as the calculation tool to get the findings from various representative models that were simulated in this study.

Finally, the equation of the vertical distribution of shear force inter-story for setback structures was modified using statistical regression to reflect the actual behaviour of setback buildings.

1.9 Thesis outline

The thesis consists of four chapters as follows:

Chapter 1 (Introduction and Theoretical Background): it is an introduction to the study, and it offers the reader a background and motivation, a design code perspective, a literature review presentation of the results of the previous studies, a problem statement, research objectives, importance of the study, limitation of the study, and the methodology.

Chapter 2 (Methodology): it describes the modelling aspects of setback buildings and the geometries considered in this study. Also, this chapter presents the methods that are used in the modelling and analysis of buildings.

Chapter 3 (Results): it starts with presenting the results of the fundamental time period and comparing them with the ASCE 7:16 equation and Rayleigh method results. Also, this chapter shows the base shear values from the three methods and compares them. Further, this chapter presents the effect of setback ratio and the story level at which setback occurs on the shear force distribution. Finally, this chapter includes a section for data fitting to modify the equation of shear force distribution for setback buildings, validating the modification based on the story's maximum drift results, and modelling a new case.

Chapter 4 (Conclusion and Future work): the final chapter contains a section to discuss the results and answer the questions, as well as there are sections to offer significant findings and conclusions of the study and future work.

Chapter Two

Methodology

2.1 Introduction

The first section of this chapter provides an overview of the different parameters that define the computational models, as well as the basic hypotheses and building geometries that were taken into account for this research. The structural models were designed per ASCE 7:16 and American Concrete Institute (ACI318, 2014) provisions.

The later sections of this chapter describe the design and analysis approach used in this study. Seismic analysis of a building can be done using various approaches with different degrees of efficiency and accuracy. LES, MRS, and THA methods were used in this study. Additionally, modal analysis was used to determine the fundamental period of the structural models.

2.2 Motivation for analysis

The MRS and THA methods can be used for all buildings without conditions on the SDC. Conversely, the ASCE 7:16 imposed limitations on the LES method, making it inapplicable to setback buildings except in two cases. The first case is when the height of the building does not exceed two floors with risk category I or II. The second case is if the irregular building is located in SDC B and C.

Based on the foregoing, it is necessary to modify the method of LES for-setback buildings. This will be done based on accurate analysis methods that use real records of ground motion, such as the THA method, to make the LES available for use without limitation on the SDC. The LES will be improved by modifying the distribution of inter-story shear. This will be explained later in chapter 3 based on the outcomes.

2.3 Computational model

The modelling and assembling of a building's different load-carrying parts is part of the modelling process. The model must all correctly represent the mass distribution, stiffness, strength, and deformability. The material characteristics, loads, structural members, and structural systems used in the modelling are discussed in the following sections.

2.3.1 Material properties

ACI 318-14 and ASCE 7:16 were used for designing concrete members. The ultimate strength method was used for the design. The strength of concrete for all structural elements is M-24 grade. The elastic modulus of concrete (Ec) is determined as follows:

$$Ec = 4700\sqrt{f'_c} \text{ MPa} \tag{2.1}$$

Where f'c is the concrete compressive strength in MPa at 28 days (24 MPa in this case), the unit weight for reinforced concrete is 25 KN/m^3. With regard to rebar, steel yield strength (fy) is 420 MPa, and the modulus of elasticity of steel (Es) is 2×10^5 MPa.

2.3.2 Vertical loads

The live loads (LL) were taken to achieve the minimum requirements of the ASCE 7:16 code. The superimposed dead load (SID) is determined based on typical finishes in Palestine (See Appendix A). Table 2.1 shows the considered vertical loads. The mass source determining the fundamental period is the dead load (DL), SID, and a quarter of LL.

Table 2.1

Summary of adopted vertical load

Loads	Value	Unit
L L	2	kN/m^2
SID	4	kN/m^2
Perimeter wall	16	kN/m

2.3.3 Structural system

There are many structural systems used to resist lateral loads resulting from an earthquake. Among the most famous of these systems, which are the most widely used, are moment-resisting frames. These frames resist gravity and earthquake loads as the structure has no shear walls.

Special proportioning and design requirements result in a frame that can resist high seismic shaking while maintaining strength and stiffness. Because of these additional considerations, these types of moment-resisting frames are known as special moment-resisting frames (SMRF), which increase the earthquake resistance compared with less detailed ordinary and intermediate moment frames (Moehle et al., 2008).

When architectural function planning flexibility is required, moment frames are commonly used as a seismic force resistance system. If concrete moment frames for structures in SDC D, E, or F are chosen, they must be detailed as SMRF. However, it may be employed in SDC B and C, although this may not result in the most cost-effective design. Nonetheless, the limitation of using the static analysis by ASCE 7:16 is that using LES is only available in SDC B and C. This type of moment-resisting frame was chosen in SDC C in this study.

2.3.4 Structural elements

Preliminary design is used to get initial dimensions for the elements before modelling the structure by ETABS, such as columns and beams dimensions. After selecting the structural system, ACI318:14 tables and equations of the slabs were used to calculate the minimal preliminary dimensions, which were then increased depending on the load applied and the necessary checks. ETABS software was used for the 3D modelling of the structure. Checks were done to confirm that the model conforms to the code (See Appendix D).

To justify the strong-column, weak-beam theory, the cross-sectional areas of the columns are kept larger than those of the beams.

The beams mostly transfer gravity loads to the columns, spreading them into foundations and the ground. The frame systems primarily resist the earthquake loads, consisting of columns and beams. The columns ending at the foundation were considered pinned in this study.

2.3.5 Seismic Parameters

The following table shows the input seismic parameters that are applied for all cases:

Table 2. 2

The seismic parameters for all cases.

Factor / Coefficient	Value
Building type	Residential
Frame type	SMRF
Seismic Zone Factor (Z)	0.15
Risk Category (RC)	II
Soil / Site Classification	B
Importance Factor (Ie)	1
Mapped spectral response acceleration parameter at a period of 1.0 sec (S1)	0.28125
Mapped spectral response acceleration parameter at short periods (Ss)	0.56250
Design spectral acceleration parameters at short periods (SDs)	0.33750
Design spectral response acceleration parameter at period of 1.0 sec (SD1)	0.15
SDC	C
Response Modification Coefficient (R)	8
Over-strength Factor (Ω)	3
Deflection Amplification Factor (Cd)	5.5

2.4 Buildings Modeling Description

The study is based on a 3D RC structure with different heights. For this study, various building geometries were used. These building geometries reflect different levels of setback irregularity. This study assumes a uniform number of bays at the base (4 bays) in both horizontal directions with a 4 m bay width. It should be mentioned that a bay width of 4 m is common in Palestine. Similarly, the study investigated three heights ranging from 5 to 13 stories, with a constant story height of 3 m. The thickness of the solid slab is calculated according to ACI318-14, and it is found to be equal to 20 cm. The checks of the slab thickness and the column dimensions were done, and they were OK (See Appendix B). The building geometries considered in the present study are shown in Table 2.3.

Table 2. 3

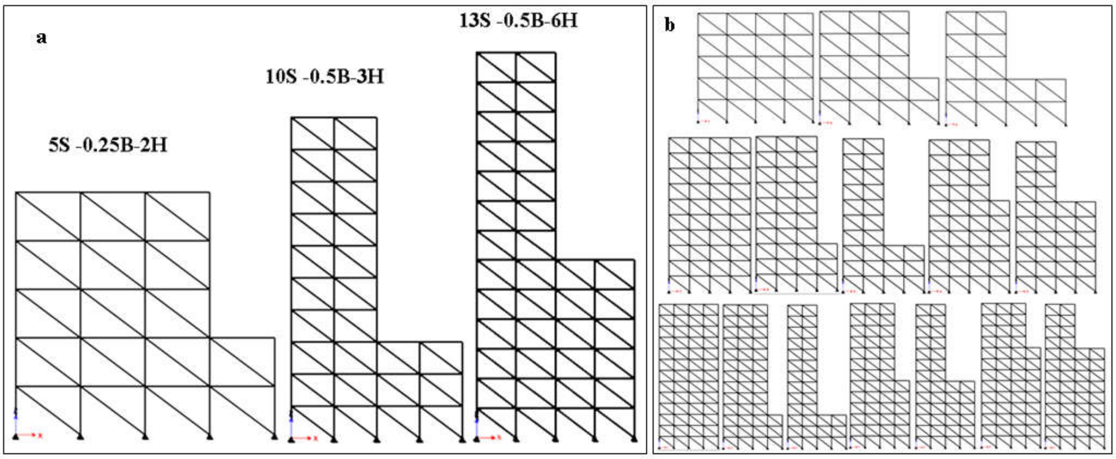
Dimension of beams and columns for various buildings

Stories' number	Beams (cm)	Columns (cm)
Five-Story	40*40	40*40
Ten-Story	40*50	50*50
Thirteen-Story	40*60	60*60

The structures are 3D, with irregularities in the setback direction. Setback frame buildings are named based on the setback ratio in the bay and the level where the setback occurred (from the ground floor), as shown in Figure 2-1(a). The symbolism of S-B-H is the adopted nomenclature of the identity of the structure, where S denotes the number of stories, the B symbol denotes the setback ratio in the horizontal x-direction, and H represents the level where the setback occurred. The R symbol represents a regular frame. To clarify, 5S-0.25B-2H represents the building with five stories, the ratio of setbacks equal to 0.25 of the total number of bays, and the setback occurred at the second level.

Figure 2. 1

(a) Example of the exact nomenclature of the structures and (b) Building models used in the study.



This study considers three different heights: 5-story, 10-story, and 13-story, as shown in Figure 2-1 (b). The setback ratios are 0.25 and 0.50. The setback occurred at the second level for 5-story, the third and sixth levels for 10-story, and the third, sixth, and ninth levels for 13-story.

Choosing this type of setback building instead of a gradual setback because it is the worst. The type of setback chosen in this study occurs suddenly, and its effect on seismic performance is greater, especially if the setback ratio is large, in contrast to the gradual setback.

2.4.1 External masonry walls

The structures used for this study were modelled with external masonry walls since typical concrete-framed structures in Palestine contain exterior walls of unreinforced concrete or void-brick layers.

The common method of analysis in Palestine is to neglect the contribution of the masonry walls in the modelling. This ignoring does not reflect the actual behaviour of the building, which leads to an incorrect prediction of the seismic response and an incorrect seismic design.

There are two different approaches for modelling masonry walls, each with varying degrees of complexity. The first approach is Macro Modeling, which substitutes infill walls with equivalent struts, while the other is based on the finite element approach and is called Micro Modeling.

Reinforced concrete frames with masonry walls can be modelled by replacing the infill wall with an equivalent diagonal strut. The defining aspect of the diagonal strut is an equivalent width, which impacts the durability of frames and lateral stiffness.

The equivalent strut is the simplest, easiest, and most acceptable method for modelling, so it is used in this investigation. The width of the strut was calculated using the Federal Emergency Management Agency (FEMA356, 2002) formula. The equivalent diagonal compression strut of width (α) is given by the following equation:

$$\alpha = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \qquad 2.2$$

Where:

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}} \qquad 2.3$$

h_{col} is the column height between centerlines of beams, m.

E_{me} and E_{fe} are the expected of the elastic modulus of infill material and frame material, respectively, MPa.

I_{col} : Moment of inertia of column, m^4.

r_{inf} : Diagonal length of infill panel, m.

t_{inf} : Thickness of infill panel, m.

The following table shows the values of the strut width for building models. For the calculation of the strut width (See Appendix C).

Table 2. 4

The strut width of building models

Case	Strut width (mm)
5-story	508
10-story	545
13-story	568

The presence of masonry walls has a direct impact on the natural period of a building. Therefore, masonry walls must be considered during the modelling and analysis process. To illustrate this impact, 5S-0.5B-2H with masonry walls was designed and analyzed. Then, the same building was analyzed without the masonry walls.

The fundamental period of the building neglecting the masonry walls was 1.21 sec, while in the case of the presence of the masonry walls, the period was dropped to 0.804 sec. In fact, this result is expected because the masonry walls increase the stiffness of the building and thus reduce the natural period, which gives higher base shear. Ignoring the impact of masonry infill walls leads to underestimating the base shear acting on a building during seismic action. The following section shows the modelling of one case of a 10-story setback building without masonry walls for comparison.

2.4.2 Case without masonry walls

In this section, one case of a 10-story setback building was modelled, which is 10S-0.25B-3H. This case was modelled without a masonry wall, and the column ends are fixed at the foundation. The building has been designed taking into account the requirements of a SMRF. A SMRF building is designed to achieve the following main goals:

1. Achieve the philosophy of strong-column, weak-beam.
2. Avoid shear failure.
3. Provide details allowing ductile flexural response in yielding regions.

To achieve the first goal, columns should provide a strong and stiff spine over the height of the building so that drift is distributed more uniformly, thus, reducing local damage. In addition, columns on a given story must support the weight of the entire building above them, whereas beams merely support the gravitational loads. As a result, column failure has more effects than beam failure. The strong-column, weak-beam theory ensures that frames behave safely during strong ground shaking. The application of a capacity-design technique prevents shear failure, this for the second goal. The ductile behaviour of RC members is achieved by confinement for the heavily loaded sections, ample shear reinforcement, and prevention of anchorage or splice failure. This summarizes the main points of design concerns for SMRF.

2.5 Analysis procedure

Seismic assessment is an essential tool in earthquake engineering that helps engineers better understand how buildings respond to earthquake excitation. Previously, buildings were planned only for gravity loads, and seismic analysis is a new development. When earthquakes are common, they must be an aspect of structural analysis and design.

Three methods of analysis are allowed by ASCE 7:16 to determine the seismic forces: LES, MRS, and THA, which will be explained in the following sections.

2.5.1 Linear Equivalent Static Analysis (LES)

Static analysis is the simplest method and works well in low-rise buildings. In this method, the seismic load (lateral force) is applied to the buildings as a constant load, and the calculations related to this method are easy, as it does not take into account the effect of higher modes. This method is not allowed for long-period buildings (fundamental period $T > 3.5$ seconds) or structures in SDC D, E, and F that have certain vertical or horizontal irregularities. At the same time, this analysis procedure can be used in SDC B and C without restrictions. The static analysis method assumes that the distribution of mass and stiffness changes gradually along the elevation and neglects the effect of torsion. Therefore, the use of this method is limited to buildings with irregularities mentioned in table 2.5.

Most of the cities of Palestine are located in the SDC D, where the SDC D constitute more than approximately 50%, based on the seismic hazard map at Palestine. Accordingly, according to Z values, the LES method is available in most areas of Hebron and west of Ramallah, as well as some areas of Tulkarm and Qalqilya, with an emphasis on the soil being rocky.

Table 2. 5

Allowed analytical procedures according to ASCE 7:16

Seismic Design Category	Structural characteristic	Equivalent Lateral Force Procedure	Modal Response Spectrum Analysis, or Linear Response History	Nonlinear Response History Procedures
B, C	All structures	P	P	P
D, E, F	Risk category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 48.8 m in structural height	P	P	P
	Structures exceeding 48.8 m in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures exceeding 48.8 m in structural height and only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4,5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP		

P: Permitted; NP: Not Permitted; $T_S = S_{D1}/S_{DS}$

The seismic base shear estimated using the LES method is based on an approximate fundamental time period (Ta) unless the period of the building is calculated by analysis. In most cases, the analysis will reveal that the actual structure period is longer than the approximate period, allowing the computed base shear as per code equations to be reduced. The obtained base shear will most likely be limited by the upper limit on the period (Cut), where the coefficient Cu is obtained from Table 2.6.

Table 2. 6

The coefficient (Cu) for upper limit on calculated period according to ASCE 7:16

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficient Cu
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

2.5.1.1 Equivalent Lateral Force (ELF)

According to ASCE 7:16, the seismic base shear (V) is determined by the following equation:

$$V = C_S W \tag{2.4}$$

Where Cs is the seismic response coefficient, and W is the effective seismic weight.

The coefficient Cs is determined as the following:

$$C_S = \max\begin{cases} \min\begin{cases} C_{S\,max} = S_{DS}\left(\frac{I_e}{R}\right) & \\ \frac{S_{D1}}{T}\left(\frac{I_e}{R}\right) & \text{For } T \le T_L \\ \frac{S_{D1}T_L}{T^2}\left(\frac{I_e}{R}\right) & \text{For } T > T_L \end{cases} \\ C_{S\,min} = \max\begin{cases} 0.044 S_{DS} I_e & \\ 0.01 & \\ 0.5 S_1\left(\frac{I_e}{R}\right) & \text{For } S_1 \ge 0.6 \end{cases} \end{cases} \tag{2.5}$$

Where:

T_L: long-period.

2.5.1.2 vertical distribution of forces

The following equations give the lateral seismic force (Fx) at any level:

$$F_X = C_{vx} V \tag{2.6}$$

$$C_{vx} = \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k} \tag{2.7}$$

Where C_{VX} is the factor for vertical distribution.

w_i and w_x are the effective seismic weights of the structure at level i or x.

h_i and h_x are the heights from the base to level i or x.

k is an exponent related to the structure period as follows: for buildings with $T \leq 0.5$ s, k = 1. For buildings with $T \geq 2.5$ s, k = 2, and for buildings with $0.5 < T < 2.5$ s, k is calculated by linear interpolation.

2.5.2 Modal Response Spectrum (MRS)

In many cases, the MRS method is preferred to consider the total dynamic behaviour of the building and to take advantage of determined, rather than approximated, time periods. The base shear from a combined response using the MRS method can be lower than the base shear predicted by the LES approach. This is another benefit of the MRS method. However, in many cases, the seismic base shear obtained from MRS should be scaled to at least 85 per cent of the LES seismic base shear.

The MRS approach has been widely utilized in practice because it effectively calculates the maximum values of the building's earthquake response and has a straightforward and small calculation methodology.

The behaviour of regular structures with a uniform shape and a uniform distribution of stiffness and mass is simple. However, the behaviour becomes complex in uneven structures or includes areas of discontinuities or irregularities. MRS analysis is utilized to estimate the important response properties for such structures, including the impact of the building's dynamic properties on lateral force vertical distribution and the increase in deformations and story shears due to the impact of higher modes.

The simplified approaches are adequate for regular buildings but do not account for complicated structures' full spectrum of seismic behaviour. As a result, linear dynamic analysis is the preferred approach for structures with uneven or irregular geometry, as mentioned previously.

2.5.2.1 Modes of vibration

When considering free vibration, the building is not exposed to any external excitation, and the only factor influencing its motion is the initial conditions. In some cases, it is required to determine how a building might move under free vibration conditions. However, the study of the building in free motion reveals the structure's most essential dynamic characteristics: the modal shapes and natural frequencies that correspond to them. According to ASCE 7:16, a minimum number of modes must be included in the analysis to achieve mass participation through a combination of modes at least 90 per cent of the real mass in each orthogonal horizontal response direction taken into account by the model.

Because mode shapes are usually acquired in the normalized form in modal analysis, the response spectrum method's results must be scaled suitably. The scaling in this study was accomplished by equating the base shear result obtained from LES with that obtained from RSM.

The model was simultaneously excited by 100 per cent of the seismic load in the critical direction (in the setback direction, x-direction in this study) and 30 per cent of the seismic load in the perpendicular direction (y-direction).

The ASCE 7:16 code offers different methods for combining the mods, including the square root of the sum of squares (SRSS) and the complete quadratic combination (CQC). CQC is the general method and should be used if the mode values are closely spaced, while SRSS is a special case of CQC. Therefore, the CQC is recommended because it is the most general approach. Most software programs, including ETABS, use the CQC method by default.

2.5.3 Time history analysis (THA)

This method involves progressively solving the equations of motion of the multiple degrees of freedom that reflect the real response of the structure in the time domain. It is the most advanced analysis technique available in structural engineering. It depends on the seismic ground motion as an input function for a particular building. Therefore, many input ground motion records are needed to obtain a good sense of the building's seismic response. This analysis approach is often limited to examining the applicability of assumptions established during the design of significant buildings instead of providing a way of allocating lateral loads themselves.

The THA approach has advantages over the MRS method, including being the most mathematically correct, preserving the sign of response quantities and not losing them due to the combination of modal responses as in MRS analysis, and providing more accurate results for story drift. Despite the method's benefits, there are a few drawbacks that make it relatively challenging since they include the need for choosing and scaling the proper ground motion, resource-intensive analysis, and a large number of results that require time for post-processing.

2.5.3.1 selection of ground motion and scaling

THA solves the structural model to obtain its dynamic response by suitability to suites of acceleration histories that match the site's intended response spectrum.

The most recent edition of ASCE 7:16 specifies that at least eleven ground motions must be employed. In this case, the mean values of the results from each ground motion are used for evaluation. The usage of at least three ground motions was required in the previous edition of the code; in this instance, the maximum values of each motion are adopted. However, if seven or more motions are used, the average values of the results are utilized for evaluation.

According to the ASCE7:16 code, some properties must be considered when selecting seismic records, including that the ground motions shall be within the same general tectonic regime, have the same magnitude and fault distances, and have a similar spectral shape to the target response spectrum.

Our region's tectonic system is closely related to that of the Middle East and the Eastern Mediterranean, which is regarded as one of the earth's primary belts and an active zone. There are several major faults in Palestine, and they are responsible for seismic activity (Malkawi et al., 1995). The Dead Sea is the main fault affecting Palestine and is considered the closest to the study area. The Nablus Earthquake, which occurred in 1927 and had a magnitude of around 6.2, was the largest recorded earthquake (Jardaneh, 2004).

Two methods for modifying ground motion records to be compatible with the target response spectrum are spectral matching and amplitude scaling.

Spectral matching is based on modifying the original time series of acceleration to match the full range of the target spectrum with minimal change of the records' history for velocity and displacement. While the amplitude scaling method measures, the motion to make the spectral acceleration ordinate corresponds to the design spectral acceleration within the fundamental period of the building.

A number of authors (Heo et al., 2011) conducted a study to evaluate the methods of modifying the ground motion and to examine their suitability. The study's results indicate that spectral matching is generally more stable than amplitude scaling in regard to bias and the resulting dispersion in the expected demands. In addition, using seven ground motions is insufficient to create median demands for higher frames in which multiple modes affect structural response. However, a spectral matching approach was used in this study.

For THA analysis in this thesis, two sets of seismic data were considered, with a total number of ground motion records of 14.

1. International Building Code (IBC, 2018) earthquake records with peak ground acceleration (PGA) of (0.15-0.2) g. For the diagram of acceleration time histories of these records (See appendix E).
2. Imperial Valley earthquake records (El Centro earthquake). These earthquake ground motions came from the Pacific Earthquake Engineering Research Center (PEER, 2022). The following table shows the characteristics of the Imperial Valley records used in this study. For the diagram of acceleration time histories of these records (See appendix E).

Table 2. 7

characteristics of the records

No.	Earthquake	Station	Year	Magnitude
1	Imperial Valley-02	El Centro Array #9	1940	6.95
2	Imperial Valley-02	El Centro Array #9	1940	6.95
3	Imperial Valley-01	El Centro Array #9	1938	5
4	Imperial Valley-03	El Centro Array #9	1951	5.6
5	Imperial Valley-04	El Centro Array #9	1953	5.5
6	Imperial Valley-05	El Centro Array #9	1955	5.4
7	Imperial Valley-08	Westmorland Fire Station	1979	5.62

Chapter Three

Results

3.1 introduction

The selected buildings modelled in this study were analyzed for an elastic dynamic response using ETABS 2016. This chapter presents and discusses the outcomes of the analytical study. According to the goals of this thesis, the presented results mainly focus on the effect of the setback on the building's elastic dynamic response, which is represented by the fundamental time period, base shear, story shear, and maximum story drift, in addition to comparison of them using different analysis methods.

This chapter also presents the solution to the problem resulting from the limitation of the use of the LES; as mentioned in the previous chapters, ASCE 7:16 code set restrictions on this method and limited its use in the SDC B and C. In other SDC, it is used for regular buildings in general. In addition, for buildings with a risk category I or II and do not exceed two stories.

The objective of this modification is to make the LES method available for use in different SDC. This method gives conservative results, and the design remains safe. The LES approach is based on manual calculations and theoretical equations, so almost all codes still use this method. On the other hand, the MRS method depends on computer programs, which can produce inaccurate results or be difficult to ascertain and judge its correctness.

After modifying the LES method, this chapter presents the results of the maximum story drift before and after the modification. Finally, this chapter includes a section to validate the results obtained.

3.2 Dynamic seismic response

The following sections present the effect of setback buildings on the dynamic response factors, which are represented by the fundamental time period of the structure, base shear, and the inter-story distribution of shear forces.

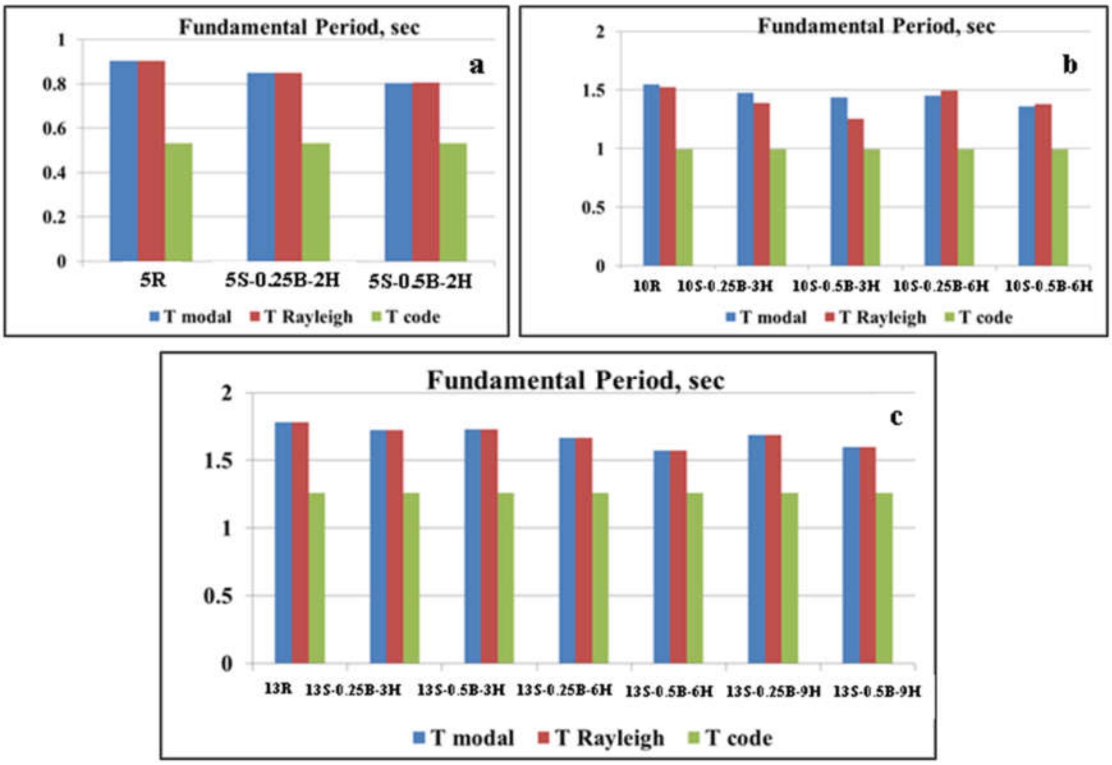
3.2.1 Fundamental period for setback structures

This section presents the time period results from all the cases selected in this study and compares them with the regular building and the code equation. As previously explained, the fundamental period is the key factor in determining the dynamic behaviour of the building.

The fundamental period for the structural models was calculated from the empirical equation of the ASCE 7:16 (Eq 1.1), as well as from modal analysis and the Rayleigh method (Eq 1.5). The considered fundamental period in this comparison is taken from the first mode. The following figure shows the results of the fundamental time period.

Figure 3.1

Fundamental time period for (a) 5-story buildings, (b) 10-story buildings, and (c) 13-story buildings.



The analytical study shows that the fundamental period of a structure is directly proportional to the overall height of the building. As the height of the building increases, the fundamental period in the regular building and setback building increases as well. Also, the fundamental period values for buildings containing setbacks are lower than regular buildings.

The fundamental period is inversely proportional to the setback ratio, where for the same story, the higher setback ratio gives the shorter fundamental period due to the lower seismic weight resulting from the irregularity, where this can be observed in the cases of 13S-0.25B-6H and 13S-0.5B-6H.

The fundamental time period is also inversely proportional to the level where the setback occurs. The building has a longer time period if the setback occurs at a lower level, as shown in the cases of 13S-0.5-3H and 13S-0.5B-6H.

When comparing the approximate fundamental period (Ta) from Equation (1.1) with that calculated from the model analysis and Rayleigh method, the fundamental period values for all building cases with the same number of floors, whether regular or with setbacks, is the same. To clarify, there are 6 cases of geometric vertical irregularities in the 13-story buildings in addition to the regular building. According to the code, these seven cases have the same fundamental time period. So, buildings of the same total height can have different fundamental periods, demonstrating that the fundamental period does not only depend on the building's overall height. This is in agreement with the previous studies discussed in chapter one of this study (Satish Paudel et al.; Patel and Takkalaki; Alam and Parakash).

The presented results found that the values of the fundamental time period of the model analysis and the Rayleigh method are quite similar, as the Rayleigh method is based on a theoretical equation, which is the closest to the actual reality. In addition to the model analysis representing building modelling as close as possible to reality. While the code formula is empirical, this was explained earlier in chapter one. Accordingly, it can be said that the ASCE 7:16 code always underestimates the fundamental period for buildings with the setback. Nevertheless, when comparing the results with taking into account the values of the coefficient Cu, which is equal to 1.6 in this study (from Table 2.6), the fundamental period of the regular buildings according to the code equation (Eq 1.1) is equal to the fundamental period of the modal. However, the fundamental period for setback buildings becomes lesser than the modal results.

Incorrectly estimating the fundamental period of the building affects other dynamic response parameters, the most important of which are the seismic base shear, the inter-story distribution of shear force and story drift. Hence, it is expected that the base shear values using LES will be greater than those from MRS and THA. This leads to incorrect and uneconomical design, and thus the prediction of the seismic response of the structure is unrealistic.

3.2.2 Seismic base shear for setback structures

The total base shear is a ratio of the seismic weight of a building and is found by multiplying the seismic weight by the coefficient of seismic response. Basic shear depends on the stiffness and mass of the buildings.

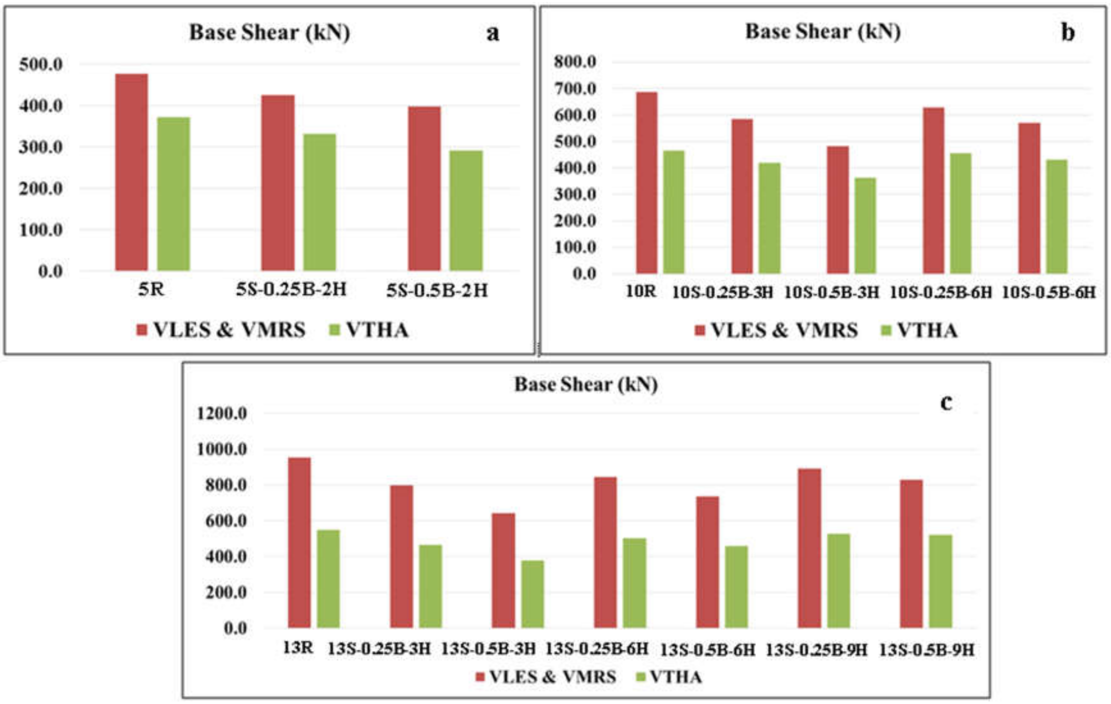
The analysis of the structures was carried out using three methods: LES, MRS, and THA. These methods were previously explained in chapter two.

The following figure shows the base shear for each model in this study. Generally, the base shear for setback structures is less than the corresponding regular structures due to the lower seismic weight. The base shear decreases when the setback ratio increases at the same floor level. It can be observed directly from figure 3-2 (a) for the 5-story structures. To illustrate, the value of the base shear was equal to 476.1 kN when the building was regular. When the setback occurred by 25% on the second floor, the base shear decreased to 425.6 kN, and when the setback ratio increased to 50%, the base shear dropped to 397.4 kN, representing 16.5 % of the base shear of a regular building. (These base shear from the LES method).

In terms of the level of setback, the base shear reduces as the height at which the setback occurred lowers. For example, in the 10-story buildings, as shown in figure 3-2 (b), the value of the base shear was equal to 686.6 kN for the regular building. When the setback occurred at the third floor by 50% (10S-0.5B-3H), the base shear dropped to 481.5 kN. The same setback ratio occurred on the sixth floor (10S-0.5B-6H), and the base shear equalled 569.5 kN. This means that the base shear increased with the setback's height.

Figure 3.2

Base shear comparison for (a) 5-story buildings, (b) 10-story buildings, and (c) 13-story buildings.



As mentioned earlier, most design codes recommend scaling the base shear from MRS to become equal to the base shear from LES analysis so the base shear of these two methods becomes equal. It should be mentioned that the MRS method gives lower base shear values than the LES.

With regard to THA, this method gives lower values of base shear compared to LES (the value of the base shear of THA was calculated from the mean of all results from the ground motion records). For instance, in the case of 5S-0.5B-2H, as shown in figure 3-2 (a), the base shear from the LES method is 397.4 kN, while the THA method is 292.1 kN. That is, it decreased by 26.5%.

The difference between the values of base shear from the LES and THA methods is acceptable up to the 10-story, but in 13-story buildings, there is a significant difference. For example, the base shear for 13S-0.25B-3H from the LES method is 798.4 kN. For the same case, the base shear from the THA method is 463.7 kN, equivalent to 42% of the base shear by the LES method.

Lower base shear values were obtained using the THA approach compared with the LES method. This confirms that the value of the fundamental time period provided by the code needs to be reconsidered, especially when the building is vertically irregular because many parameters affect the fundamental period, such as the setback ratio and the level of the setback. Hence, accurate estimation of the fundamental time period can significantly decrease the seismic design loads on the building, resulting in more economical designs.

3.2.3 Inter-story distribution of base shear

ASCE 7:16 gives a specific formula to distribute the base shear inter-story of a structure. Vertical distribution force is a function of Cvx and base shear (Eq 2.6). The formula of Cvx is equal to the weight of a one-story multiplied by the height of this story from the ground raised to the power of k; all of that is divided by the sum of the multiplication of height and weight for all stories (Eq 2.7). Cvx is the ratio that must be multiplied by base shear to calculate the seismic force at each story, as shown in Chapter two. These equations are as the following:

$$F_X = C_{vx} V \qquad 2.6$$

$$C_{vx} = \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k} \qquad 2.7$$

The following figures show the shear force distribution for 10 and 13 stories cases. For the results of 5-story buildings (See appendix F). The values are normalized to the base shear value from the LES. This is to facilitate a comparison between the results and to make sense of the proximity of the values to the LES method. For all figures, the x-axis represents V/V_{LES}, and the y-axis represents the story level.

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Figure 3.3

Distribution of story shear for 10-story buildings

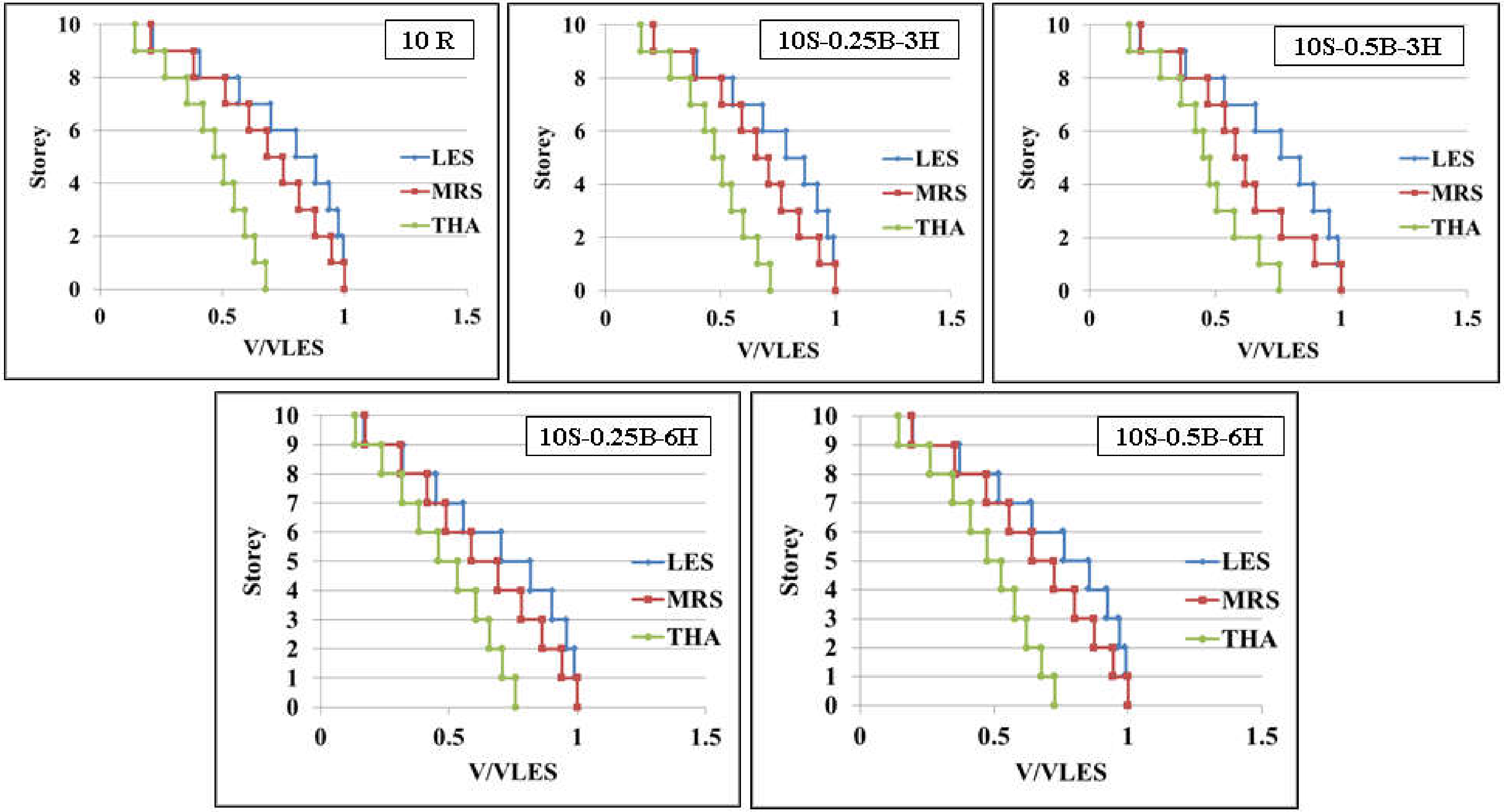
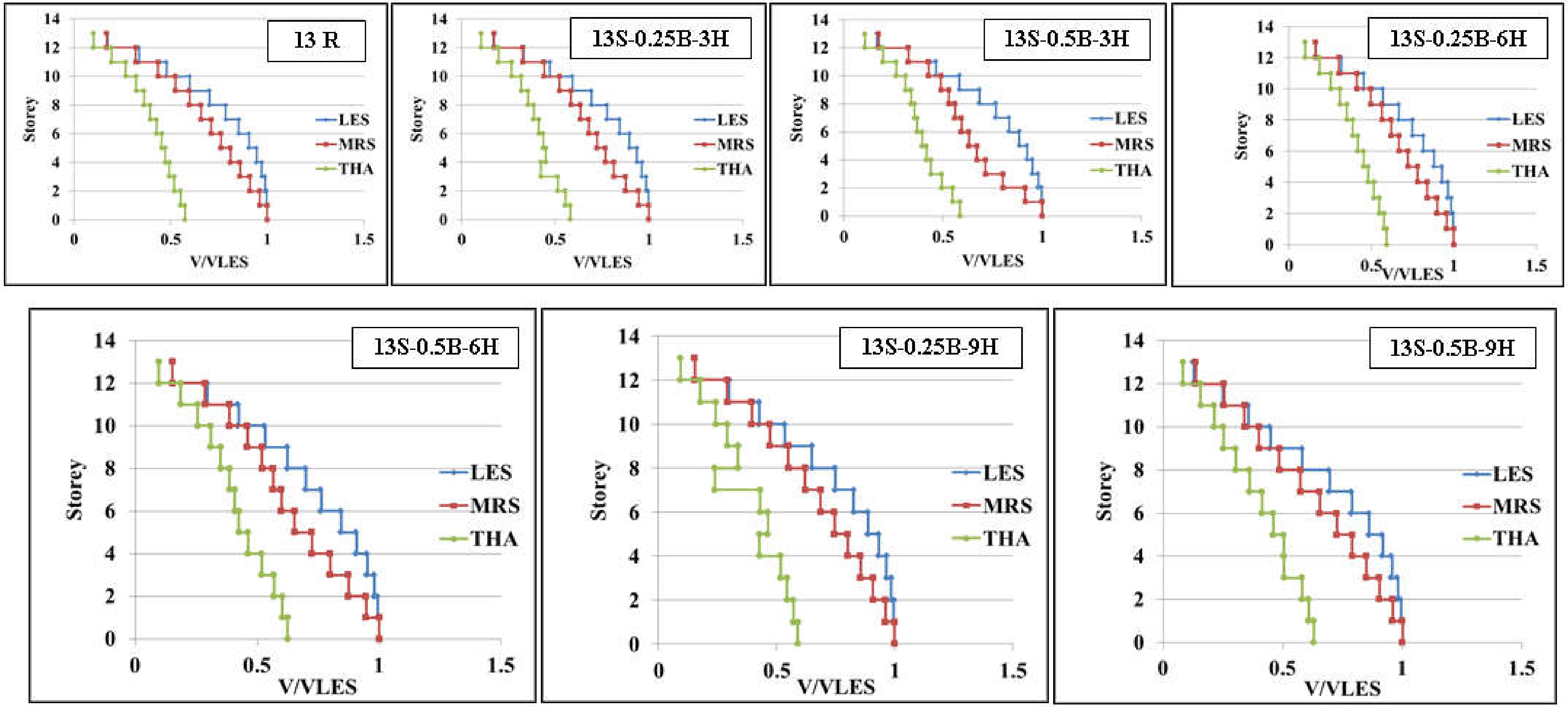


Figure 3.4

Distribution of story shear for 13-story buildings.



The previous figures show a disparity in the distribution of shear forces for the same case among the three methods used. The LES method gives small values of forces on the lower floors compared to the upper floors, in which the forces increase significantly.

The MRS method is superior to LES because it more accurately represents the effect of the higher modes and the real force distribution in the elastic range.

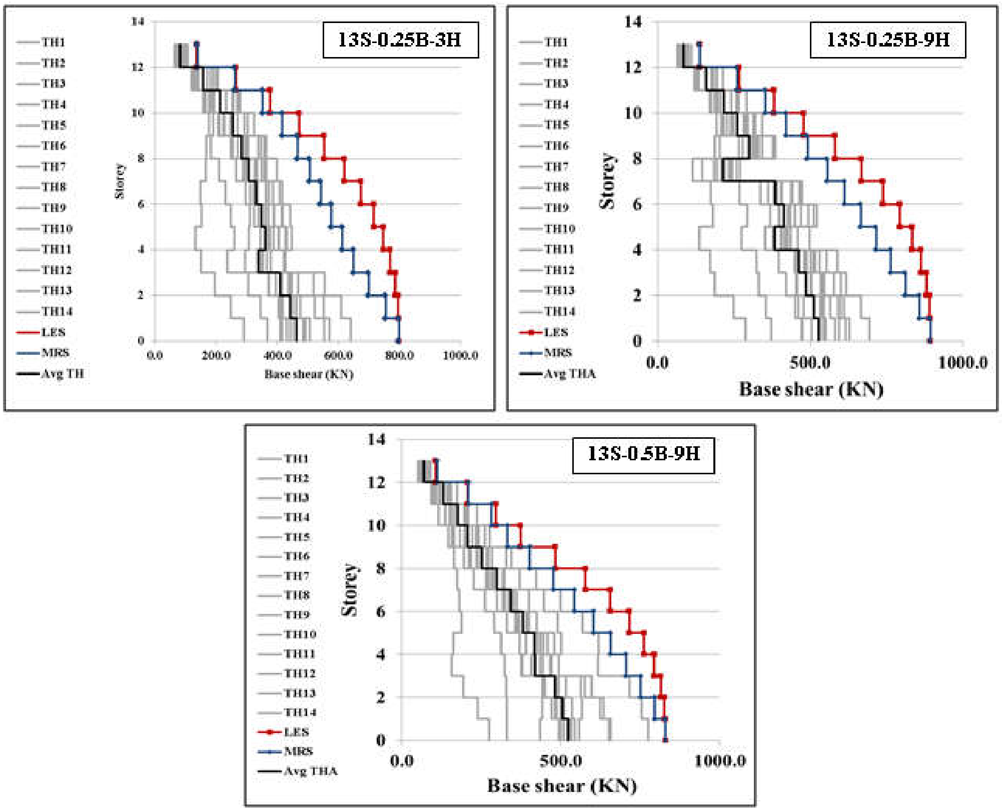
The THA method gives a complex distribution of forces for the cases of the 13-storey buildings, especially in the cases of large setbacks (special cases). These cases are 13S-0.25B-3H, 3S-0.25B-9H, and 13S-0.5B-9H. There is difficulty in tracing the resulting distribution of these cases from the THA method. For this reason, the 13-storey buildings were excluded from modifying the distribution of forces for the LES method based on the THA method.

The three methods reflect the cases in which the setback occurred clearly, as the story shear suddenly decreases at the level at which the setback occurred. For example, in the case of 10S-0.5B-3H, as shown in figure 3-4, the story shear at the third floor was 0.95 from the LES method, 0.76 from the MRS method, and it was 0.58 from the THA method. Because of the setback, the story shear on the next floor decreased to 0.84, 0.66, and 0.5, respectively. The sudden decrease in the vertical distribution of forces at the setback level means this.

Referring to the vertical distribution of forces for the 13-storey buildings resulting from the THA method. Three cases were mentioned in which the distribution is complex and difficult to trace. To confirm the validity of these results of special cases, the distribution of forces from all the records of ground motion used in this thesis was reviewed in Figure 3-5.

Figure 3.5

Distribution of story shear for special cases from all records of ground motions used in this study.



The grey lines in figure 3-5 indicate the distribution of ground motion records, while the black lines indicate the average of these records, presented in the previous results, compared to the LES and the MRS.

From figure 3-5, each recording gives a different distribution of forces from the other for the same case. All these recordings indicate the presence of setbacks in the building, as the distribution decreases abruptly at the setback level.

In the first case, 13S-0.25B-3H, the distribution on the third floor was 0.42. On the floor following the setback (fourth floor), the distribution was 0.45. That is, the distribution increased by 6.67%. This is in contrast to the previous results, in which a sudden decrease occurs in the distribution at the level of setback.

The second case is 13S-0.25B-9H. There was a decrease in the distribution at the fourth level, followed by an increase at the fifth level. The distribution stabilized until the sixth level, then a sharp decrease at the seventh level, although there was no irregularity on these floors. The distribution at the ninth level was 0.34. At the level following the setback (tenth level), the distribution was 0.29. Here, the percentage dropped by 14.7%.

This requires further studies on tall buildings and their distribution of forces based on the THA method.

The third case is 13S-0.5B-9H. The distribution at the ninth level was 0.30, and the distribution at the tenth level was 0.25. This decrease is expected and acceptable due to the occurrence of setbacks. While the behaviour in the lower levels was abnormal, the reason for this is that there is a decrease in the distribution on the third floor, and then the same distribution continues until the fifth floor, which is equal to 0.5, even though there is no irregularity in these floors.

The same earthquake recording gives a different force distribution for each case of irregularity according to the setback ratio and the setback level. These buildings are classified as high-rise buildings, which need more research and studies to understand their seismic behaviour and performance, especially using the THA method.

3.2.3 Summary

It can be said that the greater the setback ratio that occurs at, the lower levels of the building, the greater impact of the setback buildings on the dynamic response. In terms of fundamental period, the fundamental period of setback structures is less than that determined by ASCE 7:16 code equation (Eq 1.1).

Regarding the base shear of the setback buildings, due to the lower seismic weight, the base shear is lower than that of the regular buildings. The THA method also gives lower values than the LES method, so ASCE 7:16 code is overestimated.

The three methods of analysis differ in terms of force distribution. The LES method is the most conservative. Perhaps it is acceptable in terms of safety for engineers, but it does not reflect the true behaviour of the structure, and its design is uneconomical. So, the following section presents a way to modify the distribution of forces for cases up to 10 stories. The complex distribution of the THA analysis method for 13-storey buildings and the difficulty of tracking the resulting change on some floors excluded their use in modifying the distribution of forces.

3.3 Modification of the distribution of forces for setback building

The LES method will be modified because almost all codes still use the LES methodology and rely on manual computations and theoretical formulae. In addition to the restrictions placed by the ASCE 7:16 on the use of LES in SDC D, E and F, as well as the prohibition of its use for irregular buildings, this adjustment will be made to the distribution of forces to make it applicable in these SDC. This modification is for buildings up to 10 stories depending on the THA. At the same time, the 13-storey buildings were excluded from modification due to the difficulty and complexity of distribution.

The THA was used to give a correct distribution that reflects the actual reality of these buildings. That is by amending the formula of the vertical distribution of seismic forces from the LES method provided by the code. The proposed modification can be achieved by adding a new factor called alpha (α) multiplied by the k factor as follows:

$$C_{vx} = \frac{w_x {h_x}^{\alpha k}}{\sum_{i=1}^{n} w_i {h_i}^{\alpha k}} \qquad 3.1$$

The following are the steps used to o the modified force distribution:

1. The value of the base shear for both the LES and MRS methods is modified to match better with the THA method.
2. An initial value was given to the alpha, and the distribution calculations were performed for LES using the equation of 3.1.
3. Non-linear regression analysis was used to obtain the alpha value, which depends on the sum of squares of errors (SSE) between the LES method distribution and the THA method distribution for each story as the following:

$$SSE = \sum \left(CV_{X,\alpha} \times V_{LES} - CV_{X,THA} \times V_{THA} \right)^{2} \qquad 3.2$$

Where $CVx,_{THA} = (Vi / V,_{total})_{THA}$

4. The SSE was minimized by changing alpha.
5. The final alpha values for each case were obtained using Solver.
6. The inter-story forces were redistributed from the LES with alpha, also from MRS and THA.

The following table shows the final alpha values obtained using Solver for each case.

Table 3.1

Alpha values after modification.

Case	α
5 R	0.617
5S-0.25B-2H	0.790
5S-0.5B-2H	0.824
10 R	0.587
10S-0.25B-3H	0.567
10S-0.5B-3H	0.470

10S-0.25B-6H	0.610
10S-0.5B-6H	0.674

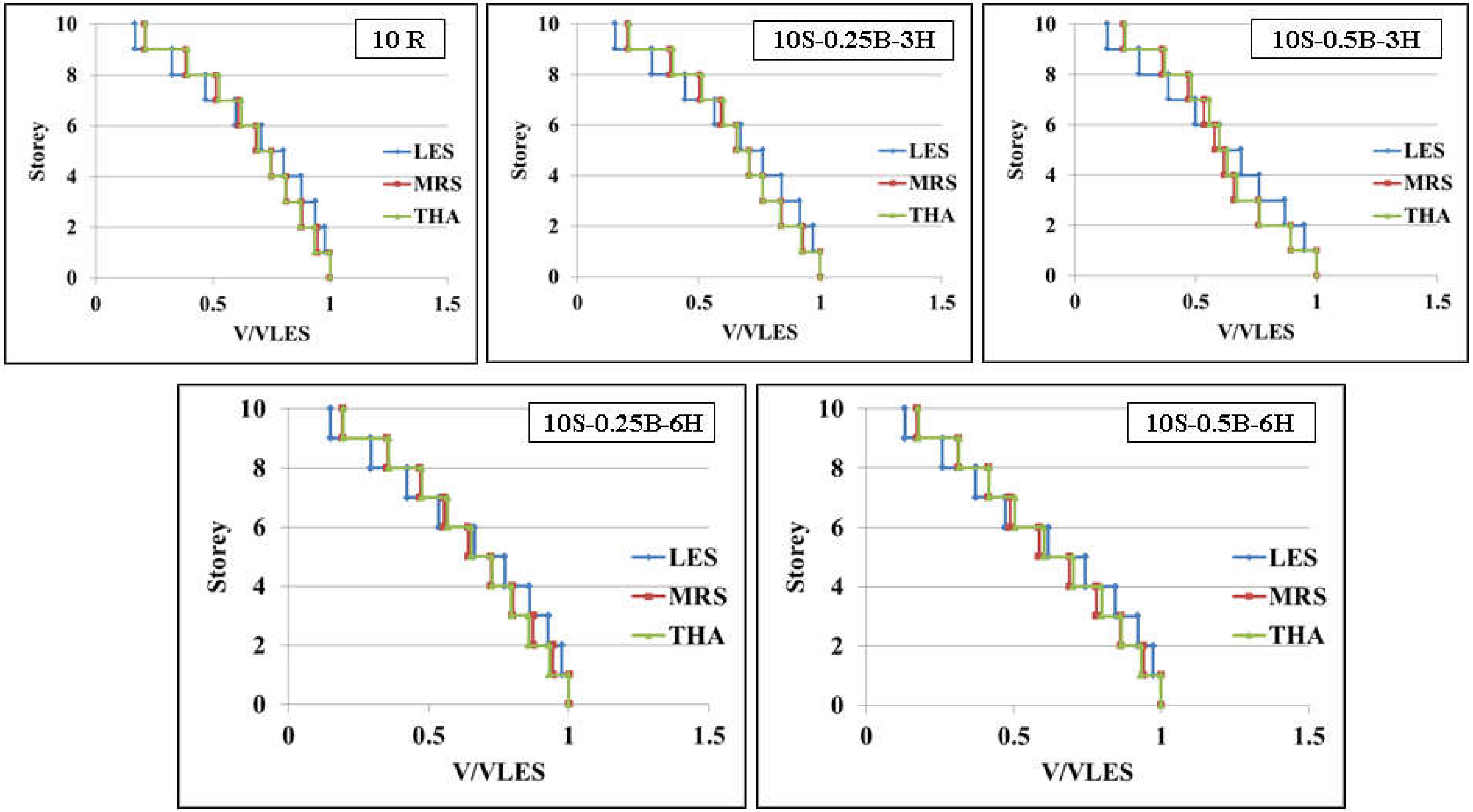
(See appendix I) to illustrate the relative error between the coefficient of the vertical distribution (CVx) from the LES method after modification and from the THA method. CVx for the THA was determined by dividing the story shear by the total base shear.

CVx obtained from the alpha for the LES method based on the CVx from the THA method for 5-story buildings are within a relative error that does not exceed 10%, which is acceptable, as shown in figure (a). For buildings consisting of 10 stories, as shown in figure (b), the percentage of error for the upper floors reaches 30%, and this is expected, as it was previously explained that the LES method gives a high distribution of the upper floors, which makes the distribution difference compares to the THA method. The percentage error for the lower floors does not exceed 15%.

The following figures is the new distribution of the forces after modification for 10-story buildings, where there is a great match of the three methods. For the results of 5-story buildings (See appendix F).

Figure 3.6

Distribution of story shear for 10-story buildings after modification with variable alpha.



3.3.1 Sensitivity study

It is shown in table 3.2 that the change in the alpha values is small, as the values are close to each other, so the average values were adopted to facilitate and simplify the adjustment process. Later in the following sections, the alpha value will be validated and approved as a constant value.

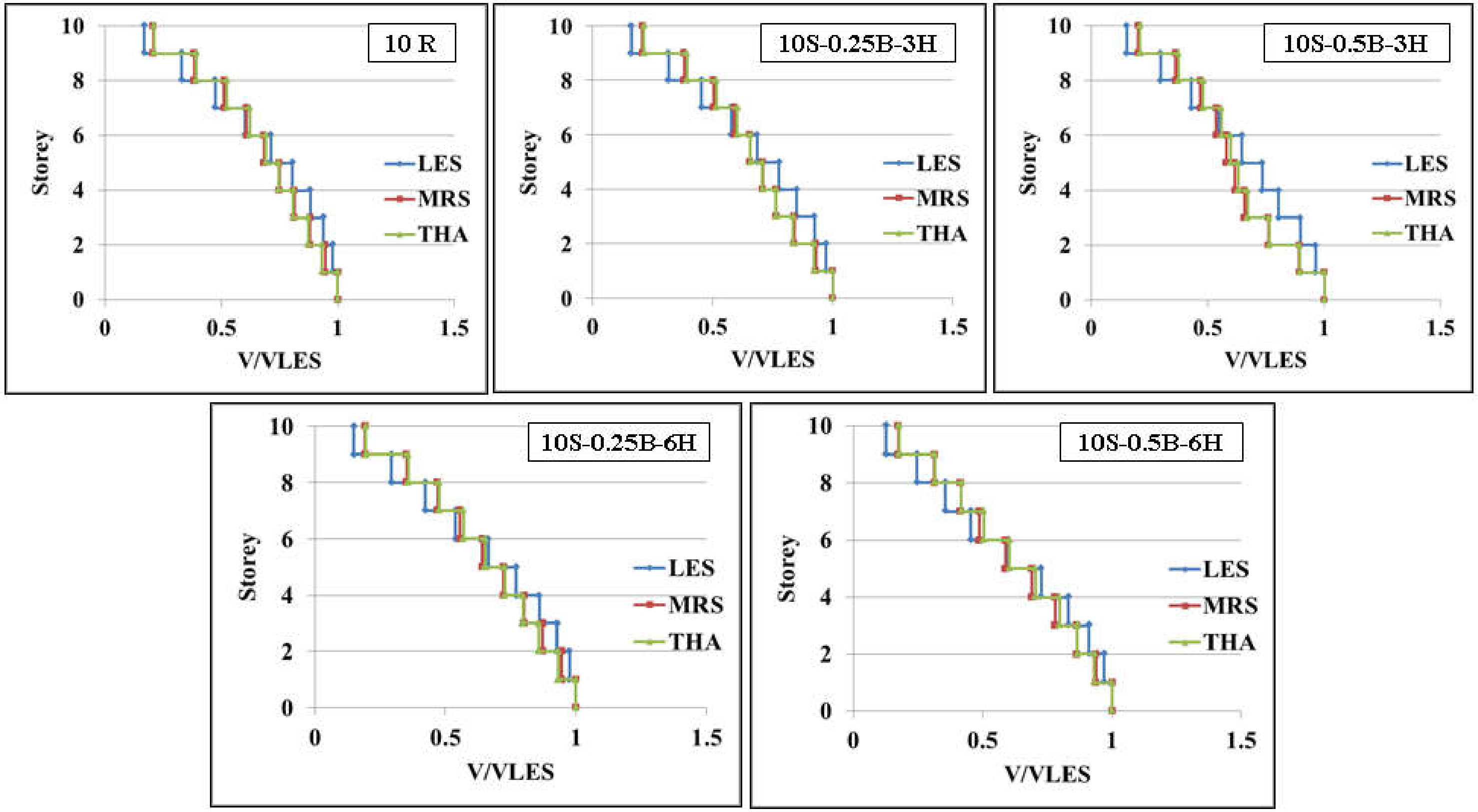
It is shown in table 3.2 that the alpha values are close to each other. One alpha value has been adopted to facilitate and simplify the adjustment process for the vertical forces distribution, which is the average of all values. Note that there are values that are far from the average, which are 5S-0.5B-2H and 10S-0.5B-3H, as these cases are considered to have a major setback, as the setback ratio is large and occurs in the lower levels of the building.

Then the forces were redistributed based on this value, which equals 0.64, and the results of 10-story cases were as follows in figure 3-9. For the results of 5-story buildings (See appendix F).

It should be noted that the adopted alpha value is valid only for the cases modelled in this study, on which the modification process was carried out. There is a need for more data and research regarding alpha value to perform statistical operations to reach one value for an alpha suitable for all setback cases that occur in one direction.

Figure 3.7

Distribution of story shear for 10-story buildings with constant alpha.



The LES method gives a higher distribution of forces than other methods up to the sixth story. The distribution becomes less on the upper floors by a small percentage. In the case of 10S-0.5B-3H, the distribution at the third level was 0.9 from the LES, while the THA and MRS method was 0.76. Giving higher values in the distribution from the LES method than other methods means that the approved alpha value is safe and correct.

It is noticeable that the distribution of forces from the LES method is less noticeable on the upper floors. The critical case is 10S-0.25B-3H, as the distribution in the upper floors, specifically on the eighth floor, was 0.316 from the LES, and the THA was 0.39, meaning that the distribution from the LES decreases by 19% from the THA method.

Generally, the alpha value of 0.64 for buildings up to 10 stories with a setback ratio of up to 50% gives an acceptable distribution using the LES method.

3.3.2 Story drift

Story drift is the displacement difference between two adjacent floors divided by the story height. Inter-story drift is an important parameter in the study of the seismic behaviour of the structures. Therefore, the maximum story drift results will be presented for two purposes. The first is to ensure the validity of the obtained alpha value by comparing the results of the story drift between the LES method and THA method before and after the adopted alpha. The second purpose is to compare regular buildings' behaviour with those of setback buildings.

(See appendix G and J) for the figures of the maximum story drift results for all the recordings used in the analysis, the average values from these recordings, and the maximum story drift values from the LES method before and after the distribution modification for 5-story buildings and 10-story buildings respectively.

The previous figure shows that the story drift of the regular building is higher compared to the setback buildings from the LES and THA methods. Generally, the drift of the lower floors is higher than the upper floors. This may be due to the decrease in the mass of the building when setbacks occur.

In addition, the story drift increases abruptly at the level above the setback. This can be attributed to the reduction of the stiffness that occurs at the level where the setback occurs. To illustrate, in the case of 10S-0.5B-3H, the story maximum drifts for the third level from the LES method before modification was equal to 1.296 mm, while it decreased to 0.915 mm after modification. Compared with the THA method, the story maximum drift was equal to 0.732 mm, the average of all recordings used in this study.

In contrast, the maximum story drifts for the fourth level from the LES method before modification was equal to 2.1 mm. The increase was 38.3% over the third level. The story maximum drift after modification dropped to 1.414 mm after modification. Compared with the THA method, the maximum story drift equals 1.1 mm.

This means the LES method still gives a higher story drift in both cases before and after the modification.

Also, the story drift increases with an increase in setback ratio, where in the case of 10S-0.25B-6H, the story maximum drift was 1.4 mm from the LES before modification. After adding alpha, the maximum story drift becomes 0.858 mm. The story maximum drift was 0.842 mm from the THA method. When the setback ratio increases to 50 % at the same level, 10S-0.5B-6H, the story maximum drift increases to 1.567 mm, 0.974 mm, and 0.953 mm, respectively.

The LES method gives story drift values higher than the average values from the THA analysis before modification. To ensure the structure's integrity and that the obtained alpha value is safe, the LES method must still give drift values higher than or equal to the average values from the THA after modification, which was achieved as shown in the previous figures.

3.4 Verification

For improved reliability, a validation will be carried out by modelling a new building with different characteristics in addition to the story drift check to confirm the obtained alpha value. An 8-story building consisting of 3 bays with a width of 5 m has been modelled. It has a setback of 0.33 at the fourth level. The columns are square with dimensions of (45 * 45) cm, and the beams' dimensions of (40 * 45) cm. The slab is solid, with a thickness of 20 cm.

The value of the period obtained from the modal analysis was 1.309 s, while it was 0.814 s from the code equation (Eq 1.1). The base shear value obtained from the LES method was equal to 402 kN, and the average result of the THA method was 321.43 kN. For the results of the force distribution using the three analysis methods before and after using alpha, (See appendix G).

3.5 The results of the modelled case without masonry walls

By modelling and analyzing this case, which is 10S-0.25B-2H without a masonry wall, the value of the fundamental time period resulting from the modal analysis was equal to 1.891 sec in the setback direction, while it was equal to 1.885 sec from the Raleigh method, and it was equal to 1 sec from the code equation (Eq 1.1). The resulting base shear from LES and MRS was 644 kN, and from THA was 319 kN.
(See appendix H) for the results of this case. Where Figure H-1(a) shows the vertical distribution of the force based on the original results, figure H-1(b) shows the vertical distribution of the force by using α for this case, which is equal to 0.6107, and figure H-1(c) shows the vertical distribution of the force by using constant α, which equals of 0.64 using the alpha used for modified cases. At the same time, figure H-1(d) shows the maximum story drift.

Although this case was modelled according to the requirements of the SMRF, and despite neglecting the masonry walls as well, and column ends were fixed at the foundation, the results indicate the same behaviour shown in the previous cases of the 10-storey building. There is still a clear difference in the values of the fundamental time period between the code equation and the modal analysis, and the difference even increased with neglecting the masonry walls due to the decrease in stiffness. Also, the difference between the results of the base shear and their vertical distribution remained large, and the same previous behaviour was observed.

Based on the results, it can be said that the complete design of the SMRF and the fulfilment of its requirements do not affect the behaviour of setback buildings as a distribution of shear force and as maximum story drift. The presence of masonry walls is the closest to realistic modelling, but neglecting their results in the same behaviour for setback buildings. The existence of these walls affects results as values, not as behaviour.

Chapter Four

Conclusions and Future work

4.1 Overview

In this study, the effect of setback buildings on seismic response parameters was studied, which are represented in the fundamental time period of the structure, seismic base shear, the inter-story distribution of forces, and the story drift of the structure using the three analysis methods, which are LES, MRS, and THA.

It was modifying the method of the inter-story distribution of forces resulting from the LES method, depending on the results of the THA method. The reason for this modification is that most of the design codes, including ASCE 7:16, set restrictions on the use of the LES method, as it prevented its use for some types of irregularities, including setback buildings, in seismic SDC D, E and F.

To achieve the study's desired aim, a set of buildings were modelled with perimeter masonry infill walls. The heights of the selected buildings range from 5 to 13 floors. The selected buildings were designed and analyzed according to ASCE 7:16 and ACI 318-14 using the commercial software ETABS 2016.

The following sections present the main results obtained from this study, a conclusion of the study, as well as future work related to this study.

4.2 Research results

The main results will be presented at two levels so that the first level includes the effect of setback irregularity on the dynamic response factors. The second level includes a comparison between the methods of analysis used in this study.

4.2.1 The effect of setbacks on the dynamic response

Based on the analytical study and the presented work, the outcomes can be summarized as the following:

1. The fundamental period for setback structures is always lower compared to regular buildings. The setback ratio is inversely proportional to the time period for the same story, so the higher the setback ratio, the lower the time period.

2. The setback buildings with the same total heights can differ in the time period value, which means that the fundamental period is not a function of the total height of the building according to the ASCE 7:16 equation. This was indicated by most of the previous studies based on different design codes.
3. The code is conservative in estimating the values of the fundamental period of the structure since it always gives lower values for the time period than the modal analysis and Rayleigh method.
4. Setback buildings' base shear is less than regular buildings due to decreased seismic weight at the setback level.
5. In the vertical distribution of force, there is a sudden decrease in the value of the force at the setback level.
6. Setback buildings have less story drift than regular buildings, and the drift on the lower floors is greater than on the upper floors due to the reduction in mass at the setback floor.
7. In general, the story's drift increases with the setback ratio. Also, there is a sudden increase in the drift values at the setback level due to a decrease in the stiffness and seismic weight.

4.2.2 A comparison of analysis methods

1. The LES method gives a higher base shear than the MRS and THA methods, which means that the code is also conservative in estimating the base shear.

2. The LES method gives a higher force distribution than other methods, specifically on the upper floors. The MRS method is more realistic in force distribution than the LES because it represents the effect of the higher modes and the real force distribution in the elastic range.

3. The distribution of forces using the method of THA is really up to a 10-story; there are cases in 13-storey buildings in which the distribution of forces is complex and difficult to trace.

4. The LES gives a higher value for the story drift than the average values from the THA.

4.3 Proposed modification to base shear distribution

Solving the problem of the restriction on the use of the LES method in estimating the base shear and the vertical distribution of forces is important. This gives an acceptable method and predicts the seismic response parameters realistically and correctly. The modification was done based on the THA method, which uses real ground motion records.

The proposed modification was done by adding a new factor called alpha (α) multiplied by the k factor to the formula of the vertical distribution of forces as follows:

$$C_{vx} = \frac{w_x {h_x}^{\alpha k}}{\sum_{i=1}^{n} w_i {h_i}^{\alpha k}} \qquad 3.1$$

Nonlinear regression analysis was used to obtain the alpha values used in modifying process. Then the average values of the alpha results were calculated and found to be 0.64, and they were used for the modified cases.

In order to ensure the safety of the resulting alpha value, the values of the maximum story drift were presented from the LES method before and after the modification, in addition to the THA method. It was confirmed that the LES method gives drift values higher than or equal to the average of the story drift from THA. In addition to modelling a new building with different characteristics to confirm the validity of the results.

In this way, an effective solution was presented to the problem of the use of the LES analysis for setback buildings in SDC D, E and F because the modification was made based on the THA method.

In addition, it solves the LES problem, which is a very conservative method and gives values greater than their actual reality. This way, the design is economical, and the seismic response of the structure can be predicted correctly.

4.3.1 A Limitations on the use of the proposed solution

The proposed solution is used in the modification for cases that meet the following conditions:

1. The building height ranges from 5 to 10 floors, with a floor height of 3 m.
2. The number of bays is 4 with a width of 4 m.
3. The setback ratio does not exceed 50% of the number of bays.
4. The height of the building to its width (H/B) is from (1 - 1.8).

For alpha to be appropriate for any case of setback buildings, more data and statistical analysis are required before it can be adopted as a constant value.

4.4 Future work

The following are ideas and suggestions for further study and research:

1. Studying the effect of setback building on seismic response parameters using non-linear dynamic analysis (NLTHA).
2. This study is limited to studying the effect of a setback on the dynamic response of SMRF structures. The researcher recommends working on other types of frames.
3. The researcher recommends working on more alpha values in the modification process.
4. Study other cases of vertical geometric irregularity in buildings, such as buildings that have setbacks in both directions with different heights and setback ratios.
5. The researcher recommends using the THA method to study high-rise buildings with setbacks.
6. Consider the effect of soil structure interaction in future studies.
7. The researcher recommends working on other cases of horizontal and vertical irregularity.

List of Abbreviations

Abbreviation	Meaning
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
CQC	Complete Quadratic Combination
DL	Dead Load
EC 8	European Code
ELF	Equivalent Lateral Force
FEMA	Federal Emergency Management Agency
IBC	International Building Code
IS1893	Indian Standard
LES	Linear Equivalent Static
LL	Live Load
MRF	Moment Resisting Frame
MRS	Modal Response Spectrum
PEER	Pacific Earthquake Engineering Research
PGA	Peak Ground Acceleration
RC	Reinforced Concrete
SDC	Seismic Design Categories
SID	Superimposed Dead Load
SRSS	Square Root of The Sum of Squares
SMRF	Special Moment Resisting Frame
THA	Time History Analysis
UBC	Uniform Building Code

References

ACI318. (2014). *Building Code Requirements for Structural Concrete (ACI 318M-14): an ACI standard: Commentary on building code requirement for structural concrete*. American Concrete Institute.

Alam, F., & Prakash, S. (2017). EFFECT OF SETBACK ON FUNDAMANTAL PERIOD OF RC FRAMED BUILDINGS. *INTERNATIONAL JOURNAL OF ENGINEERING SCIENCES & RESEARCH TECHNOLOGY*, *6*(10).

ASCE7. (2016). *Minimum design loads for buildings and other structures*. American Society of Civil Engineers.

Bekele, B. N. (2022). Effect Of Setback Irregularity On Elastic Seismic Demand Of Reinforced Concrete Building. *Journal of University of Shanghai for Science and Technology*, *24*(02), 182-193.

ETABS. (2016). *Computers and Structures, Inc. Integrated software for structural analysis and design*. In https://www.csiamerica.com/products/etabs

Eurocode8. (2004). *Design of structures for earthquake resistance, part-1: general rules, seismic actions and rules for buildings*. European Committee for Standardization (CEN).

FEMA356. (2002). *Prestandard and commentary for the seismic rehabilitation of buildings*. Federal Emergency Management Agency (FEMA).

Goel, R. K., & Chopra, A. K. (1997). Period formulas for moment-resisting frame buildings. *Journal of Structural Engineering*, *123*(11), 1454.

Heo, Y., Kunnath, S. K., & Abrahamson, N. (2011). Amplitude-scaled versus spectrum-matched ground motions for seismic performance assessment. *Journal of Structural Engineering*, *137*(3), 278-288.

IBC. (2018). *2018 IBC: International Building Code*. ICC, International Code Council, Incorporated.

IS1893. (2016). *Indian Standard Criteria for Earthquake Resistant Design of Structures Code of Practice*. Bureau of Indian Standards.

Jardaneh, I. G. (2004). Evaluation of Ground Response Due to Earthquakes-Case Study. *Journal of Applied Sciences*, *4*(3), 364-368.

JC. (2006). *Jordanian code for loads and forces*. Building Research Center for Ministry of Public Works and Housing.

Khaleeluddin, M., & Azeem, M. (2017). Effect of Setback on Fundamental Period of RC Frame Buildings with Brick Infills.

Kumar, & Manjunath. (2018). Seismic analysis of vertically geometric irregular structures. *International Journal of Scientific Development and Research (IJSDR)*, *3*(5), 510-516.

Malkawi, A., Al-Homoud, A., & Liang, R. (1995). Seismic hazard mapping of Jordan. *Quarterly Journal of Engineering Geology*, *28*, 75-81.

Patel, P., & Takkalaki, S. R. (2019). Effect of Setback on Fundamental Period of RC Framed Buildings. *International Journal of Research in Engineering, Science and Management*, *2*(11), 283-286.

PEER. (2022). *Pacific Earthquake engineering Research centre, ground motion database.* https://peer.berkeley.edu/

Rana, D., & Raheem, J. (2015). Seismic Analysis of Regular & Vertical Geometric Irregular RCC Framed Building. *International Research Journal of Engineering and Technology*, *2*(04), 2395-0056.

Saraswathy, B., Udaya, K., & Rahul, L. (2014). Effect of vertical irregularity on performance of reinforced concrete framed buildings. Proceedings of the Second International Conference on Advances in Civil, Structural and Environmental Engineering,

Sarkar, P., Prasad, A. M., & Menon, D. (2010). Vertical geometric irregularity in stepped building frames. *Engineering Structures*, *32*(8), 2175-2182.

Satish Paudel, Aayusha Pradhananga, Aayusha Khadka, Manjila Shrestha, & Rabina Bista. (2019). Effect of setback on fundamental period of RC Structures. *International Conference on Earthquake Engineering and Post Disaster Reconstruction Planning*, 61-70.

Sayyed, O., Kushwah, S. S., & Rawat, A. (2017). Seismic analysis of vertical irregular RC building with stiffness and setback irregularities. *IOSR Journal of Mechanical and Civil Engineering*, *14*(01), 40-45.

Thakur, V. S., & Rai, R. (2018). EFFECT OF SETBACK ON FUNDAMANTAL PERIOD OF RC FRAMED BUILDINGS.

Thejaswini, Govindaraju, & Devaraj. (2020). Dynamic Response of Symmetric and Asymmetric Setback Buildings. *International Journal of Innovative Technology and Exploring Engineering (IJITEE)*, *9*(4), 3197-3183.

UBC. (1997). *Uniform Building Code*. International Conference of Building Officials.

Uday Neelam Smt, & Bajaj, R. (2017). Effect of Setback on Fundamental Period of RC Framed Buildings. *IJSRD - International Journal for Scientific Research & Development*, *5*(08), 349-351.

Varadharajan, S., Sehgal, V., & Saini, B. (2014). Fundamental time period of RC Setback Buildings. *Concrete Research Letters*, *5*(4), 901-935.

Appendices

Appendix A

Determination the superimposed dead load (SID)

The densities of typical building materials according to Jordanian code (JC, 2006) for forces and loads are shown in Table A.1.

Table A. 1

Densities of typical materials used in Palestine

Material type	Density (kN/m^3)
Tiles	24
Reinforced concrete	25
Plastering	22
Mortars	22
Fill materials (fine aggregate)	18

The following are the most commonly thicknesses for materials covered by the slab:

- The thickness of tile: 3 cm
- The thickness of mortar: 2 cm
- fills under the tiles: 10 cm
- The thickness of plaster: 1.5 cm

The SID determining as the following:

$$\mathrm{SID} = 0.03 \times 24 + 0.02 \times 22 + 0.1 \times 18 + 0.015 \times 22$$

$$\therefore \mathrm{SID} = 3.29\ \mathrm{kN/m^2} \rightarrow \text{use } 4\ \mathrm{kN/m^2}$$

Appendix B

Check the sizes of structural elements thickness

Appendix B- 1: Check for the slab thickness

According to Table 8.3.1.2 in ACI318-14, if the average value of α_f for all beams on edges of a panel ≤ 0.2, then Table 8.3.1.1 is applied. Where α_f is α_f the ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerline panels, if any, on each side of the beam.

According to Table 8.3.1.1 the minimum thickness for the slab with fy = 420 Mpa, no drop panels with edges beams is equal to $\frac{l_n}{30}$, $\frac{l_n}{33}$ for exterior panels and for interior panels, respectively. The thickness shall be increased by 10 percent.

In all models, the critical case is equal to 4 m for exterior panels:

$$h_{min} = 1.1 \times \frac{l_n}{30} = 1.1 \times \frac{4}{30} = 0.147\ m$$

The slab thickness of 0.2m is acceptable.

Appendix B- 2: Check the columns sections

Table B.1 shows the ultimate self-weight for the components of the structures located inside the tributary area for one story of a 5-Story building. Table B.2 shows the ultimate weight of distributed load across sections of the required and available columns.

Table B. 1

The ultimate self-weight for the structural components located inside the tributary area for a 5-Story building.

Type of element	Load factor	γ_c (kN/m^3)	Dimension (m)			Factored weight (kN)
			L	W	h	
Slab	1.2	25	4	4	0.2	95
Column	1.2	25	0.4	0.4	3	14.4
					∑	109.5

Table B. 2:

The ultimate weight of distributed load across the tributary area for one story of 5-Story building.

Load Pattern	Load factor	Distributed load (kN/m^2)	Tributary area (m)		Factored weight (kN)
			L	W	
SID	1.2	4	4	4	76.80
LL	1.6	2	4	4	51.20
				∑	128

So, for one story, the total ultimate load:

For 5-Story = 109.5+128 = 237.5 kN

For 10-Story = 117.5+128 = 245.5 kN

For 13-Story = 127.4+128 = 255.4 kN

Table B. 3:

The final outcomes for the cross sections of the required and provided columns.

Building type	**Total ultimate load (kN)**	**Squared column required (cm)**	**Provided column (cm)**	**Safe or not**
5 R	1187.5	35×35	40×40	safe
10 R	2455	49×49	50×50	safe
13 R	3320.2	58×58	60×60	safe

Appendix C

Calculation of strut width of masonry

Figure C-1 shows the diagonal strut replacement for the infill wall and the following are the calculations of strut width for 5-story:

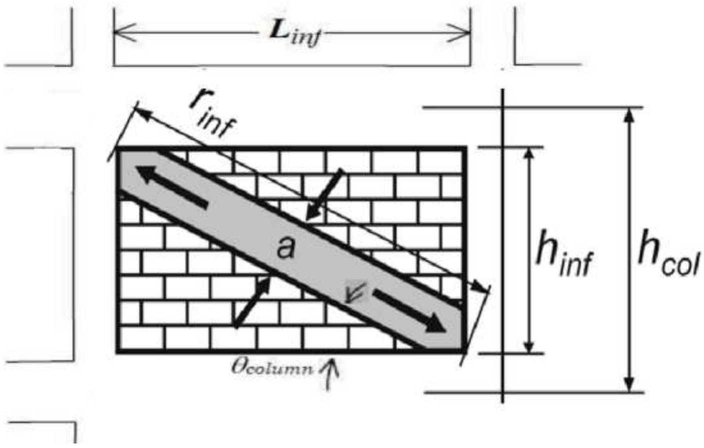
$$\lambda_1 = \left[\frac{0.1 * 23025 * 10^3 * 0.2 * sin35.837}{4 * 23025 * 10^3 * \frac{0.4^4}{12} * 2.6}\right]^{\frac{1}{4}} = 0.961$$

$$\alpha = 0.175(1.71 * 3)^{-0.4} * 4.44 = 508\, mm$$

The same approach was used for other cases, and it was found, $\alpha = 426\, mm$ for 10-story and, $\alpha = 442\, mm$ for 13-story.

Figure C- 1

The diagonal strut replacement for the infill wall



Appendix D

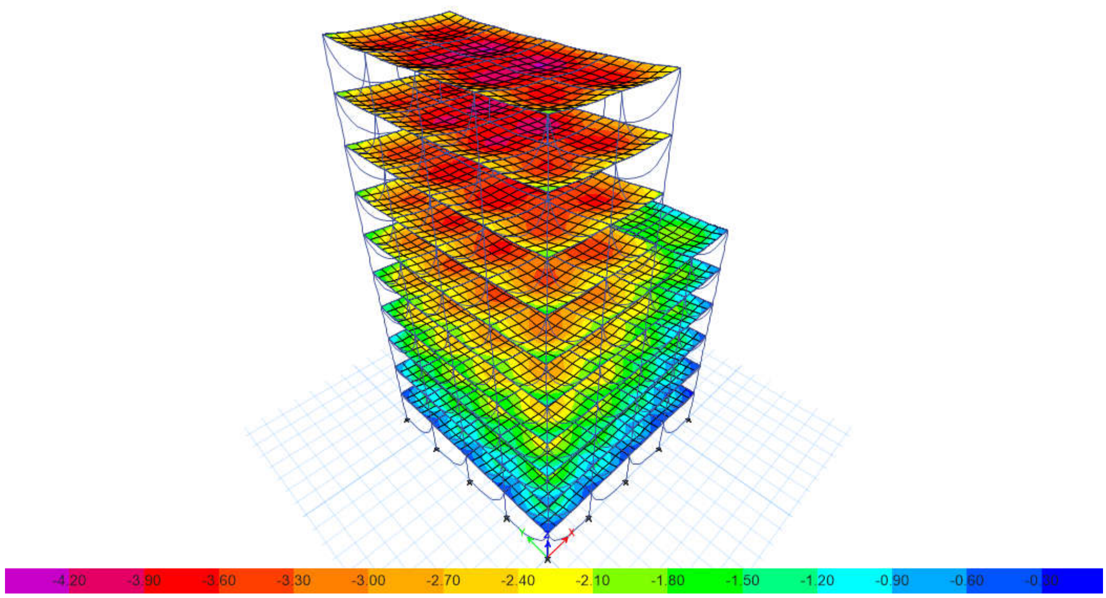
Modal checks

Appendix D- 1: Gravity checks

The necessary checks were carried out for all buildings selected in this research. The results of the checks will be shown for 10S-0.5B-6H Building, as all buildings were done in the same way. The first check is the compatibility check. The building is compatible as all the building elements are connected together as one part as shown in figure D-1.

Figure D- 1

Model deformed shape



The second check is the Equilibrium check. The following table shows the base reaction from ETABS.

Table D. 1

Base reaction for 10S-0.5B-6H model.

Base Reactions

Load Case/Combo	FX kN	FY kN	FZ kN
Dead	0	0	19917.5153
Live	0	0	4096
SID	0	0	17408

<u>Manual Calculations:</u>

- Live Load:

$$\mathrm{L.L} = \text{floor area} \times \text{live load value} = (256 \times 6 + 128 \times 4\ ) \times 2 = 4096\ \mathrm{kN}$$

$$\%\mathrm{Error} = \frac{\text{Etabs value} - \text{Manul value}}{\text{Etabs value}} = \frac{4096 - 4096}{4096} \times 100\% = 0.0\% < 5\%$$

$$\rightarrow \text{Check is OK.}$$

- Superimposed Dead Load:

$$\begin{aligned}\mathrm{SID} &= \text{floor area} \times \text{superimposed value} + \text{beam length} \times \text{superimposed value} \\ &= [(256 \times 6 + 128 \times 4\ ) \times 4] + [(16 \times 4 \times 6) \\ &+ ((16 \times 2 + 8 \times 2) \times 4))] \times 16 = 17408\mathrm{kN}\end{aligned}$$

$$\%\mathrm{Error} = \frac{\text{Etabsvalue} - \text{Manul value}}{\text{Etabs value}} = \frac{17408 - 17408}{174080} \times 100\% = 0.0\% < 5\%$$

$$\rightarrow \text{Check is OK.}$$

- Dead Load

$$\begin{aligned}\text{Columnsdeadload} &= (0.5 \times 0.5 \times 25) \times (18 \times 25 + 12 \times 15) \\ &= 3937.5\mathrm{KN}\end{aligned}$$

$$\text{Beams dead load} = (0.5 \times 0.4 \times 25) \times (144 \times 6 + 77 \times 4) = 5860\mathrm{KN}$$

$$\text{Slab dead load} = 0.2 \times 25 \times (6 \times 16 \times 16 + 8 \times 16 \times 4) = 10240\mathrm{KN}$$

$$\text{Total dead load} = 20037.5\mathrm{KN}$$

$$\%\text{Error} = \frac{\text{Etabs value} - \text{Manul value}}{\text{Etabs value}} = \frac{19917.5153 - 20037.5}{19917.5153} \times 100\% = 0.6\%$$

$$< 5\% \quad \rightarrow \text{Check is OK.}$$

Appendix D- 2: Seismic checks

The necessary seismic checks were also conducted to ensure the safety of the structure. The first check that was performed was the drift check, ASCE 7:16 states that structural design drift shall not exceed the limit of allowable as per Table 12.12-1.

$$\delta_i = \frac{C_d \delta_{ei}}{I_e} \qquad \Delta_i = \delta_i - \delta_{i-1}$$

Where δ_{ei} is elastic displacement, δ_i is amplified displacement (inelastic), and Δ_i is story drift.

Cd = 5.5 I_e = 1 allowable drift factor=0.02 story height (h_{sx}) =3000 mm

Allowable story drift $(\Delta_a) = \text{allowable factor} * h_{sx} = 0.02 * 3000 = 60\ \text{mm}$

The following table shows the results of the drift check for 10S-0.5B-6H model in x direction. All models have been checked in the same way in x and y direction and make sure that the value of the story drift does not exceed the maximum allowed.

Table D. 2

Drift check results for 10S-0.5B-6H model.

Story level	δei elastic (mm)	δi inelastic (mm)	inelastic drift Δ_i (mm)	Allowable inelastic drift Δ_a (mm)	Result
Story 10	13.72	75.46	8.943	60	SAFE
Story 9	12.094	**66.517**	5.885	60	SAFE
Story 8	11.024	**60.632**	7.5955	60	SAFE
Story 7	9.643	53.0365	7.964	60	SAFE
Story 6	8.195	45.0725	5.984	60	SAFE
Story 5	7.107	39.0885	6.556	60	SAFE
Story 4	5.915	32.5325	7.095	60	SAFE
Story 3	4.625	25.4375	7.4305	60	SAFE
Story 2	3.274	18.007	7.755	60	SAFE
Story 1	1.864	10.252	10.252	60	SAFE
Base	0	0	0	60	SAFE

Also, P- delta check was carried out, and it was confirmed that the stability coefficient (ϴ) was less than 0.1 for all floors, and thus the effect of the second-order analysis was neglected. Table C.3 shows the results of P-delta check for 10S-0.5B-6H model. The stability coefficient is determined as the following:

$$\boldsymbol{\theta} = \frac{\mathbf{P_x \Delta I_e}}{\mathbf{V_x h_{sx} C_d}}$$

Where Px is the total vertical design load, Δ is design story drift, and Vx is the seismic shear force.

Table D. 3

the results of P-delta check for 10S-0.5B-6H model.

Story level	Px (KN)	Vx (kN)	δei (mm)	Δ_i (mm)	ϴ	checks
Story 10	2842.2513	88.82	13.72	1.626	0.01734	SAFE
Story 9	5684.5026	169.54	12.094	1.07	0.01195	SAFE
Story 8	8526.7538	237.79	11.024	1.381	0.01650	SAFE
Story 7	11369.0051	294.20	9.643	1.448	0.01865	SAFE
Story 6	16377.7568	371.63	8.195	1.088	0.01598	SAFE
Story 5	21386.5085	432.56	7.107	1.192	0.01964	SAFE
Story 4	26395.2602	476.90	5.915	1.29	0.02379	SAFE
Story 3	31404.0119	506.3249	4.625	1.351	0.02793	SAFE
Story 2	36412.7636	522.83	3.274	1.41	0.03273	SAFE
Story 1	41421.5153	528.97	1.864	1.864	0.04865	SAFE

Appendix E

The diagram of the ground motions records

Figure E-1 shows the diagram of acceleration time histories of PEER records and figure E-2 shows the diagram of acceleration time histories of IBC.

Figure E. 1:

Acceleration time histories of PEER records.

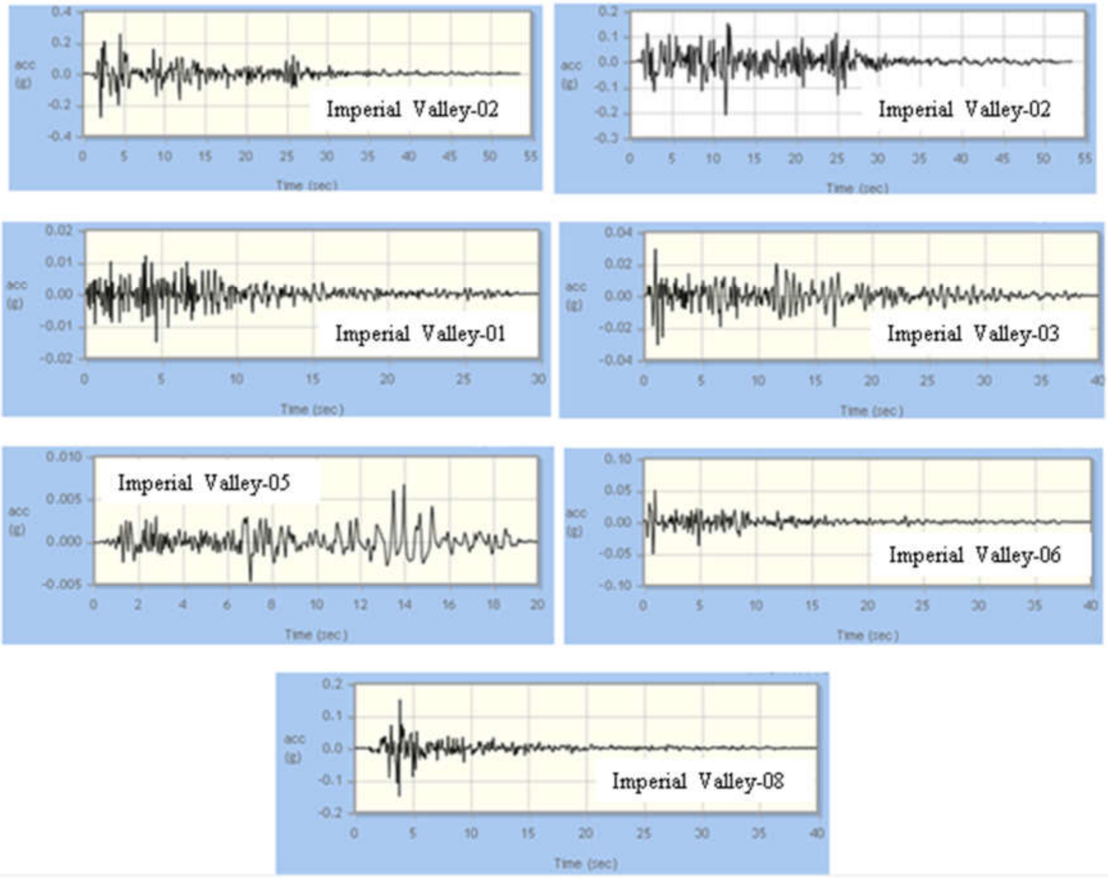
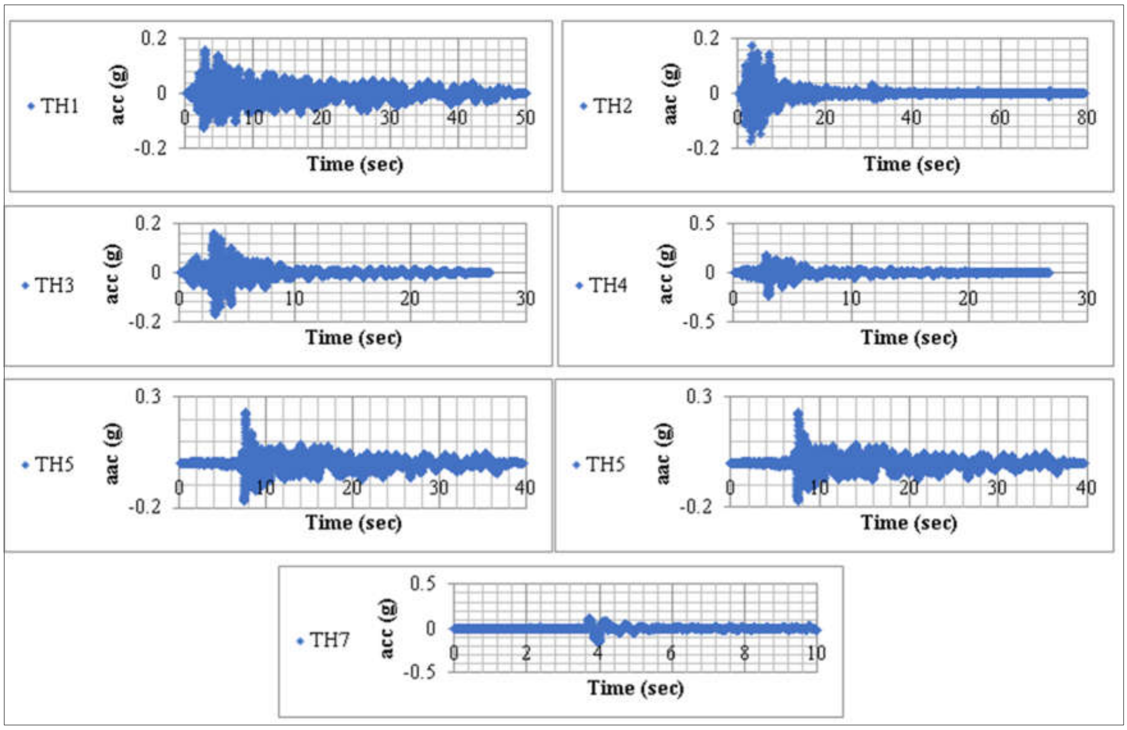


Figure E. 2

Acceleration time histories of IBC earthquake records.



Appendix F

The results of 5 story buildings

Figure F-1 shows the distribution of shear force inter-story from the three methods of analysis before modifying the law of vertical distribution with alpha. Figure F-2 shows the distribution of shear force after modification. Figure F-3 shows the distribution of shear force with α equal to 0.64.

Figure F.1

Distribution of story shear for 5-story buildings.

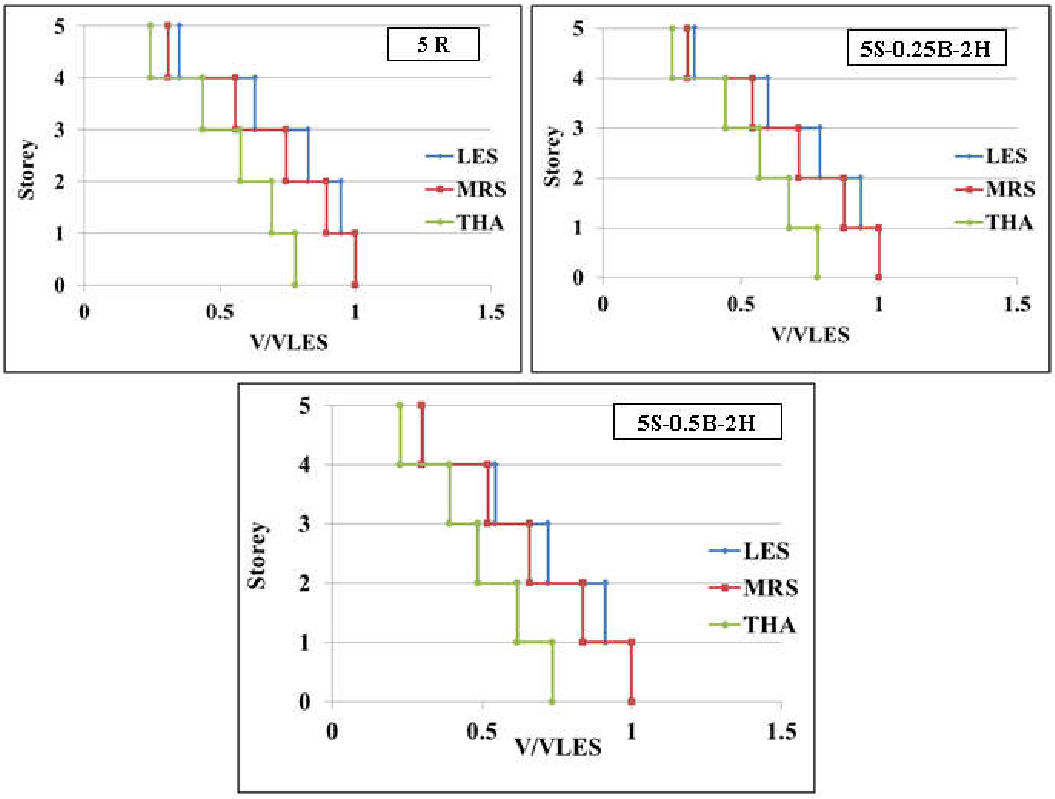


Figure F.2

Distribution of story shear after modification using variable alpha.

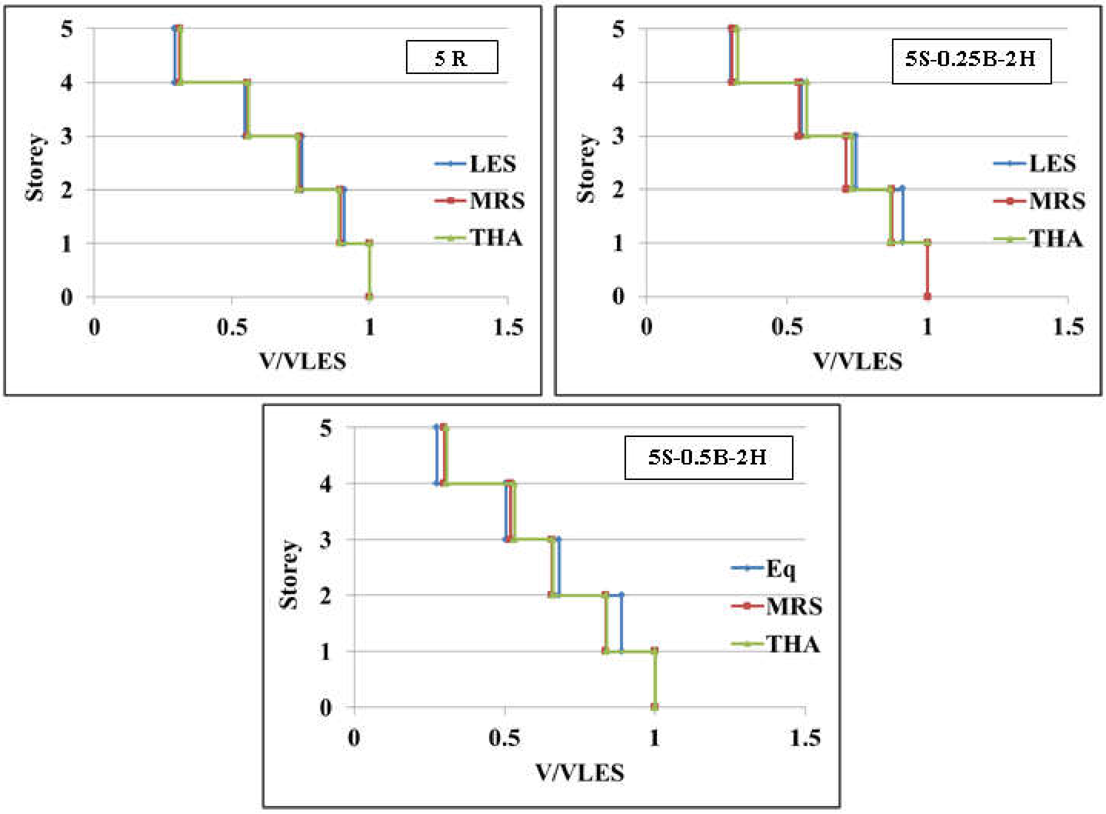


Figure F. 3

Distribution of story shear with constant alpha.

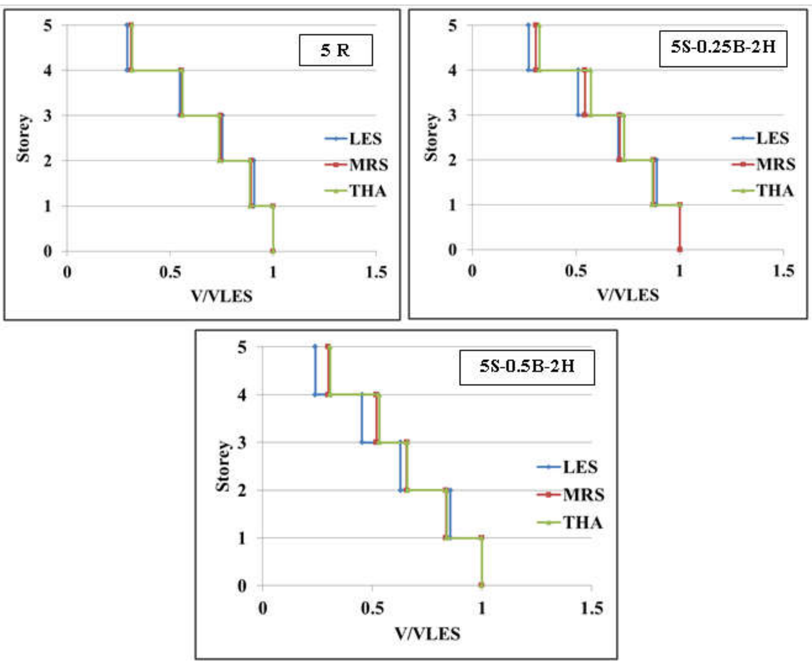
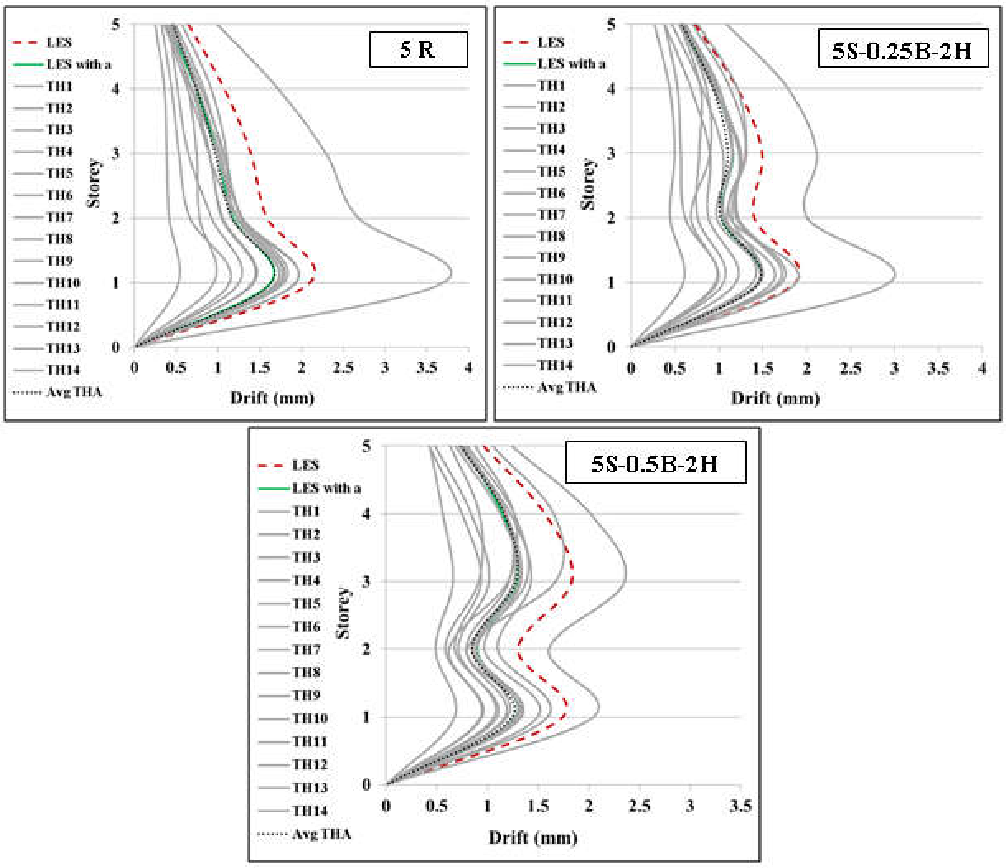


Figure F. 4

Comparison of story drift for 5 story buildings.

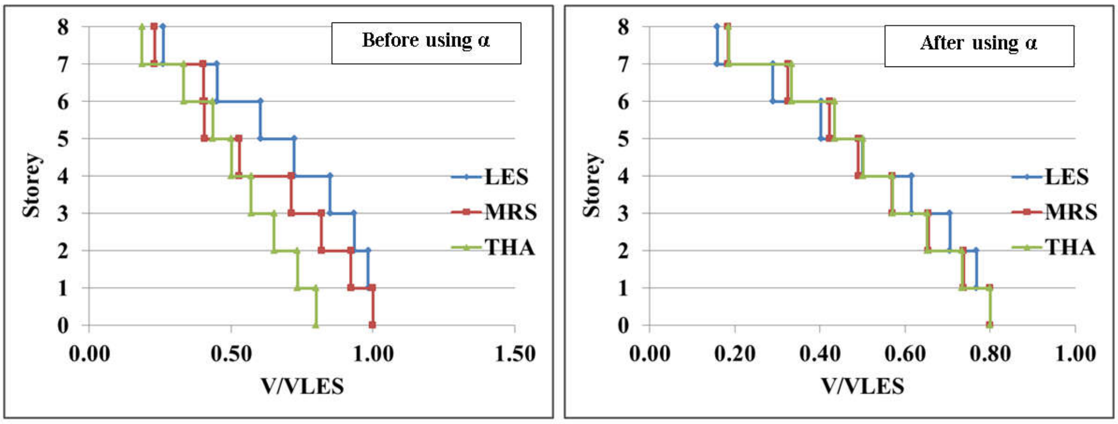


Appendix G

The results of the verification building

Figure G.1

Distribution of story shear before and after using alpha.

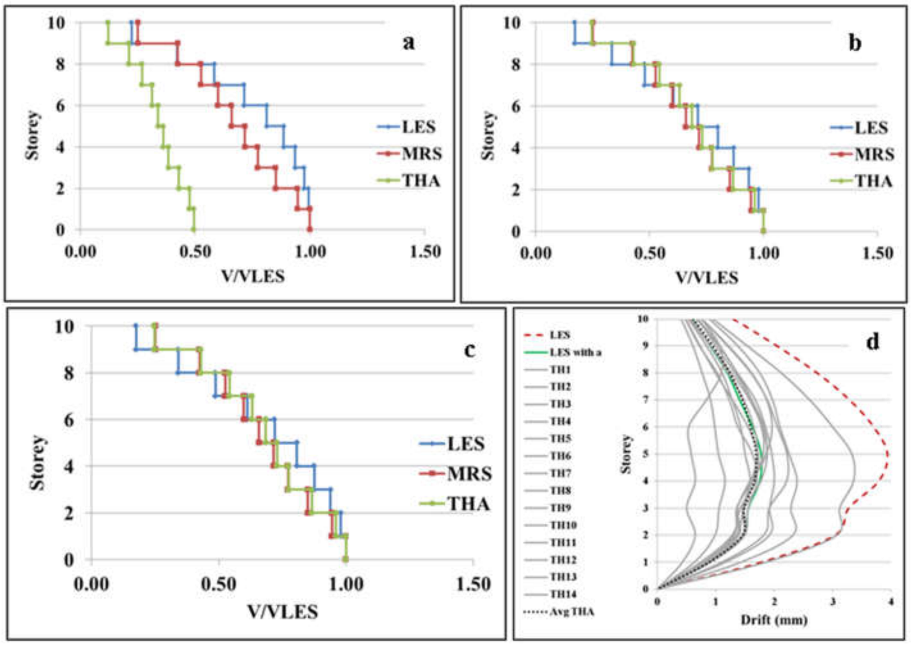


Appendix H

The results of the modeled case without masonry walls

Figure H.1

The results of 10S-0.25B-2H without masonry wall.

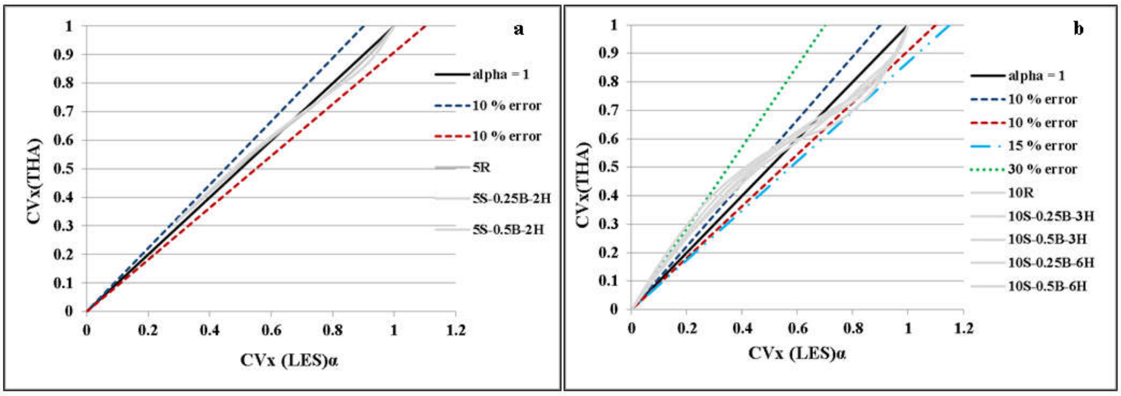


Appendix I

The relative error between the coefficient of the vertical distribution (CVx)

Figure I. 1

The vertical distribution coefficient (CVx) from the LES and the THA (a) for 5-story buildings and (b) for 10-story buildings.

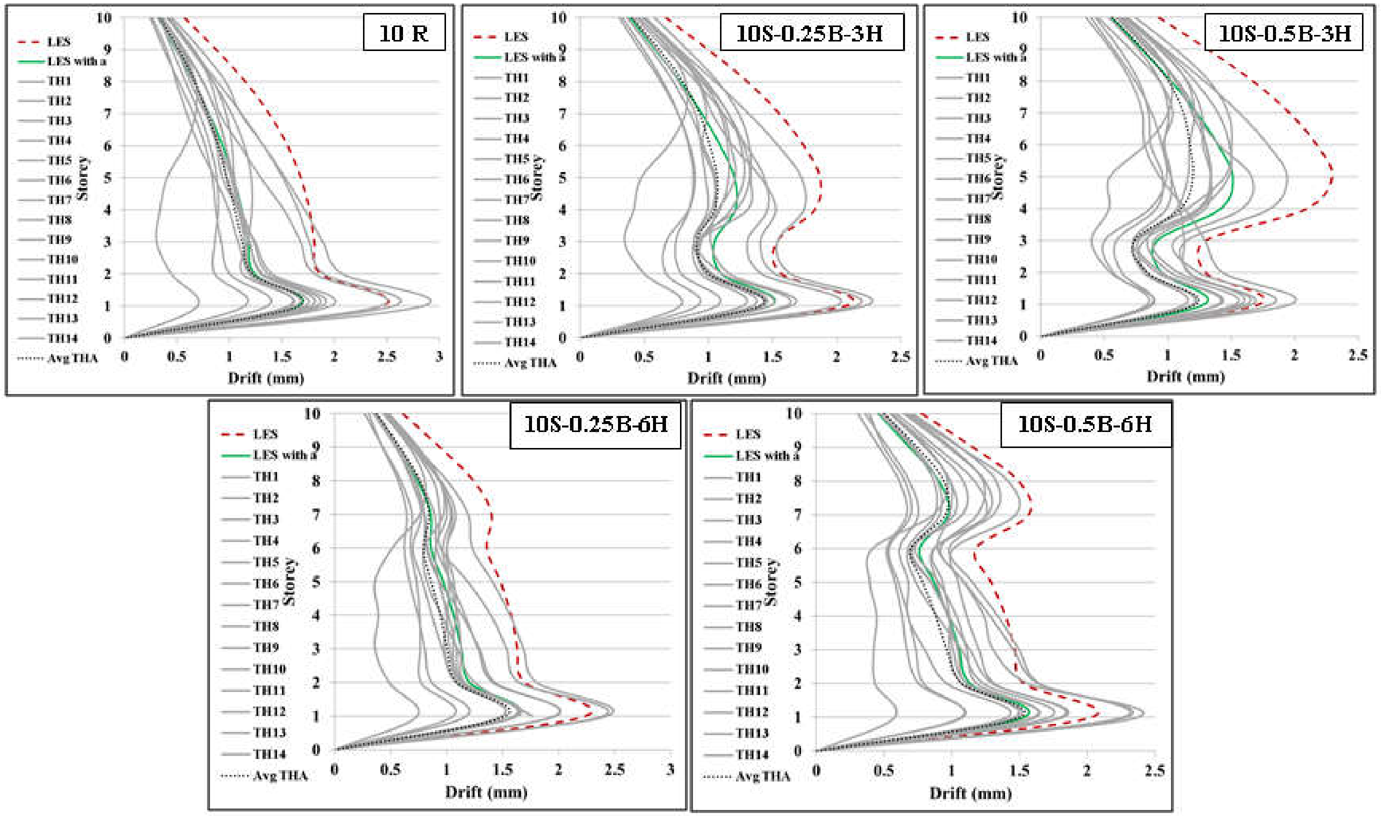

a
CVx(THA)
CVx (LES)α
alpha = 1
10 % error
10 % error
5R
5S-0.25B-2H
5S-0.5B-2H
b
CVx(THA)
CVx (LES)α
alpha = 1
10 % error
10 % error
15 % error
30 % error
10R
10S-0.25B-3H
10S-0.5B-3H
10S-0.25B-6H
10S-0.5B-6H

Appendix J

Comparison of story drift for 10 story buildings

Figure J. 1

Comparison of story drift for 10 story buildings.



جـامعـــة النجاح الوطنية

كليـــة الدراســـات العليـــا

تأثير الانتكاس على الاستجابة الديناميكية المرنة لهياكل الاطارات الخرسانية المسلحة

إعداد

أسيل جمال فريد بدران

إشراف

د. محمود دويكات

قدمت هذه الأطروحة استكمالا لمتطلبات الحصول على درجة الماجستير في هندسة الانشاءات، من كلية الدراسات العليا، في جامعة النجاح الوطنية، نابلس–فلسطين.

2023

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الملخص

الخلفية: يبدأ الانهيار الناتج عن الحمل الزلزالي عادةً في مواقع ضعف النظام الهيكلي في المبنى. غالبًا ما تحدث نقاط الضعف بسبب انقطاع الكتلة أو القوة أو الصلابة بين طابقين متجاورين. نوع معروف من عدم الانتظام الهندسي العمودي في الهياكل هو الارتداد ، تُعرّف على أنها انخفاض مفاجئ في البعد الجانبي للمبنى عند ارتفاع معين.

مشكلة البحث: يضع كود ASCE 7:16 قيودًا على الطريقة الخطية المكافئة الثابتة (LES) ، حيث لا تسمح باستخدام هذه الطريقة لبعض أنواع المخالفات ، بما في ذلك عدم انتظام الارتداد ، والتي تقع في فئة التصميم الزلزالي (SDC) D، من ناحية أخرى ، يمكن استخدام طريقة LES في SDC B و C دون قيود. يسمح الكود باستخدام طيف الاستجابة المشروط (MRS) وتحليل تاريخ الوقت (THA) لجميع المباني دون قيود على SDC.

الأهداف: تبحث هذه الدراسة في تأثير الارتداد على معاملات الاستجابة الزلزالية المرنة للهياكل المؤطرة بالخرسانة المسلحة ، مثل الفترة الأساسية ، وقص القاعدة الزلزالية ، والتوزيع بين الطوابق لقوى القص ، وانجراف القصة. أخيرًا ، تبحث هذه الدراسة في حل مشكلة قيود الكود على استخدام طريقة LES.

المنهجية: تم تحليل المباني ذات الجدران الحجرية المحيطة بنسب ارتداد مختلفة على مستويات مختلفة باستخدام البرنامج التجاري ETABS 2016 وفقًا لبنود ACI318-14 و ASCE 7:16. تم إجراء التحليل باستخدام طريقة LES وطريقة MRS وطريقة THA. تمت مقارنة نتائج الاستجابة المرنة للمباني المرتدة مع المباني المتشابهة ولكن بدون ارتدادات ، بالإضافة إلى مقارنة نتائج طرق التحليل المستخدمة في هذه الدراسة.

الخلاصة: بناءً على النتائج ، تم تعديل التوزيع الرأسي لقوة القص من LES اعتمادًا على نتائج طريقة THA. يسمح هذا التعديل باستخدام طريقة LES في العديد من SDCs.

الكلمات المفتاحية: هياكل الانتكاسة. استجابة زلزالية التحليل الثابت الخطي المكافئ (LES) ؛ طيف الاستجابة النموذجية (MRS) ؛ تحليل تاريخ الوقت (THA).