A SIMPLIFIED DISPLACEMENT-BASED PROCEDURE FOR DESIGN & PERFORMANCE EVALUATION OF LOW-RISE MASONRY INFILLED RC FRAME BUILDINGS IN PAKISTAN

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I certify that this research work titled "*A simplified displacement-based procedure for design & performance evaluation of low-rise masonry infilled RC frame buildings in Pakistan"* is my own work. The work has not been presented elsewhere for assessment. The material that has been used from other sources it has been properly acknowledged / referred.

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Copyright Statement

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First and most of all, I am Thankful to Allah Almighty, who has bestowed me an honor to accomplish my dream. I am Thankful to Allah Almighty, for letting through all the difficulties. I am Thankful to Allah Almighty for bestowing the wisdom, the strength, the consistency, the piece of mind and the good health to complete this research study.

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Dedication

I dedicate this research to my exceptional parents whose tremendous support and cooperation led me to this wonderful accomplishment.

Abstract

Reinforced concrete frame structures with masonry infill walls correspond to a typical building topology in Pakistan and around the world. Masonry infill walls are regularly used in construction as internal partitions and external walls. Practicing engineers often employ the code based static procedures for structural design, neglecting the impacts produced by infill walls, since infill walls are believed to be non-structural components. However, the presence of infill walls may contribute to early strength, initial stiffness and energy dissipation capacity of the frame, thus considerably modifying the global seismic performance of frame buildings. There's no practical scheme available to incorporate the effects of these infill panels in conventional static design procedures. To overcome this issue, this study presents a simplified displacement-based design (SDBD) procedure based on the concept of equivalent linearization and using the framework of direct displacement-based design (DDBD) method. The fundamental philosophy of this approach is that buildings should be designed to attain a definite performance level, when subjected to a definite hazard level. This procedure is developed using three case study RC infilled frame buildings (3-, 4-, 5-story high) and the accuracy of this proposed procedure is evaluated for a 5-story high building under specified level of ground motions. In this evaluation, the inelastic seismic demands estimated by nonlinear time history analysis (NLTHA) procedure are employed as a benchmark. It is shown that the proposed SDBD procedure can accurately estimate the local & global responses for infill RC frame building under input ground motions.

KEYWORDS

Equivalent linearization, nonlinear analysis, infill RC frames, direct displacement-based design method, response spectrum analysis

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List of Abbreviations

ASCE	American Society of Civil Engineers
BCP	Building Code of Pakistan
UBC	Uniform Building Code
FEMA	Federal Emergency Management Agency
MMP	Multi-Modal Pushover Analysis
RC	Reinforced Concrete
RSA	Response Spectrum Analysis
ELF	Equivalent Lateral Force
DDBD	Direct Displacement-Based Design method
NLTHA	Nonlinear Time History Analysis
MPA	Modal Pushover Analysis
EL	Equivalent Linearization
CSM	Capacity Spectrum Method
SD	Spectral Displacement
DCM	Displacement Coefficient Method
SDOF	Single Degree of Freedom
MDOF	Multi-Degree of Freedom
PGA	Peak Ground Acceleration
SDBD	Simplified Displacement-Based Design
SHM	Simple Harmonic Motion
OMRF	Ordinary Moment Resisting Frame
LATBSDC	Los Angeles Tall Buildings Structural Design Council
ICT	Islamabad Capital Territory

Chapter 1

Introduction

1.1 Background

During earthquakes, the structures may experience the ground shaking which induces a relative lateral displacement between the base and subsequent stories of the structure. In other words, we can say that earthquakes apply lateral displacement demand on structures. For a building to withstand the earthquake loading, it should have sufficient lateral displacement capacity to resist the displacement demand. The variation of earthquake shaking is random in nature and will be different from one place to another. The damage because of this shaking is affected by the source to site distance, soil conditions, faults types, detailing of structure, the height of the structure, and many other parameters. If the dominating time period of earthquake ground motion is less then generally it is likely to affect the low-rise buildings which have less natural periods and vice versa. Nevertheless, for convenience, seismic design codes recommend the design of buildings using an equivalent lateral force procedure [1] in which induced lateral force is considered rather than lateral displacement that is imposed.

Pakistan is geographically overlapping with Indian and Eurasian tectonic plates, Sindh and Punjab provinces lie in the northwest corner of the Indian Plateau, while the Baluchistan and most of the Khyber-Pakhtunkhwa regions lie with the Eurasian plateau. Gilgit-Baltistan and Azad Kashmir are located on the edge of the Indian plate and will be prone to severe earthquakes in case of collision of two tectonic plates. Thus, Pakistan is subjected to many moderate to severe earthquakes every year, especially in the North and West regions [2].

Reinforced concrete frame structure with masonry infill walls of low- to mid-rise height is the most common structure type in Pakistan and many developing countries [3], [4]. Masonry infill walls are assumed to be a non-structural component so these infill walls must not be bonded with the rest of the structure system to avoid any stiffness contribution or to obtain some forces. This is why the behavior of infill walls is not accounted for in the code-based design procedures but in Pakistan and many countries, the masonry infill walls are glued or bonded with the structural elements. Various studies have demonstrated that bonded masonry infill walls attract forces and greatly affect the behavior of structure by imparting initial stiffness and lateral strength [5], [6]. Due to the low ductility and brittle behavior of masonry infill walls, this is the first element to collapse in the structure at low drift ratios and can cause consequences. The masonry infill walls of the ground or first floor may crack or collapse before any other structural component during any strong ground motion and this thing will lead to the formation of soft story mechanism in those buildings [7]. As the behavior of infill walls is strictly brittle; therefore, it is not wise to model the infill walls with linear behavior like code-based design procedures. To account for the masonry infill walls' effects on the structure, a non-linear force-deformation behavior is required, adequately sophisticated to capture the failure modes of masonry infill walls.

1.2 Problem Statement

Huge stock of reinforced concrete frame structures with masonry infill walls is present in different moderate to severe seismic zones of Pakistan. Pakistan is in a seismically active region and lies in the global tectonic framework with three major tectonic plates (Eurasian, Indian and Arabian) existing in this part of the world. This indicates that there's a very chance of seismic activity in this region. Earthquakes are a disaster in nature and can cause severe damage to buildings and human lives. This existing stock of RC frames with infill walls is designed using code-based procedures and code-based design methods do not account for infill walls' effects. Several research studies [8] concluded that masonry infill walls must be considered in the design process as it affects the behavior of structures significantly. These infill walls can change the time period of the structure considerably which may affect the selection of spectral acceleration values from response spectra developed for that area. Furthermore, the behavior of infill walls is brittle; therefore, this behavior cannot be captured by a linear model.

However, modeling the structures nonlinearly demands a considerably higher level of expertise and detailed insight into various complex interactions in comparison to linear elastic modeling. It additionally requires significant computational effort and resources. Therefore, the detailed nonlinear time history analysis (NLTHA) procedure and post-processing of results might cost a significant amount of time. Furthermore, an ordinary design office may not have the requisite expertise and resources to go through this process for every project. As a result, currently, there is no practical method available to account for the effects of these infill walls in the design of RC components of such buildings. Therefore, there's a dire need for a method that can include the masonry infill walls' effects in its formulation.

This study proposes a practical scheme to account for the effects of masonry infill walls in the analysis of existing buildings and the design of new buildings. It is based on the direct displacement-based design (DDBD) procedure of Priestley [9] for low-rise frame structures with infill walls, which uses the equivalent linearization (EL) approach to determine the equivalent linear properties. This procedure is based on the assumption that a properly tuned linear elastic SDOF model (with elongated natural period and additional damping) can approximately represent the behavior of a nonlinear SDOF model and hence can provide reasonable estimations of peak nonlinear seismic demands for that system. This procedure retains the convenience offered by linear elastic modeling for practicing engineers, as it neither requires nonlinear modeling nor nonlinear analysis but provides suitable results in comparison to the NLTHA procedure.

The DDBD method is a potential method and vast research has been carried out on this method as it is been developed for various structural systems. The assumptions considered in the design stage to estimate the design lateral force are satisfactory. The DDBD method provides graphical curves against the ductility ratio to get the equivalent linear properties (e.g. elongated period and additional damping) of structural systems. A range of hysteretic models is available in the DDBD method for approximately every structural type like RC frames, RC frames with shear walls and core wall structures, etc. This DDBD method has previously been used to design and determine the nonlinear seismic demands of concrete frames and accounts for masonry infill effects implicitly, but it does not incorporate the effects caused by masonry infill panels precisely in RC frame structures when subjected to other than low drift levels.

This study provides relationships against the roof drift ratio instead of the ductility ratio incorporating the effect of masonry infill walls to get the equivalent properties of buildings by using the actual cyclic behavior of buildings instead of specified hysteretic models. In this study, a simplified displacement-based design (SDBD) procedure is proposed to design and evaluate the low-rise RC frame structures incorporating the effects of masonry infill walls with reasonable accuracy.

1.3 Methodology

A review of the specific area of study is done and the research topic is narrowed down by putting the research work in the strategic framework. A detailed study of available literature on various nonlinear static procedures is done to understand the level of research already carried out. The focus of the literature review is the past research studies examining different design methodologies lying under the umbrella of performance-based seismic design of low-rise RC frame structures and the effects of masonry infill walls on the structural behavior.

The four RC frame buildings with infill walls selected for this research purpose are representative of typical low-rise buildings of Pakistan. The linear elastic models of these three buildings are prepared in “ETABS” software [10]. The buildings are modeled on the same parameters on which these were designed e.g. the same reinforcement is used to calculate the capacities of each member. These capacities are then used to develop the nonlinear finite model in “PERFORM 3D” software [11] to capture the real behavior of buildings. Columns are modeled nonlinear using the fiber hinges while beams are modeled nonlinear using the plastic hinge modeling approach in accordance with ASCE 41-17 [12].

Selection of appropriate ground motion records is done that closely match the required field strata. These earthquake records are then scaled and matched to our target response spectra to meet the hazard level. The nonlinear time history analysis (NLTHA) procedure has been performed using the suite of seven compatible ground motions to check the accuracy of the proposed SDBD procedure. The seismic demands of the SDBD procedure are compared with the standard response spectrum analysis (RSA) method and the benchmark demands of the NLTHA procedure.

1.4 Research Objectives

Based on the detailed studies of literature, it has been found that no scheme or method is available which considers the effects of masonry infill walls in RC frame structures to capture the real building behavior. To fill this gap in research, the following research objectives have been established.

- To examine the effects of masonry infill walls on the structural performance of RC frame buildings.
- To develop a simplified displacement-based seismic design (SDBD) procedure for low-rise RC frame structures incorporating masonry infill walls in the context of Pakistan.
- To evaluating the performance of the proposed SDBD procedure by comparing the results to the benchmark detailed nonlinear response history analysis and the conventional code-based response spectrum analysis (RSA) method.

1.5 Organization of Thesis

Chapter 1: This chapter presents the general introduction to structural configurations of the buildings and the seismic hazard level of Pakistan. The next segment of the chapter deals with the problem statement, the methodology adopted for this study, research objectives, the scope of this work, and the limitations of this study.

Chapter 2: This chapter describes the overview of conventional seismic design procedures, performance-based design methodologies, and the behavior of buildings under earthquake loadings. In addition to this, the basic concepts of displacement-based design are also explained in the later section of this chapter.

Chapter 3: This chapter covers the common techniques used to nonlinearly model the structural components, equivalent linearization methods, ground motion selection, and spectral matching techniques. Furthermore, it deals with the development of the proposed simplified displacement-based design procedure incorporating the infill walls.

Chapter 4: This chapter deals with the seismic performance evaluation of the proposed simplified displacement-based design procedure in comparison to the detailed nonlinear time history analysis procedure. This comparison also includes standard response spectrum analysis for more insights.

Chapter 5: This chapter discusses the conclusions drawn from this study and provides insightful recommendations for future work.

1.6 Scope and Limitations of Research

The consideration of nonlinearity in the structure has not been practiced by design engineers as they practice only linear elastic modeling of the structures. Designing reinforced concrete buildings to resist imposed displacements elastically requires substantially large sizes of structural elements and a significant percentage of reinforcement. Therefore, it is uneconomical to design buildings which remain elastic during severe earthquake shakings. Consequently, for regular buildings, the seismic design codes recommend the design of buildings such that they tend to resist the imposed displacement through inelastic actions. But for the RC frame structures with masonry infill walls, seismic design codes do not provide any guidelines for their design.

The scope of this research study is to propose a simplified displacement-based design procedure that will consider the effects of masonry infill walls in its formulation and to evaluate the performance of this proposed SDBD procedure by comparing the results with benchmark results of the NLTHA procedure. This study also highlights the significance of masonry infill walls on the local and global behavior of RC frame structures. The implementation of this SDBD procedure does not require nonlinear modeling or analysis; therefore, the practicing design engineers will be able to get the appropriate results using linear elastic modeling techniques.

This proposed SDBD procedure is based on the direct displacement-based design (DDBD) method which requires the design of the building for the severe level of earthquake shaking. Performance analyses of designed buildings using this method indicate that this analysis does not present all necessary guidelines to ensure that buildings have achieved their desired performance. The limitations of this study are that opening in masonry infill walls, micro-modeling of infill walls, use of plastering on infill walls, higher modes effects, and the soil structure interaction of RC frame structures are not included in this proposed SDBD procedure. This proposed scheme is valid for the low-rise height of infill RC frame structures only.

Chapter 2

Literature Review

2.1 Introduction

Low-rise reinforced concrete (RC) frame structures with masonry infill walls are commonly constructed in Pakistan. Most building structures are designed according to the code-prescribed equivalent static force procedure, which is a very crude method and does not account for masonry infill walls behavior in its formulation. With the improvement in computational power and availability of advanced tools, appropriate and convenient techniques can be developed to account for the behavior of masonry infill walls.

This chapter presents the overview of seismic design of buildings, the behavior of buildings under lateral loadings, research already carried out regarding the various nonlinear static procedures, and displacement-based design methods lying under the umbrella of performance-based design methodologies. This chapter also discusses the underlying idea of the direct displacement-based design (DDBD) method in detail and the behavior of RC frame structures incorporating masonry infill walls. In the end, a summary of the literature review is presented which is focused on the effects of masonry infill walls on RC frames under lateral loading.

2.2 Overview of Seismic Design Approaches

Buildings are designed to fulfill the safety requirements against the loading effects. For this purpose, buildings must possess sufficient lateral stiffness to prevent damage occurrence in structural and non-structural components under slight shaking. Buildings must have sufficient lateral strength to prevent damage in structural components during substantial shaking and sufficient displacement capacity to prevent collapse when subjected to extreme shaking. This is the philosophy of earthquake resistant design that focuses on acceptable performance of designed structures under various intensities of ground motions [13].

During the early 20th century, structures were designed such that they remain linear elastic during any level of seismic loadings [14]. The initial stiffness was the only governing parameter in the design of buildings under earthquake resistant design method; therefore, this design technique was labeled as stiffness-based design. Buildings that were designed by this approach had bulky cross sections of elements and a high initial cost. Buildings so designed behaved inadequately under strong ground motions. This motivated the development of a new design methodology in which buildings were designed to withstand a minimum lateral force demand, which would be much smaller than the force that a structure will experience to remain elastic and this modeling approach was termed strength-based design or force-based design. Then this initial stiffness and lateral strength became the controlling parameters in earthquake resistant design philosophy and buildings so designed withstood strong ground motions by enduring damage in structural components resulting in low initial construction costs [15].

The limitation of the force-based design was that the brittle shear failure of structural elements occurred when the building is subjected to strong ground motions. The strength-based design was subsequently altered to avoid the brittle shear failure of structures by providing satisfactory shear capacity and corresponding flexural capacity at beam-column joint to limit inelastic actions at pre-determined locations and elements. This revision in strength-based design was called the capacity-based design procedure [16]. Consequently, the guidelines or provisions for capacity-based design were incorporated into seismic codes.

In the last two decades, new seismic design procedures have been developed that express the lateral force demand of structure as a function of displacement capacity. Displacement capacity is a new input in this design procedure, therefore, this procedure was named a displacement-based design procedure [9]. This design procedure guarantees that the building will undergo desirable inelastic behavior; inelastic actions will only develop at pre-determined locations, and ensures the buildings so designed have an appropriate lateral strength. Now the three controlling parameters in earthquake-resistant design are initial lateral stiffness, lateral strength, and lateral displacement capacity.

In the near future, the lateral force demand of structures is going to be characterized as a function of their energy dissipation capacity and this design strategy will be referred to as energy-based design. This procedure guarantees the desirable inelastic behavior of structures so designed, restricts the inelastic energy dissipation at pre-determined locations of certain elements, as well as ensures sufficient lateral displacement capacity and a specified energy dissipation capability of structures. Consequently, the four controlling parameters in earthquake-resistant design are initial lateral stiffness, lateral strength, lateral displacement capacity, and energy dissipation capacity of the structures [17].

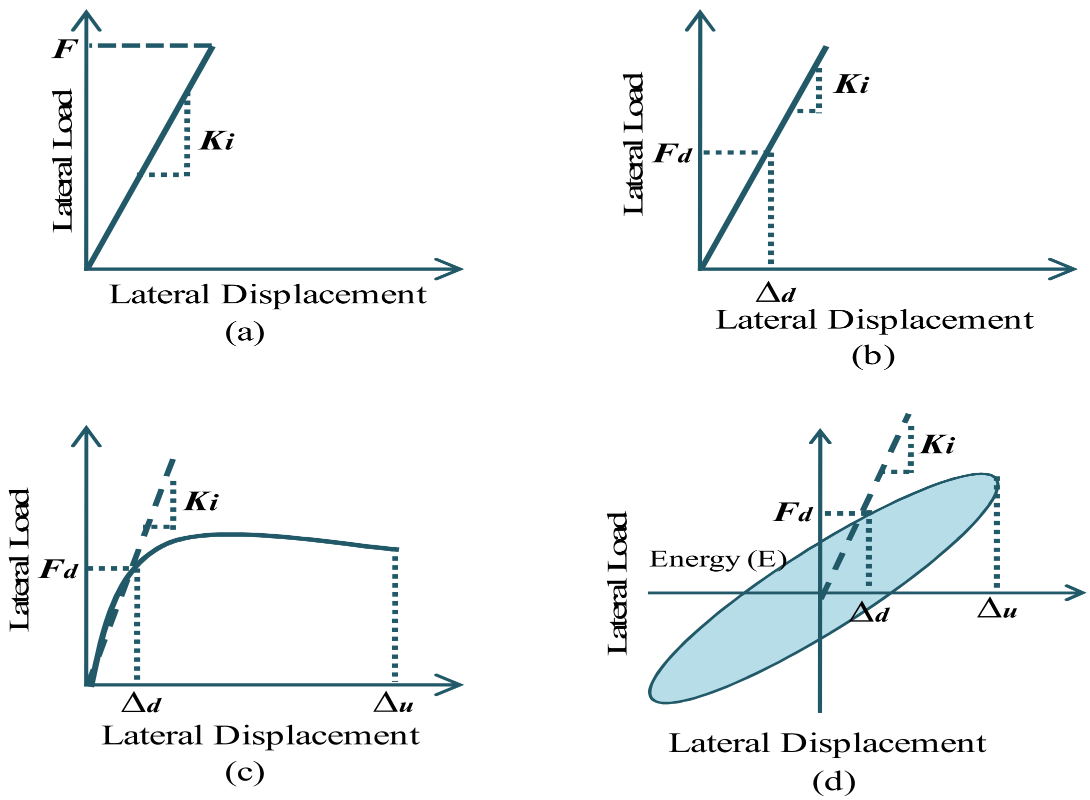


Figure 2-1: Different seismic design approaches (a) Stiffness-based design, (b) Strength-based design, (c) Displacement-based design, and (d) Energy-based design methods

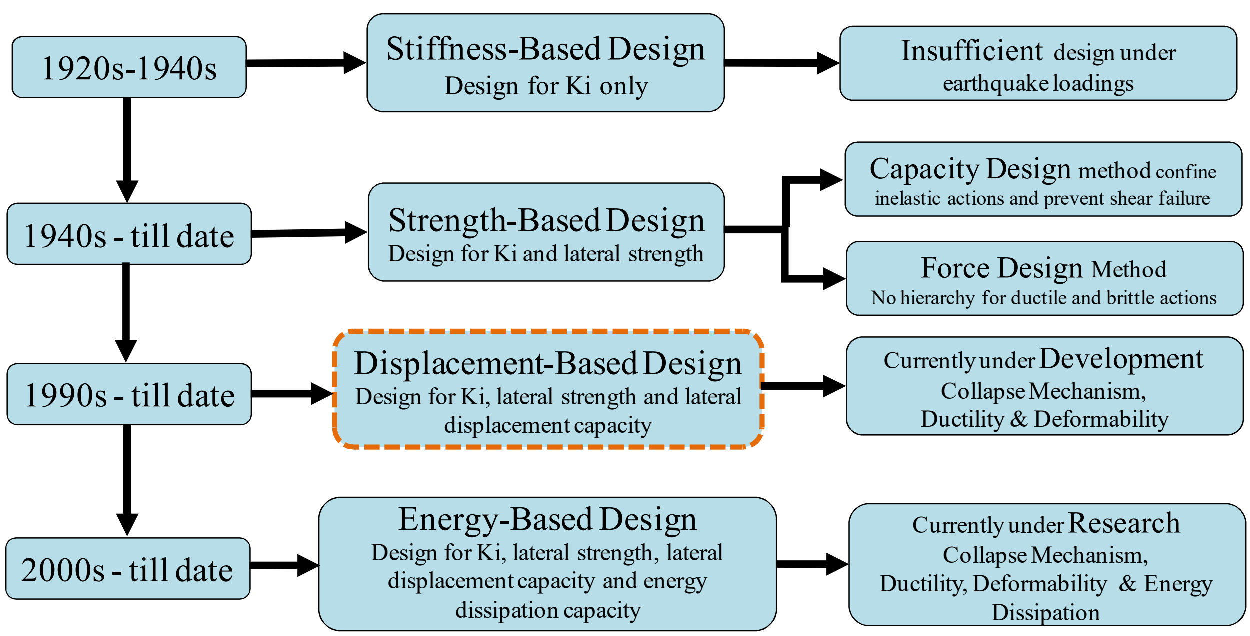


Figure 2-2: Overview of various seismic design approaches

2.3 Behaviour of Buildings under Earthquake Loadings

Understanding the actual behavior of building structures is necessary before aiming to improve the performance of structures under earthquake loadings. Building configuration has a primary impact on its behavior along with other key parameters. Building configuration includes overall geometry, size & shape of the building, distribution & positioning of lateral load resisting structural elements as well as location & proportion of non-structural elements in the building. In 1971, during the earthquake of San Fernando, buildings of irregular configuration suffered heavy damage. Subsequently, understanding the effects of architectural features on a building's behavior became an area of wide interest [18]. Studies on the effects of basic geometry on the performance of buildings demonstrated that convex form buildings (elevation and plan) showed better results under earthquake shakings than those concave form buildings. The reason is that concave form buildings do not have a direct load transfer path while convex form buildings transfer the load through a direct path [17].

The fundamental natural period of the building is the key dynamic parameter in understanding the behavior of buildings. It depends upon the magnitude and distribution of seismic mass along with the initial lateral stiffness of the building. In the 1930s, the overall flexibility of buildings was recognized as a vital parameter that affects the lateral force demand on structures during ground shakings [19]. Typically, flexible buildings attract lower forces than the stiffer buildings which motivated the construction of flexible structures but the inadequacies in the design lead to heavy damage accumulation in these flexible buildings under strong ground shakings. The deficiencies are significant damage to non-structural components, undesirable collapse mechanisms, and large P-delta effects. Moreover, local flexibility may lead to undesirable soft story mechanisms that caused the collapse of buildings. To lower the detrimental effects of local and global flexibility, seismic design codes restrict the building flexibility by defining a maximum permissible inter-story drift ratio (IDR) under the lateral force design [1]. The distribution of stiffness along the plan and elevation affects the seismic mass participation in each vibration mode. As per seismic design codes, buildings should be proportioned such that the fundamental vibration mode of the building accounts for 90% of the seismic mass of the building.

To reduce the construction costs, the building must resist the severe earthquake with some inelastic damage while preventing collapse occurrence. In addition to various other factors, the actual lateral strength of the building depends upon design lateral force which is a function of response modification factor R. The three major elements that determine R are the over-strength factor, ductility factor, and redundancy factor [20]. The over-strength factor is quantified as the ratio of lateral over-strength to the design lateral force of the building. This over-strength factor depends upon various factors involving partial load safety factors, partial resistance factors, strain hardening of materials, confinement of cross sections, as well as the difference in actual and expected material strengths and second order P-delta effects. This shows that R is related to the inherent ductility of the structure, for example, ordinary moment resisting RC frames of little or no ductility have the R-value of 3 while special moment resisting RC frames with adequate ductility have the 8 R-value [1].

Adequate ductility capacity of the building is necessary to prevent the collapse mechanism during strong earthquakes. The ductility capacity of the building is defined as the ratio of displacement under consideration or lateral displacement capacity to the yielding displacement of the building. It determines the nonlinear displacement capacity of a building normalized to yield displacement while carrying the loads safely. The higher the ductility, the better the building's response to seismic shaking, and this ductility entirely depends on the design & detailing of the structural elements. Generally, this ductility of the building is made up of member level ductility (which is made up of cross-section level and material level ductility), cross-section level ductility (which is dependent on material level ductility), and material level ductility. Consequently, improving the building ductility demands designing and detailing the structural elements in such a way that ductility improves at each of the three levels.

The ductility and deformability both play an important role in determining the behavior of buildings during earthquakes. Buildings with less or no ductility/deformability are most likely to fail in a brittle manner which is undesirable. Both ductility and deformability are related to the lateral displacement capacity of the building. But, ductility measures the plastic displacement capacity while deformability determines the total lateral displacement capacity of the building. Deformability refers to the building's ability to withstand lateral displacements without risking the gravity load carrying capacity at the local or global level [17]. Similar to ductility, the deformability of the building is also made up of member-level deformability, cross-section level deformability, and material level deformability. Furthermore, it depends on the collapse mechanism, plastic rotation capacity of structural components, and sequence of hinges development on the way to collapse. The beam sway mechanism is the ideal collapse mechanism as it dissipates the highest amount of energy by forming plastic hinges at all beams before collapse occurrence. The beam sway mechanism's greater energy dissipation capacity increases the deformability of the building, which enhances the likelihood of the building surviving without collapsing under severe earthquakes. An increase in the plastic rotation capacity of structural components can increase the deformability of the structure and providing confinement in RC sections is one tactic to enhance the plastic rotation capacity. Current seismic codes do not provide guidelines to control the sequence of plastic hinge formations. If the plastic rotation capacity of any plastic hinge is achieved before the formations of all other plastic hinges then this will lead to unsuccessful utilization of rotation capacities. Consequently, inadequate hinges formation at desirable locations will reduce the energy dissipation capacity and deformability of the building structure.

2.4 Existing State of Design Practices

Generally, design offices use the force-based design method to design the buildings, where design lateral force is the design input. First, loading demands resulting from gravity, seismic and miscellaneous loads are determined for the building under consideration. Then the required capacities are provided at the specified locations of structural elements to withstand these loading effects.

Normally, seismic design codes presume that the buildings so designed will have adequate ductility and deformability. Past earthquakes revealed that designing the buildings for lateral force only is not appropriate and there's a need to estimate the lateral displacement demand on the buildings. Furthermore, the lateral displacement capacity of the buildings should be adequate when compared to the lateral displacement demand.

2.4.1 Development of the Force-Based Design Method

In the early 20th century, force-based design method was first used to design buildings. A basic approximation of design lateral force was provided based on the idea of inertial force as a proportion of the total building weight.

$$H = \mathrm{C} * \mathrm{W} \qquad (2.1)$$

Here, C is called the seismic coefficient and W is the weight of the building. The value of C ranged from $0.1 - 0.15$ for regular and special structures respectively [14]. Until the $1930s$, C was supposed to be exclusively a function of ground motion parameters. Revolutionary work was undertaken in $1930s$ which employed the principles of structural dynamics to analyze the building's behavior under earthquake shakings. It was found that apart from being the function of ground motion characteristics C also depends upon the dynamic features of the building (e.g. modes of vibration, natural period, and inherent damping). Subsequently, with a better understanding of the behavior of the structures, seismic codes altered the estimation of the seismic coefficient to account for the building's fundamental natural period, as:

$$H = (\mathrm{C1}/\mathrm{T1}) * \mathrm{W} \qquad (2.2)$$

Here, $\mathrm{C1}$ is a constant whose value ranges from$0.015 - 0.025$, $\mathrm{T1}$ is the natural period of 1st mode of vibration, and W is the total seismic weight of the building [21].

Buildings designed using the above approaches were expected to remain elastic during seismic actions that resulted in a significant increase in stiffness and lateral strength of structural components. Consequently, this resulted in a huge rise in initial construction costs of buildings. This situation led to the development of a seismic design procedure based on the energy balance concept and acceptable inelastic behavior. This approach suggests that structures should be designed such that they resist the earthquake loadings partially through elastic actions and partially through inelastic actions at specified locations [15]. During the late $1950s$, seismic codes recommended that design lateral force should be estimated as follows:

$$H = \left(\frac{0.5}{\sqrt[3]{\mathrm{T1}}}\right) * \mathrm{C2} * \mathrm{W} \qquad (2.3)$$

Where $\mathrm{T1}$ is the natural period of the fundamental mode of vibration and $\mathrm{C2}$ is the system factor that accounts for the arrangement and type of lateral load resisting system whose value for moment frames is 0.67 [13]. This C2 factor is the outdated version of the response modification factor R which is being used in current seismic codes. During $1960-1990$, substantial improvements were done in the seismic codes e.g. development of elastic design spectra, the introduction of the importance factor I to restrict damage in important/critical buildings, the introduction of zone factor Z that accounts for seismicity of regions, and improvement of response modification factor R for various types of structures [1], [20].

2.4.2 Lateral Force Demand

Seismic design codes determine the design lateral force from elastic lateral force demand and response modification factor R. First, the seismic hazard of any region must be known to estimate the intensity of ground shaking. Seismic codes employ the elastic displacement spectra (EDS) to account for the dynamic characteristics of the structure. This EDS is developed from the response of various single degree of freedom (SDOF) systems having different natural periods which are subjected to different ground motion histories [22]. The EDS depends upon the soil type and damping level considered.

The seismic force demand on structural components can be determined by using the linear elastic method, equivalent static method, and linear dynamic methods. The linear elastic method also known as the equivalent lateral force (ELF) procedure is often employed by the seismic design codes and its application is limited to regular low-rise structures. When multiple modes are considered then the equivalent static method is referred to as response spectrum analysis (RSA) which is utilized in seismic codes for irregular low-mid rise buildings. The dynamic methods (e.g. modal time history analysis, linear time history analysis) comprise a suite of ground motion histories that are spectrally scaled and matched with the target response spectrum for that site. Seismic design codes allow the use of dynamic methods to estimate the demands on structural components of all high-rise buildings.

2.4.3 Lateral Displacement Demand

The understanding of lateral displacement demand imposed by earthquakes on structures is necessary to make the buildings safe against earthquake loadings. To determine the displacement demands, seismic codes generally recommend the equivalent lateral force procedure (ELF), response spectrum analysis (RSA) method, and linear time history analysis (LTHA) procedures.

But the methods that are used extensively in research studies to determine the lateral displacement demands are the substitute structure approach or equivalent linearization (EL) technique and nonlinear time history analysis (NLTHA) procedure. These nonlinear static methods will be explained later.

2.4.4 Lateral Displacement Capacity

When the buildings are designed to resist the earthquake loadings elastically then the only linear elastic analysis is adequate to design and analyze the buildings. But when buildings are designed to resist the strong lateral forces and displacements through elastic and inelastic actions then estimating the lateral displacement capacities involves nonlinear analysis that incorporates the various level of nonlinearity. To determine the lateral displacement capacity of structures, nonlinear static analysis (modal pushover analysis) and incremental dynamic analysis (IDA) are generally used. Both approaches involve the nonlinear modeling of the buildings and their components meticulously.

To model the building nonlinear, capturing the nonlinearity at the material and geometrical level is important. If the material is modeled nonlinear then a cross-section made up of that material would also exhibit nonlinearity and similarly, if the cross-section is modeled nonlinear then the element of this cross-section would also exhibit nonlinearity. When nonlinearity is captured at the material level then this nonlinear modeling approach is referred to as fiber modeling and when nonlinearity is introduced at the member or cross-section level then this nonlinear modeling approach is called plastic hinge modeling approach. Both approaches are discussed in detail in the later section.

The modal pushover analysis provides the load-deformation curve of the building by pushing it generally in the fundamental mode of vibration until it collapses. The analysis is carried out gradually by increasing the deformation demand for buildings. The building is considered to be collapsed when at least one structural component has reached its deformation capacity. Various nonlinear static analysis procedures have been discussed in detail in a later section.

Incremental Dynamic Analysis (IDA) determines the response of the building under increasing levels of earthquake loadings. IDA uses a suite of compatible ground motion histories to estimate the response of the building which makes it more computationally demanding. However, the responses determined from this approach are more reliable than that of nonlinear static analysis procedures [23].

2.4.5 Limitations of the Force-Based Design Method

In the force-based design method, the input is design lateral force which demands buildings to have appropriate lateral force capacity to be safe during earthquakes. However, the lateral displacement capacity of buildings is not necessary to be examined, as a result, buildings so designed couldn't withstand past earthquakes. Therefore, seismic designs should implement a displacement capacity check on buildings to ensure the displacement capacity is more in comparison to displacement demand [24].

Furthermore, existing force-based design methods do not regulate the development of plastic hinges in structures. Additionally, seismic design codes are based on the first mode of vibration and do not limit the participation of higher modes in response of buildings whereas higher modes have considerable impacts on the behavior of flexible/taller buildings. Buildings designed by this method have less energy dissipation capacities, less lateral displacement capacities, and greater plastic rotation demands on structural components of buildings [12]. Consequently, buildings so designed hardly exhibit the desirable behavior when subjected to ground motions.

2.5 Displacement-Based Design of Buildings

Because of the limitations of force-based design procedures, new seismic design procedures, referred to as displacement-based design procedures, have been developed that employ lateral displacement capacity as a design input. A substantial number of these procedures have been described in various studies, including Capacity Spectrum Method [25], Displacement Coefficient Method [25], Direct Displacement-Based Design Method [26], Multi-Modal Pushover Procedure [27], and Target Period Method [28].

2.5.1 Capacity Spectrum Method

The capacity spectrum method (CSM) [25] correlates the earthquake ground motion with associated building performance. The design lateral force is used as design input and specified elements are detailed to have adequate plastic rotation capacities. This method is a graphical representation of the pushover capacity curve of structure and the reduced response spectrum of earthquake demands, which is reduced for non-linear effects of structure, in the acceleration-displacement response spectra (ADRS) to estimate the maximum displacement. The point where the demand response spectrum intersects with the capacity pushover curve in the ADRS spectra is called the performance point, which represents the demand displacement. This performance point depicts that the seismic capacity of the building is equal to the seismic demand imposed on the building for that particular predetermined ground motion. This procedure has no control over the displacement demand imposed on the structure; as a result, the design of the building requires particular iterations to be performed.

2.5.2 Displacement Coefficient Method

The displacement coefficient method (DCM) [25] is the modified version of equal displacement approximation. Modal pushover analysis (MPA) needs to be performed to estimate the target displacement in this method. The target displacement is the maximum displacement likely to be experienced by the structure during its design life under specified ground motions. This method is based on a statistical analysis of the NLTHA results which were performed on several SDOF systems of different types. It provides appropriate coefficients to convert the target displacement of the linear elastic SDOF system into nonlinear displacement which then will be used to determine the resulting deformations and forces. These modifying coefficients are basically the ratio of inelastic to elastic displacements. Due to its simplicity, this method is widely accepted and used for various building structures.

2.5.3 Multi-Modes Pushover Analysis Procedure

In 1998, Sasaki [27] described the multi modes pushover (MMP) procedure that maintains the simplicity of basic pushover analysis and helps in identifying the failure mechanism of long-period structures by incorporating the effect of higher modes of vibration in the pushover analysis. The conventional pushover analysis is unable to identify the failure mechanism as failure modes of long period structures are significantly influenced by other higher modes. This procedure uses the capacity spectrum method for each significant vibration mode to compare the structure's capacity to the earthquake demand forces. The load patterns applied in MMP are based not only on the first mode but also on higher modes to get a better structural response. The load patterns used in this method are based upon the elastic mode shapes of the structure being evaluated. For long-period structures, MMP results more closely match the actual damage than conventional pushover analysis. He believed that MMP is not been developed for design purposes but it is useful in identifying the failure mechanisms because of the inclusion of higher modes effects.

2.5.4 Target Period Method

The target period method [28] takes into account the longest natural period of the building beyond which lateral drift ratios would exceed the acceptable limit. The target period is determined as a function of the slope of displacement response spectra, intended average drift ratio, and modal participation factor of the fundamental mode of vibration. This method involves proportioning of structural elements based on gravity load analysis first and then re-proportioning the element sizes until the 1st mode natural period is considerably more than the target natural period. Finally, the force-based design procedure is used to design the structural elements with adequate plastic rotation capacities to withstand target displacement. This method requires iterations to be performed until the displacement demand on the buildings is less than that of the target displacement.

2.5.5 Direct Displacement-Based Design Method

In this study, the DDBD approach is used as theoretical background to present an effective and practical technique to design and estimate the inelastic seismic demands of RC infill frames. The mathematical formulation and framework of the DDBD method are used to propose a simplified displacement-based design procedure incorporating the masonry infill walls' effects based on the equivalent linearization technique. Therefore, it is important to shortly review the fundamental concepts and basic assumptions of the DDBD method.

The DDBD method has undergone extensive development to mitigate the deficiencies in force-based design and analysis procedures. The basic problem of force-based design is that it uses initial stiffness to calculate the natural period of buildings and distribute the forces in structural elements as per the initial stiffness (stiffer elements attract more forces). This concept is irrational as it tends to concentrate strengths in elements that are most likely to fail brittle. Furthermore, the determination of natural period from initial stiffness would be typically low. Lower natural periods result in higher seismic forces leading to oversized structural cross sections and more reinforcing steel. The displacements calculated using shorter natural periods would be also unrealistically small. The structures of low displacement capacity would not be able to withstand the moderate to severe level earthquake displacement demands.

The DDBD method requires the design of the structure for the severe level of ground shaking. The design input in this method is the most critical of all the inter-story drift ratios. Instead of characterizing the real structure to be designed by its initial elastic characteristics, the DDBD technique characterizes that structure by a single degree of freedom (SDOF) system and uses the secant stiffness; significantly lower than that of initial stiffness, representing the performance at the peak response level. To be compatible with secant stiffness, a level of equivalent viscous damping, which is the combination of elastic damping and hysteretic inelastic response damping, is employed. Consideration of secant stiffness for all hysteresis rules studied produces higher peak displacements than initial stiffness. This DDBD method is based on the substitute structure approach developed by [29], which demonstrates that properly tuned linear elastic SDOF systems can approximately represent the nonlinear SDOF systems with an elongated period and additional damping and hence can accurately predict the nonlinear seismic demands. The underlying idea of equivalent linearization is to replace the nonlinear inelastic system with an equivalent linear elastic system that has energy dissipation characteristics identical to the nonlinear system in some sense. Subsequently, this approach was improved considering the building as a multi-degree of freedom (MDOF) system [30].

Figure 2-3 depicts a fundamental philosophy for the conversion of a nonlinear SDOF system into an equivalent linear SDOF system characterized by secant stiffness K_e and corresponding

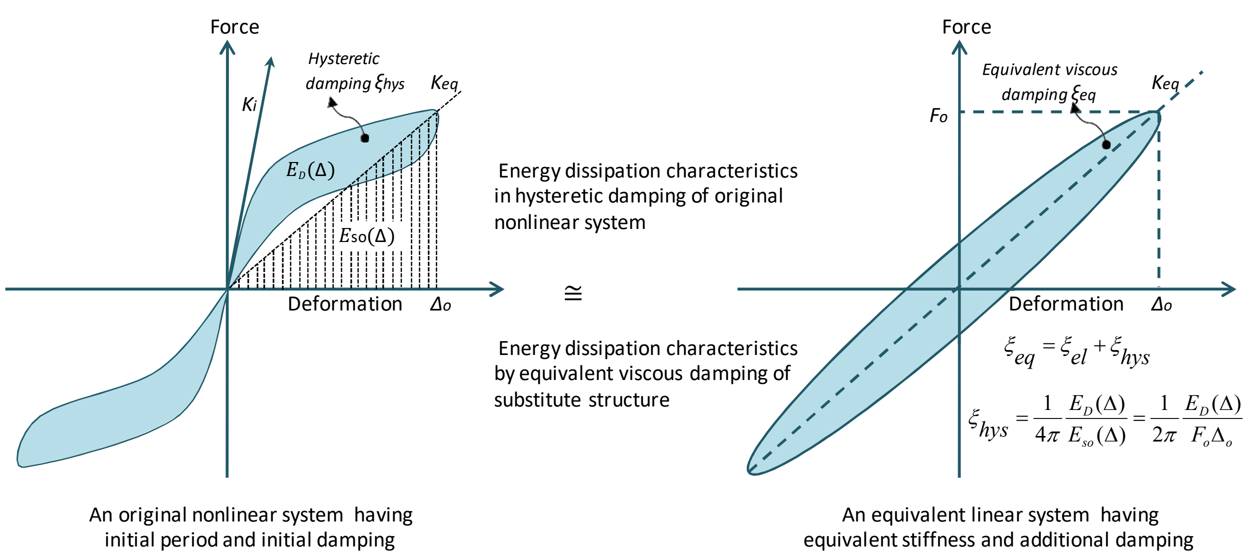


Figure 2-3: Conversion of nonlinear SDOF system into equivalent linear system

hysteretic damping at maximum displacement response.

The principles of the DDBD technique are shown below in Figure 2-4, with an emphasis on SDOF representation of frame structure, while the basic concepts are applicable to all structural types. The lateral force-displacement response of frame building represented by the SDOF system is illustrated as bilinear envelops where an initial stiffness K_i changes to post-yield stiffness rK_i.

The design of structures using this method involves the estimation of the design lateral force of the structure as a function of design input and the design and detailing of elements of that structure. The process for the estimation of design lateral force is as follows:

Estimate lateral displacement demands Δ_d on the building using the assumed inter-story drift profile and select the critical inter-story drift. Estimate yield lateral displacement Δ_y of the building using slenderness ratio of beams, estimate displacement ductility μ_Δ of the building using Δ_y and Δ_d, and estimate equivalent viscous damping ξ_{eff} using μ_Δ and structural type as shown in Figure 2-4. Once the design displacement at the peak response level has been figured out and respective equivalent structural damping has been determined from the expected ductility demand, the effective period T_{eff} at peak response, corresponding to the effective height H_e, can be obtained from a series of displacement spectra for various levels of equivalent damping as illustrated in Figure 2-4. The effective stiffness K_e of the analogous SDOF system at the peak response can be obtained from the simple harmonic motion equation for the natural period of SDOF system given as:

$$\boldsymbol{K_e = \frac{4\pi^2 m_e}{T_{eff}{}^2}} \tag{2.4}$$

Here m_e represents the effective mass of building that is participating in the first mode of vibration of the building. The design lateral force or design base shear can be determined using K_e and Δ_d as follows:

$$F = V_{base} = K_e\, \Delta_d \tag{2.5}$$

The design base shear estimated from equation (2.5) is then transferred to different floors in accordance to the product of mass and displacement as follows:

$$F_i = V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \tag{2.6}$$

Then the structure is analyzed using the force vector characterized by equation (2.6) to estimate the requisite flexural strength at the designated plastic hinges. Therefore, the design idea is quite straightforward. The complexity exists in determining the equivalent linear properties of substitute structure, estimation of the maximum design displacement, and formation of design displacement spectra.

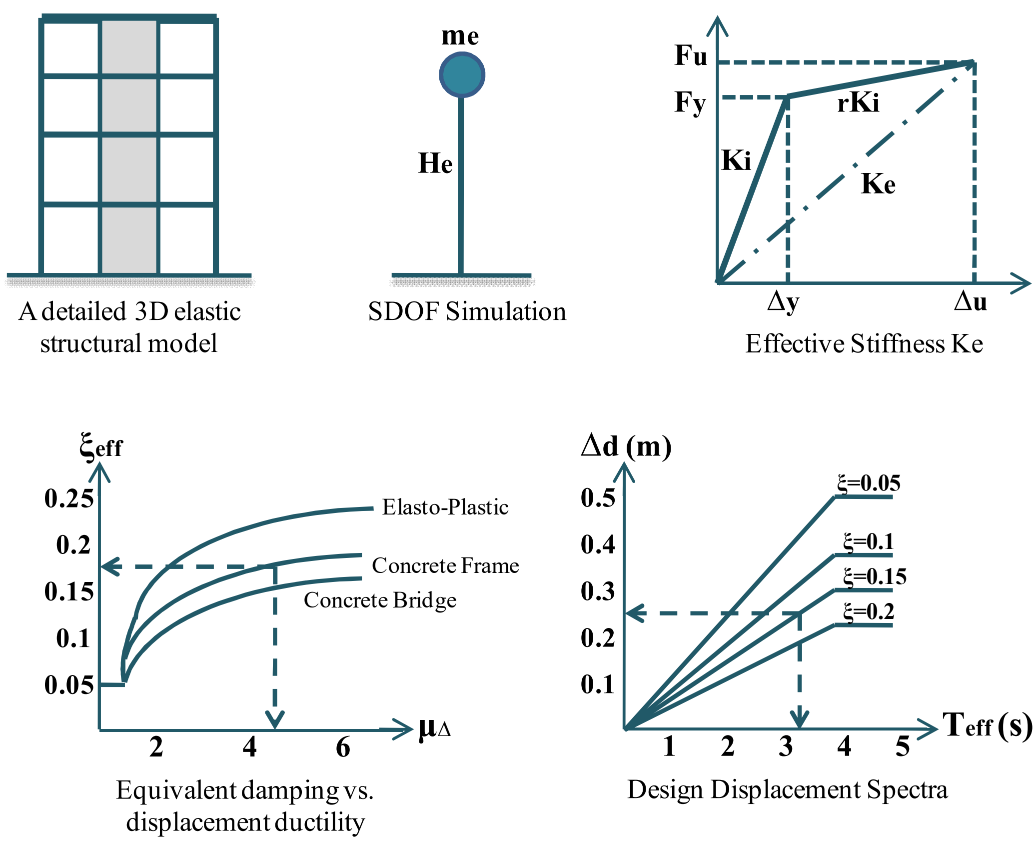


Figure 2-4: Fundamentals of the DDBD method

The procedure for the design and detailing of elements is as follows:

After the distribution of base shear force along the height of the building in proportion to the inelastic displacement profile, estimate the design element forces using equilibrium analysis. Estimate flexural demand on beams and columns due to design lateral force and design the beams for flexural demand of lateral force alone as the flexural demand arising due to gravity loads is not taken into account. Design the columns for axial loads estimated from gravity load analysis and design the columns against flexure demand estimated from lateral load analysis separately. As a result, this technique does not directly account for the change in the axial demand in columns caused by coupling actions occurring during strong ground motion, but it implicitly caters to this effect by increasing the flexural capacities of columns using capacity protection factors.

The DDBD method estimates the demand at specified plastic hinge locations to meet the design objectives in terms of target displacements. Capacity design procedures are required to ensure that plastic hinges form only at those locations where adequate detailing for ductility has been provided and brittle modes of deformation (shear failure of frame elements) don't occur. It is necessary to make sure that only the ideal beam sway mechanism is formed due to the higher energy dissipation capability of this mechanism. The design concept of weak beams-strong columns is justified by stiffness ratio as column's stiffness should be considered EI with no reduction for ductility and beam's stiffness should be reduced by some ductility factor. This ideal sway mechanism demands that plastic hinges in columns are allowed to occur only at the base and all other columns at any floor level are supposed to be linear elastic. The formation of plastic hinges in columns except at the base level may lead to the column sway mechanism rather than the beam sway mechanism. It is well known that failure of a single column might cause an entire collapse of the structure, while a single beam failure is doubtful to be critical.

A two-stage design scheme is required when masonry infill walls are used in frame structures. During serviceability level earthquake (SLE), infill failure does not occur but during the maximum considerable earthquake (MCE) only the bare frame is responsible to withstand the ground shaking. The DDBD method assumes that masonry infill walls have no structural importance during severe levels of the earthquake and the bare frame must be subsequently detailed to provide the required capacity. This method restricts the drifts to 0.005 for serviceability level earthquake and to $0.02 - 0.025$ for the maximum considerable earthquake in accordance with design codes. For the masonry infill RC frames, a linear elastic analysis should be performed with the masonry infill walls modeled along with other frame elements to get realistic stiffness estimates. The displacements at story heights using the mode shape and response displacement (displacement from response spectra 5% damped using the actual structure's period) are determined. The story drifts estimated from the displacement profile are then compared with the prescribed drift limit.

The theoretical concepts of the DDBD method discussed in this section can be applied easily to various types of structures including dual frame systems, core wall systems, and bridges etc. Several studies have used this approach to case study RC frame buildings and assessed its accuracy by comparing it to the detailed NLTHA procedure. These studies depicted that the DDBD method combined with the proper hysteretic models can fairly estimate the floor displacements, inter-story drift ratios, story shears, and story overturning moments. Detailed information on this technique and its validation can be found in Priestley [9].

2.6 Behaviour of Masonry Infill RC Frame Buildings

In the structural design of buildings, masonry infill walls are generally overlooked and considered non-structural elements. Two different approaches being used in the construction of RC infill frame buildings. One approach is to insert the plastic material between the RC frame and infill so that there's no connection between these two elements (RC frame and infill) and they can deform independently under the design hazard level besides at the base of the infill. This approach requires that some amount of reinforcement must be provided in infill walls along with the provision of out-of-plane support [31]. In the alternate approach, masonry infill walls are bonded to the frame elements and their structural interaction is considered in the design stage. Generally, the bonded infill walls are more vulnerable to damage because they are unreinforced and may fail at comparatively low drift values. In countries like Pakistan where later construction is in practice, the philosophy is that infill walls' failure is non-structural and may be repaired after an earthquake. Therefore, the behavior of infill walls is not accounted for in the code-based design procedures with the assumption that masonry infill walls have no structural significance [32].

The contribution provided by infill walls to structural performance shouldn't be ignored as these infill walls often act as a primary structural element and ignoring them will result in significant strength prediction inaccuracy [8]. The masonry infill walls affects the global seismic performance of the structure by imparting initial lateral translational stiffness, initial lateral bearing strength, and enhanced energy dissipation capacity at low drift levels [5]. Infill frames showed up to three times more stiffness and base shear relative to bare frames depending upon the no. of infill panels and their distribution in the structure [33]. When the distribution of infill walls is not proper in the structural plan, it may be the most hazardous situation. The non-uniform distribution of infill panels might cause undesirable consequences like additional torsional effects and brittle shear failure of columns [8]. At all costs, this scenario must be prevented. The presence of infill walls has also a major impact on building behavior at the local level; this interaction may change the distribution of internal forces leading to brittle collapse mechanisms of structural elements and even collapse of self-stable frame structures [34]. These infill walls are incapable to withstand lateral displacements without endangering the gravity load carrying capacity due to their non-ductile behavior. The response spectrum analysis procedure depicts that structure's fundamental natural period is considerably reduced due to the presence of infill panels leading to a change in demand forces on the structure [35]. The behavior of infill walls is strictly brittle in nature; therefore, it is not wise to model the infill walls with the linear elastic modeling technique.

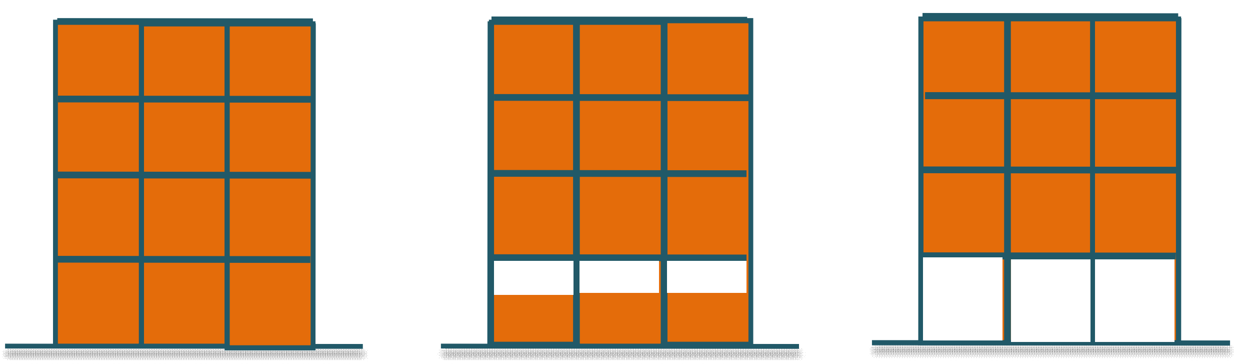


Figure 2-5: Different vertical arrangement of the masonry infill walls in typical frame buildings (a) infill walls present at all floor levels, (b) infill walls constructed up till a certain height of a story, (c) infill walls not present in a story

2.7 Summary

The presence of masonry infill walls in frame building increases the initial stiffness, lateral bearing strength, hysteretic modal damping, and energy dissipation capacities of the building at low drift values. It is useful for the buildings as long as the deformation capacities of masonry infill walls are not achieved. Under the higher level of ground shaking, the presence of infill walls can cause soft story mechanisms, brittle shear failure of columns, and even the failure of self-stable frames, if not included in the design properly. When the distribution of infill panels is not regular then it may cause additional torsion to the buildings leading to their brittle failure. The presence of infill walls can completely alter the damage characteristics and behavior of buildings under lateral loadings. To obtain the realistic results, masonry infill walls must be included in the modeling and design phase of the building.

Chapter 3

Methodology

In this study, a simplified displacement-based design (SDBD) procedure is proposed using the three case study buildings (3-, 4-, and 5-story high) and the accuracy of the SDBD procedure is investigated using the 5-story high case study building. Firstly, the selection of case study buildings is done then linear and nonlinear models of the buildings are developed using computer software. The case study buildings are infill RC frames located in the Islamabad Capital Territory (ICT) of Pakistan. The linear elastic models of the case study building are prepared in ETABS software using architectural drawings and other structural details; while their nonlinear models have been prepared in PERFORM 3D software. The results of the NLTHA procedure are used as a benchmark here to evaluate the SDBD procedure. Different sets of ground motions are selected and matched with different target response spectra to apply the NLTHA procedure. The results of the code-based RSA method are also included for comparison purposes. The results are compared for both local and global responses. The overview of the methodology is shown in Figure 3-1.

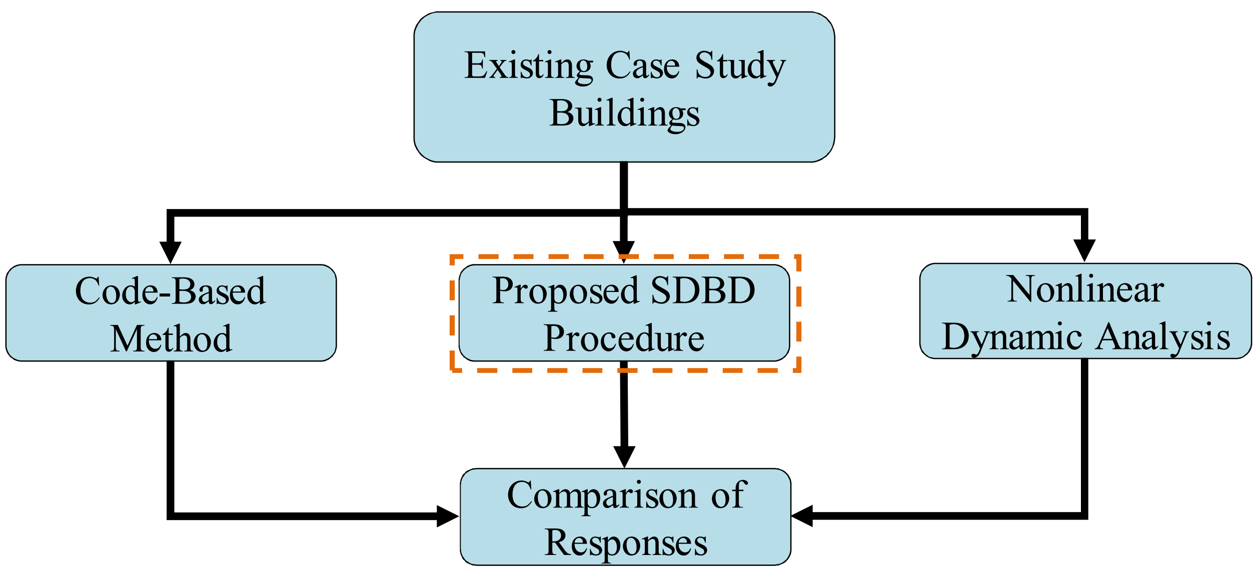


Figure 3-1: An overview of the methodology of this study

3.1 Description and Modeling of Case Study Buildings

The SDBD procedure has been proposed for designing new structures and evaluating the seismic performance of existing structures. As a design method, it aims to produce fairly accurate estimations of actual nonlinear seismic demands of new buildings to determine the requisite lateral capacity to withstand earthquake loadings. After establishing the required capacities at critical defined locations of structural elements as per demands determined from the SDBD procedure, the building can be reevaluated for seismic performance by utilizing the initial elastic characteristics.

In this study, three existing low-rise case study buildings with masonry infill walls are selected to propose this SDBD procedure. These buildings (3-, 4- and 5-story high, designated as B1, B2, and B3, respectively) are located in Islamabad, the capital city of Pakistan, and are already designed against gravity and seismic loads. They can represent typical existing RC frame buildings in many developing countries across the globe. In these case study buildings, the masonry infill wall thickness is 9 inches for both interior and exterior walls. A 5-story high RC infill frame building (designated as B4), also located in Islamabad, is used to evaluate the performance of the proposed SDBD procedure in nonlinear seismic demands estimation.

The typical floor plans in addition to 3D finite element (FE) models of all case study buildings are presented in Figure 3-3. The gravity loads are mainly resisted by RC beam-column frames along with RC slabs. The lateral load in these selected buildings is primarily resisted by beam-column frames. These buildings have isolated foundations at the base of columns. Masonry infill panels are used in the interior and exterior frames of all these buildings. Being practical and

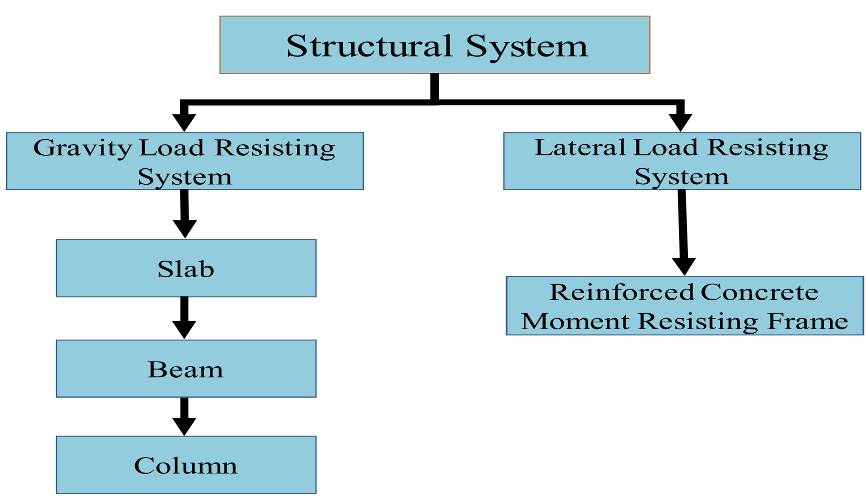
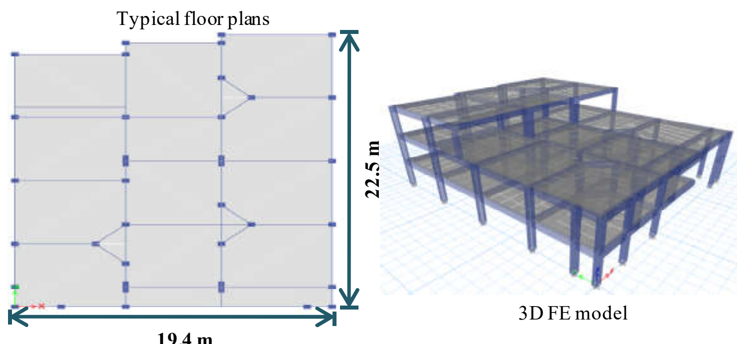
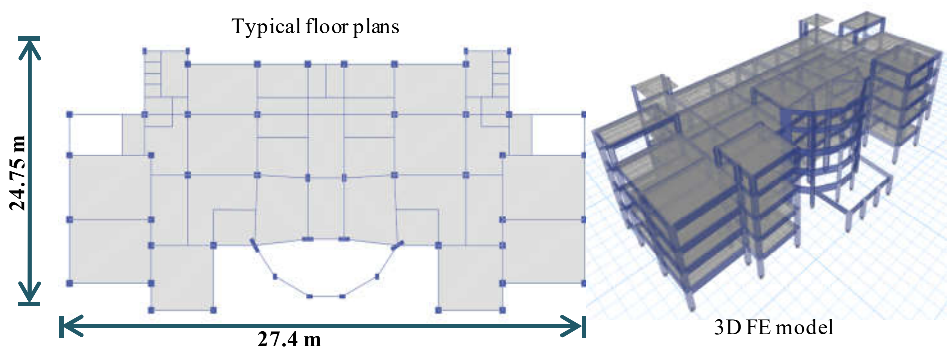


Figure 3-2: Structural system of case study buildings

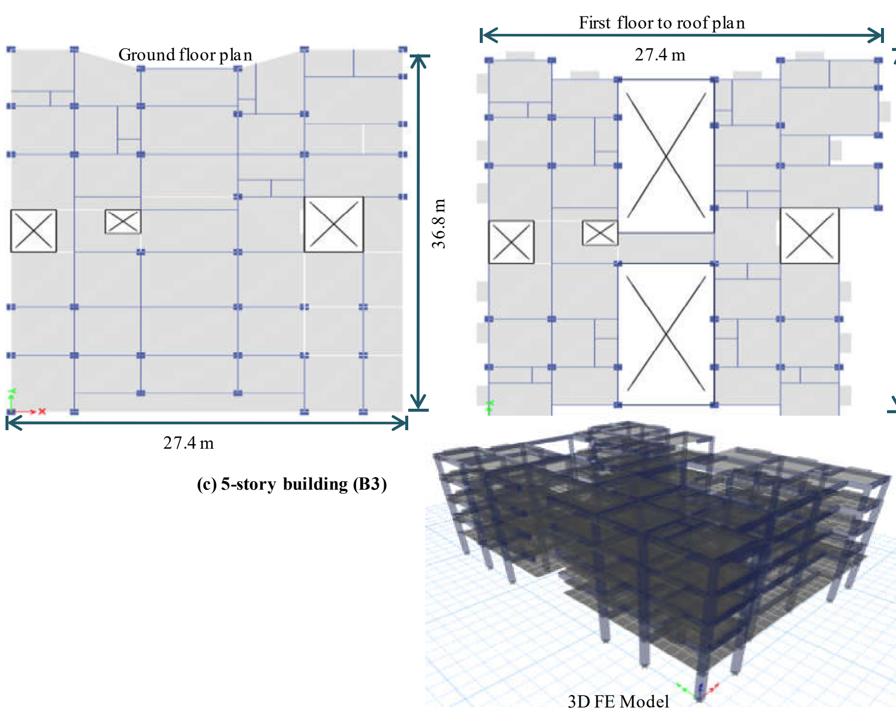
realistic examples, the case study buildings under consideration are regarded as appropriate to propose and examine the efficiency offered by the SDBD procedure. Major architectural and structural characteristics of selected buildings are presented in Table 3-1.



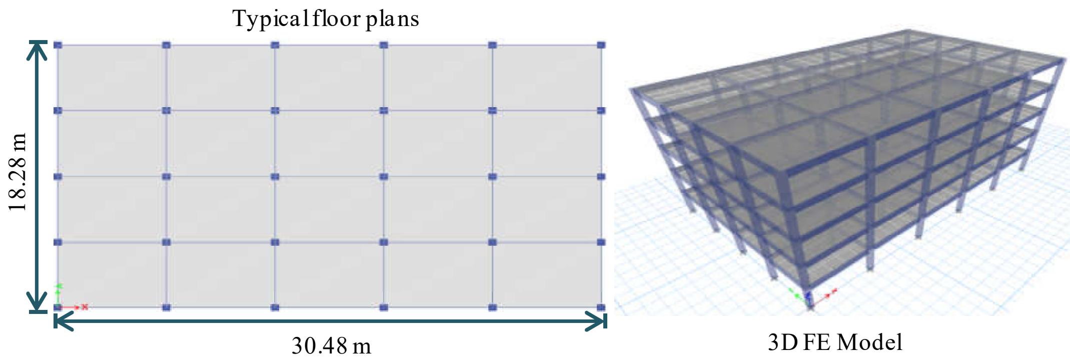
(a) 3-story building (B1)



(b) 4-story building (B2)



(c) 5-story building (B3)



(d) 5-Story Building B4

Figure 3-3: Plans and 3D views of case study buildings

Table 3-1: Salient features of case study buildings

Building			B1	B2	B3	B4
Height (m)			9.9	15.62	17.4	18.28
No. of stories			3	4	5	5
Typical story height (m)			3.3	3.28	3.35	3.65
Total footprint area (mxm)			19.35x22.52	51.81x24.75	27.38x36.8	30.48x18.28
Natural periods (sec)	X-direction	T1	0.2287	0.3702	0.4482	1.035
		T2	0.0823	0.1226	0.1388	0.3379
		T3	0.0598	0.0778	0.0718	0.1957
	Y-direction	T1	0.1902	0.3494	0.3987	0.7705
		T2	0.07602	0.1165	0.1317	0.2552
		T3	0.0577	0.0717	0.0759	0.1477
RC column area/ total foot- print area (%)			1.417	2.08	2.821	1.13
Infill thickness (m)			0.2286	0.2286	0.2286	0.2286
Typical column size (mm)			457x355	609x609	609x914	457x457

Linear elastic structural models of the case study buildings are developed in ETABS [11] software to carry out the SDBD procedure and code-prescribed RSA method. All the columns and beams are modeled as elastic frame elements while all the slabs are modeled as elastic shell elements. The masonry infill walls are modeled by utilizing the equivalent diagonal strut model.

All the infill walls are represented by equivalent concrete diagonal struts capable of developing the axial compression behavior only. The properties of masonry infill walls used to model these equivalent concrete struts in the context of Pakistan [36]–[39] are listed in Table 3-2. The isolated foundation at the base of the ground floor columns is considered to be fixed support. For the SDBD procedure, code-prescribed stiffness modifiers to account for cracking before yielding are assigned to structural frame elements to effectively estimate the natural periods and subsequent responses under seismic excitations. These preliminary natural periods in addition to the initial inherent 5% damping ratio are considered to estimate T_{eq} & ξ_{eq}, following the current EL scheme. The typical RSA method is also performed using the cracked stiffness modifiers prescribed by design codes.

Table 3-2: Properties of the masonry infill walls used in this study

Symbol	Properties	Value
$\boldsymbol{f_m}$	Masonry compressive strength (MPa)	4.4
$\boldsymbol{f_{tu}}$	Masonry tensile strength (MPa)	0.15
$\boldsymbol{E_m}$	Elastic modulus of masonry (MPa)	1310
$\boldsymbol{f_{vi}}$	Masonry shear strength (MPa)	0.27

The complete nonlinear FE models of these case study buildings are developed in PERFORM 3D [10] software to carry out the cyclic pushover analysis and the detailed NLTHA procedure. All the RC columns are modeled by distributed plasticity approach to save computational effort and time, using the nonlinear fiber elements at its two ends. To account for biaxial bending, fibers are created in both cross sectional dimensions of columns as illustrated in Figure 3-4. This fiber model accounts for axial-flexure interaction $(\mathrm{P}-\mathrm{M_x}-\mathrm{M_y})$ in both the axes. In this fiber modeling, the shear and torsion behavior is assumed to be elastic and uncoupled from the axial-flexure interaction. The plasticity is distributed at both ends for a length of D, where D is the depth of the column cross-section. The unassigned portion of column length is treated as linear elastic element. The concrete fibers of column elements are modeled by utilizing the Mander’s stress-strain model of unconfined concrete [40] idealized as a tri-linear behavior in PERFORM 3D, shown in Figure 3-4. The bilinear stress-strain model having the strain hardening effect is used to model the steel fibers as indicated in Figure 3-4.

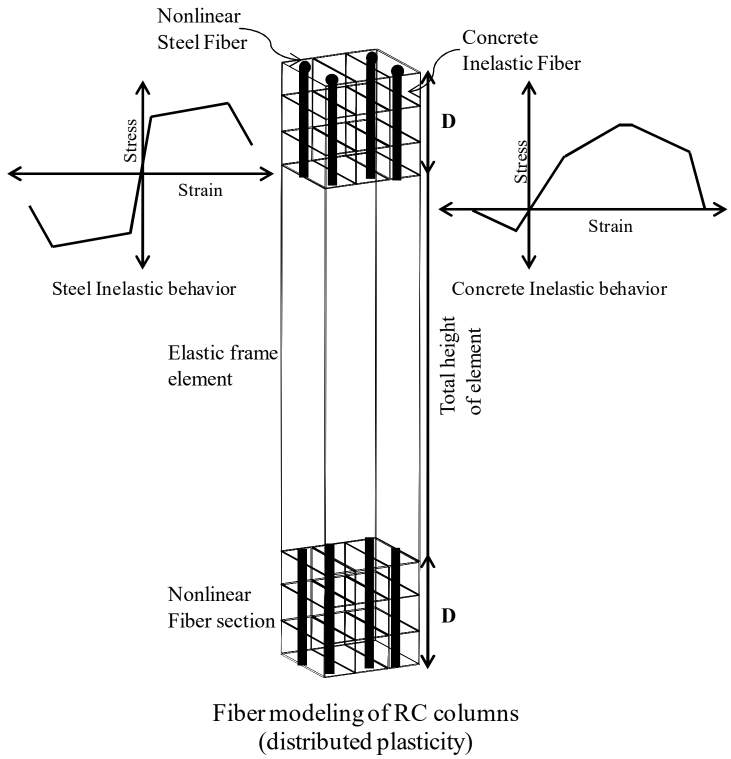


Figure 3-4: The approach used to model the RC columns nonlinear

All the RC beams are modeled using the moment rotation plastic hinge modeling approach following the ASCE-41 [12] with all inelastic behavior concentrated at ends defined by zero-length hypothetical elements (plastic hinges), as shown in Figure 3-5.

Several types of plastic hinges are available in PERFORM 3D software to specify the inelastic force-deformation behavior at certain locations of the member for any degree of freedom. Here, the moment M3 is considered critical in RC beam elements, and moment rotation $(\mathrm{M}-\theta)$rigid plastic hinge is used to assign nonlinear force-deformation behavior to this degree of freedom only. The remaining portion of frame elements is assigned to linear elements with effective stiffness. Figure 3-5 illustrates the key features of a flexure plastic hinge model at member level for RC beam element with inelasticity concentrated at ends. To completely define the plastic hinge, there's a need for a backbone curve and cyclic behavior. For cyclic degradation of concrete structural elements, the YULRX approach is used and parameters are taken from the [41] study. All the RC slab elements are considered to be elastic and modeled as elastic shell elements. The foundations are assumed to be fixed support beneath the ground floor columns.

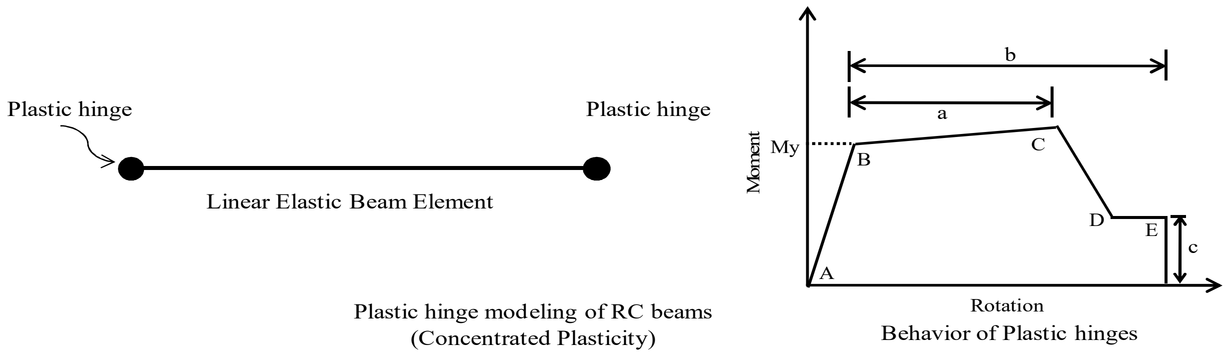


Figure 3-5: ASCE-41 modeling approach to model the RC beams nonlinear

Figure 3-6 illustrates that masonry infill panels are modeled with two equivalent concrete diagonal struts with brittle compression behavior only. The stiffness, axial strength, and nonlinear deformation capacity of all masonry infill walls modeled using diagonal struts are estimated from the material and geometrical properties of these walls as per the FEMA-356 [42] guidelines. The expected strength of material properties is used in all the nonlinear models of buildings considering that the true strength of materials usually exceeds the nominal strength assigned by the designer. To account for this, the yield strength of steel is multiplied by 1.17, and specified concrete compressive strength is multiplied by 1.3 per LATBSDC [43] guidelines.

The masonry infill panels as diagonal struts modeled using the FEMA-356 guidelines are capable only to develop the in-plane compression behavior only. The equivalent concrete strut has the thickness and modulus of elasticity of the infill panel under consideration. The width of the diagonal strut a is given by the following equation:

$$a = 0.175\ (\lambda_1 h_{col})^{-0.4} r_{inf} \tag{3.1}$$

Where:

$$\lambda_1 = [\frac{E_{me} \times t_{inf} \times \text{Sin}2\theta}{4E_{fe} \times I_{col} \times h_{inf}}]^{0.25} \tag{3.2}$$

Here, h_{col} is the height of the column between centerlines of beams, λ_1 is coefficient that is used to find the equivalent width of the diagonal struts, r_{inf} is the diagonal length of the infill walls, E_{me} is the elastic expected modulus of the material of infill panels, t_{inf} is the thickness of infill panels, θ is the angle in radians whose tangent is height to length of infill panels, E_{fe} is the expected modulus of elasticity of the material of frame elements, I_{col} is the moment of inertia of column elements, and h_{inf} is the height of masonry infill panels.

The area A_{inf} of the diagonal concrete strut is determined using the following equation:

$$A_{inf} = \mathrm{a} \times t_{inf} \tag{3.3}$$

The stiffness of the equivalent strut is estimated as:

$$K_{inf} = \frac{E_{me}(\mathrm{a} \times t_{inf})}{r_{inf}} \tag{3.4}$$

The strength of the compression strut is determined as:

$$Q_{CE} = A_{inf} \times f_{vie} \tag{3.5}$$

Here, Q_{CE} is the expected shear strength of concrete strut and f_{vie} is the expected shear strength of masonry infill panels.

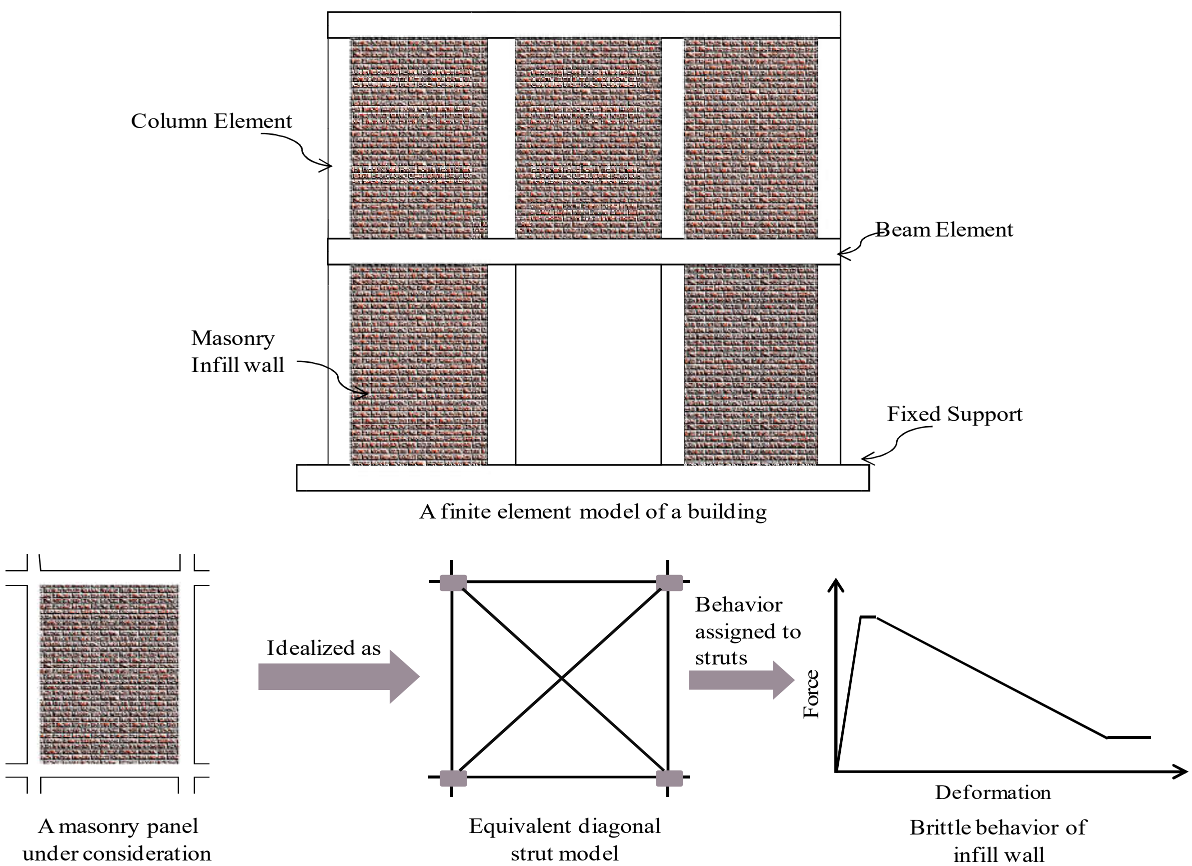


Figure 3-6: Modeling of masonry infill walls using diagonal struts

3.2 Selection of Representative Ground Motions

The selected case study buildings situated in Islamabad Capital Territory (ICT) are expected to experience a wide variety of seismic excitations in their intended design life. The ICT lies in the region of Pakistan that is seismically active and has faced several earthquakes of different magnitudes in past years. The two faults that mainly contribute to the seismicity of ICT are main boundary thrust and strike-slip faults [44]. In this study, a suite of seven ground motion histories for these two fault types having magnitudes in the range of M 6.3–7.8 are employed to carry out the detailed nonlinear time history analysis (NLTHA) procedure. These ground motion records are accessed using the NGA-West2 coast of (PEER ground motion database) and are most likely to occur within 10-50kms of the source to site distance with the site undergoing 490-620m/s velocity of shear waves. Table 3-3 shows the criteria used to select the ground motions for the Islamabad Capital Territory. Table 3-4 shows the details of the ground motion parameters of representative ground motions while Table 3-5 shows the details of the ground motions in comparison to Table 3-3. These time histories of ground motions are adjusted and scaled by spectral matching in SEISMOMATCH software to match with the target response spectra of UBC-97. The target spectrum is developed for a 5% damped design base earthquake (DBE) level for the region of ICT with $C_v = 0.32g$ and $C_a = 0.24g$ [1]. Figure 3-7 shows the response spectrum for the seismic hazard of ICT region. Figure 3-8 displays the response spectra of adopted time histories in their original form. Figure 3-9 illustrates the matched response spectra of selected ground motions with the target response spectrum. The case study buildings are expected to achieve various deformation and damage levels when subjected to these ground motions, therefore, these earthquake records are considered suitable for the evaluation of the proposed SDBD procedure for the low-rise masonry infill RC frame buildings.

Table 3-3: Representative parameters for the ground motion selection from the database

Parameters	
Fault type	**Strike slip + Reverse**
Magnitude	**6.3 to 7.8**
R_{rup} (km)	**10 to 50**
V_{S30} (m/s)	**490-620**
Duration(sec)	**15-60**

Table 3-4: Ground motion parameters of time histories used in this study

Parameter	Direction	GM-1	GM-2	GM-3	GM-4	GM-5	GM-6	GM-7
PGA (g)	H1 H2	0.174 0.224	0.289 0.237	0.135 0.127	0.252 0.212	0.242 0.138	0.189 0.119	0.165 0.202
Max Velocity (cm/s)	H1 H2	24.21 18.52	35.25 39.71	10.23 14.11	37.03 36.17	22.09 12.94	13.46 11.71	12.19 12.5
Max Displacement (cm)	H1 H2	9.86 9.19	11.11 25.67	7.67 8.05	38.60 21.93	8.49 6.95	6.46 7.36	6.83 7.05
Arias Intensity (m/s)	H1 H2	0.664 0.715	0.796 0.87	0.576 0.425	1.139 0.924	0.26 0.11	0.31 0.20	0.429 0.402
Specific Energy Density (cm2/s)	H1 H2	958.6 798.4	2134 5915	424.8 452.3	7255 5397	878.7 396.4	383.3 278.1	526.1 851.8
Cumulative absolute velocity (cm/s)	H1 H2	1038 1005	1067 1082	969.9 857.7	1215 1152	450.3 354.8	516.7 447.3	602.2 631.4
Housner Intensity (cm)	H1 H2	71.52 75.35	116.7 125.3	43.17 49.65	102.5 100.7	91.81 45.19	53.02 44.23	42.16 40.62
Sustained max acceleration (g)	H1 H2	0.146 0.182	0.17 0.198	0.13 0.108	0.174 0.169	0.125 0.06	0.154 0.092	0.147 0.138
Effective Design acceleration (g)	H1 H2	0.174 0.223	0.28 0.235	0.135 0.126	0.251 0.211	0.241 0.132	0.181 0.120	0.166 0.200
A-95 parameter	H1 H2	0.172 0.221	0.286 0.234	0.132 0.128	0.249 0.209	0.24 0.137	0.186 0.116	0.162 0.198
Predominant period (s)	H1 H2	0.44 0.28	0.46 0.62	0.26 0.2	0.32 0.26	0.32 0.18	0.42 0.22	0.16 0.16
Mean period (sec)	H1 H2	0.6 0.56	0.88 0.999	0.46 0.5	0.63 0.67	1.02 1.02	0.61 0.55	0.42 0.55
Damage index	H1 H2	0.827 0.98	0.628 0.592	0.82 0.645	1.326 1.029	0.242 0.107	0.399 0.312	0.893 0.845
No of effective cycles	H1 H2	4.85 3.53	1.48 1.40	6.38 5.14	6.33 5.64	0.91 1.84	2.30 5.21	4.66 4.15
IP index	H1 H2	44.88 56.99	31.05 27.94	97.75 62.90	33.62 32.68	21.88 29.96	39.78 39.89	50.77 51.53
Average Spectral acceleration (g)	H1 H2	0.175 0.19	0.278 0.286	0.103 0.121	0.247 0.239	0.211 0.11	0.129 0.112	0.108 0.105
Significant Duration (s)	H1 H2	29.8 26.3	32.27 24.12	31.92 31.15	19.8 21.35	13.65 21.73	18.19 21.13	12.6 20.28

Table 3-5: Details of selected ground motions to perform the NLTHA procedure

Event	Year	Station	Magnitude	Mechanism	R_{rup}(km)	V_{S30} (m/s)
Chichi Taiwan	1999	CHY010	7.62	Reverse oblique	19.96	538.69
Chichi Taiwan	1999	CHY029	7.62	Reverse oblique	12.65	573.04
Chichi Taiwan	1999	CHY087	7.62	Reverse oblique	28.91	505.2
Chichi Taiwan	1999	TCU042	7.62	Reverse oblique	26.31	578.98
Chichi Taiwan_06	1999	CHY029	6.3	Reverse	41.36	544.74
Chuetsu	2007	Nadachiku Joetsu city	6.8	Reverse	35.93	570.62
Iwate Japan	2008	Misato Akita City	6.9	Reverse	41.72	552.38

R_{rup}: The closest distance from the site to the earthquake rupture plane

V_{S30}: The average seismic shear wave velocity from the surface to a depth of 30 meters

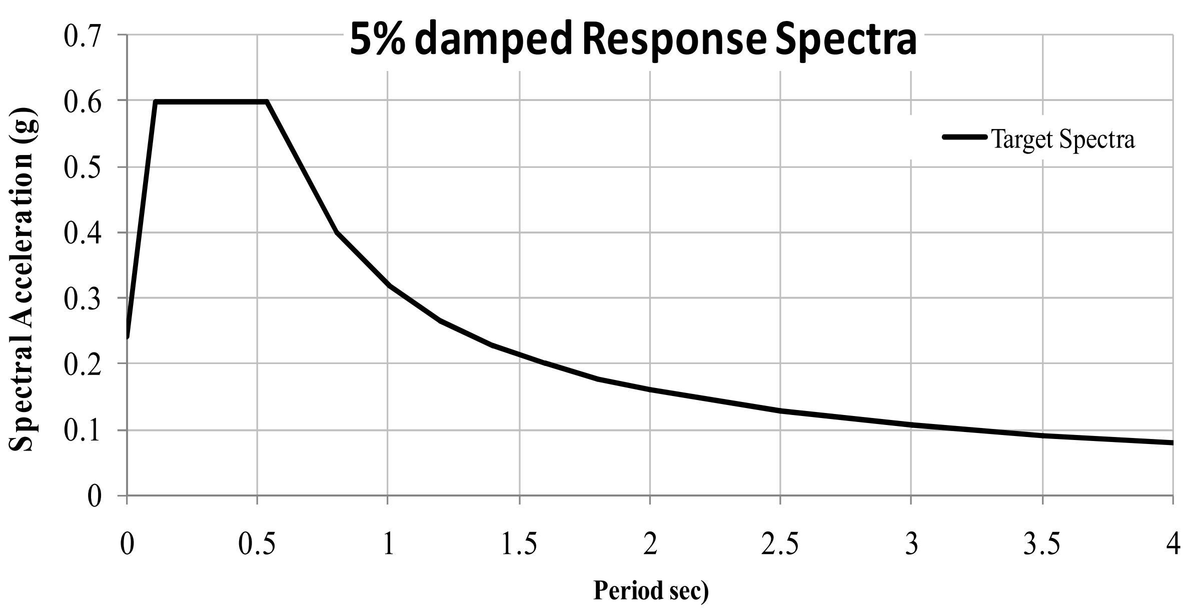


Figure 3-7: Target response spectrum to which ground motions spectra is matched

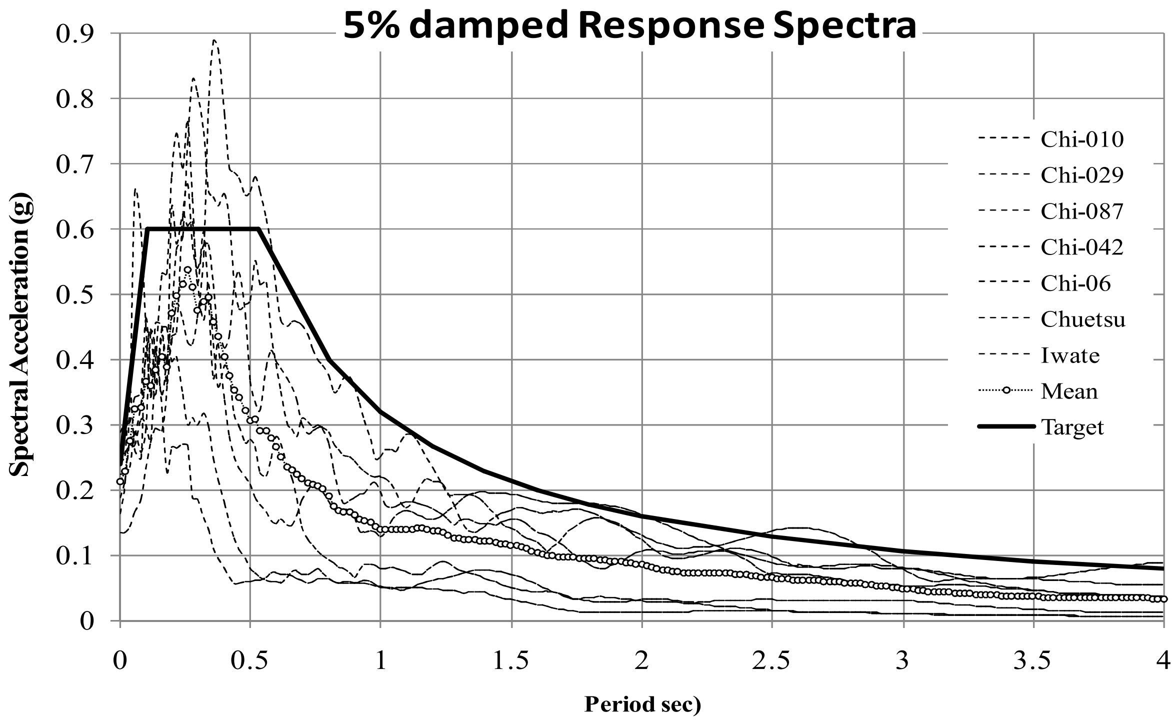


Figure 3-8: The unmatched acceleration response spectra of ground motions

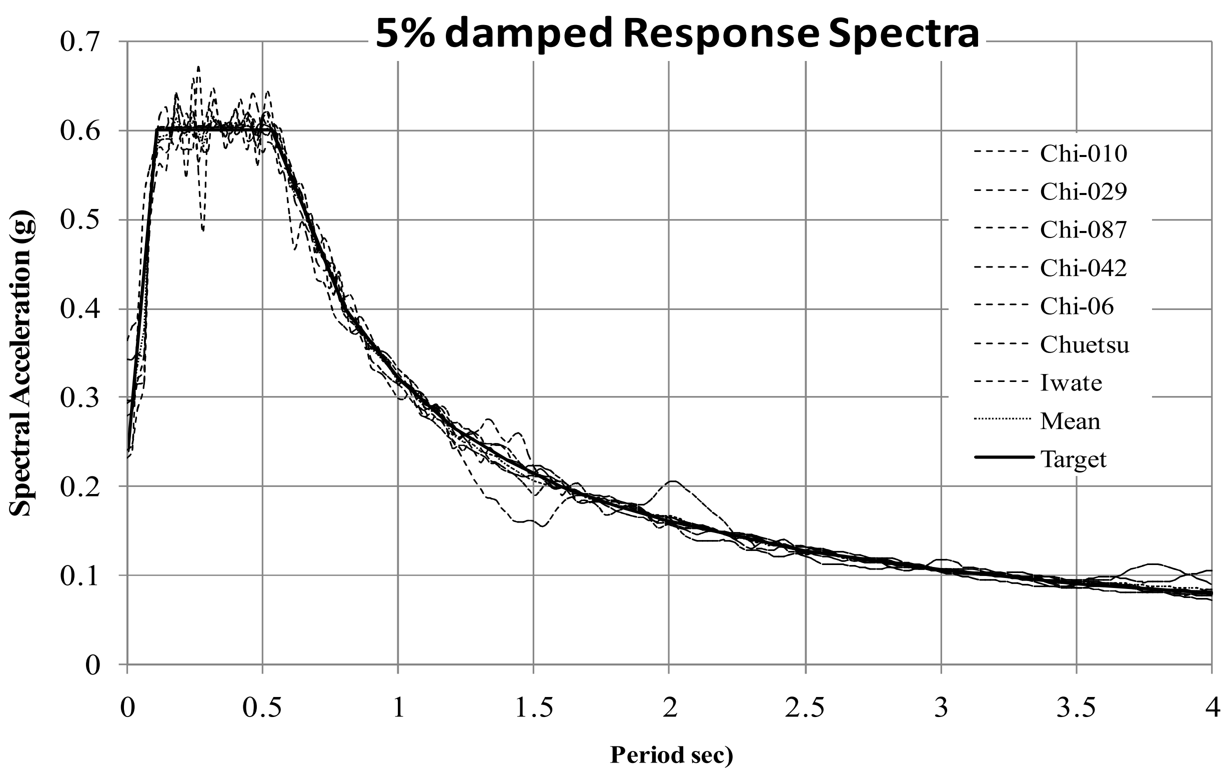


Figure 3-9: The acceleration response spectra of selected ground motions matched with the target spectrum

3.3 The Proposed Simplified Displacement-Based Design (SDBD) Procedure

As mentioned earlier, the most common type of structure in Pakistan is masonry infill RC frames and the structural failures of these infill frames can be frequently seen in Pakistan during earthquakes.

To capture the effects imposed by infill walls on structural response, there are macro- and micro-models available which can be used depending upon the circumstances. Generally, micro-models are utilized when there's a need to capture the local effects in adjacent RC frame members caused by infill panels, and if the overall global structural response of infill frames is focused then macro-models are employed. Macro-models are efficient and offer obvious simplicity in terms of computation. The macro-models account for reasonable stiffness representation of masonry infill panels in their formulation. In the preliminary level of seismic response, an infill wall that is built in contact with the frame elements on all sides acts as a diagonal strut. This study also employs the diagonal struts macro model having compression behavior only as masonry infill walls are non-ductile in nature. This diagonal strut model provides the overall satisfactory stiffness estimation of infill frames and the axial forces induced in frame elements due to excessive in-plane stiffness of infill panels under lateral loadings [45]. When subjected to earthquakes, significant compressive stresses induce at the ends of the masonry strut and the other two corners are subjected to tensile stresses which lead to the separation of the infill wall and the frame. The further increase in lateral loading causes the infill panels to crack in various patterns. Typically at 0.01 drift angle, failure of infill walls occur and structure action changes from braced frame to bare frame with no structural significance of infill walls afterward. The gross stiffness of the structure is significantly increased due to the presence of infill panels and initial periods are relatively low compared to the bare frames [8].

The RC frame elements and masonry infill panels must dissipate the same amount of energy in the actual nonlinear system and the equivalent linear system to be consistent with the equivalent linearization (EL) technique [46]. A substantial number of equivalent linearization (EL) approaches [47], [48] have been presented in the past few decades that provide fairly accurate approximations of equivalent linear characteristics $(T_{eq}\ \&\ \xi_{eq})$. Numerous subsequent studies extended the EL procedures to incorporate the effect of various hysteretic behaviors along with their governing parameters. It may not be appropriate to simply use an existing EL approach with its underlying assumptions at this initial stage of the simplified displacement-based design procedure for masonry infill RC frames. In particular, applying the fundamental EL concept directly to specific case study buildings and evaluating the efficiency of the proposed SDBD procedure is more appropriate. The idea can be developed further if reasonably accurate estimations of nonlinear seismic demands are obtained.

In this study, the most familiar and conventional scheme to determine the substitute structure characteristics or equivalent linear properties is followed, known as the equivalent natural period T_{eq} approach. The original nonlinear structure is replaced with an equivalent linear structure with the belief that both the structures are at the same displacement response level. The equivalent period T_{eq} from the secant stiffness K_e at the given maximum nonlinear displacement can be determined using equation (2.4), leaving the additional damping estimation problem. The total equivalent viscous damping ξ_{eq} of a structure is the addition of initial inherent elastic damping ξ_{el} and hysteretic damping ξ_{hys} of inelastic response as given:

$$\xi_{eq} = \xi_{el} + \xi_{hys} \tag{3.6}$$

The equal energy principle, initially proposed by Jacobsen [49], can be implemented to determine the amount of hysteretic damping during inelastic response. Jacobsen suggested that an equivalent linear system having additional damping can simulate the steady state response of the original nonlinear system. According to Jacobsen's original concept, both equivalent linear and nonlinear systems have the same initial stiffness, both systems are subjected to the constant amplitude of sinusoidal excitation, both systems are at resonance, and both systems dissipate the same amount of energy in each cycle. These premises are rarely satisfied as actual ground motions have diverse frequency content and are non-harmonic in nature. The assumption of both the systems being at resonance is absolutely not compatible with the cyclic modal pushover analysis to determine the hysteretic damping ξ_{hys} but studies demonstrated that it is a reasonable approximation to believe that hysteretic damping ξ_{hys} is independent of the loading frequency [50]. The equivalent damping predicted the displacements under seismic loadings in fine accordance with time history results for systems having low energy dissipation in hysteretic responses like the Takeda model for concrete frame structures [51].

This study has adopted Jacobsen's linearization approach; however, Jacobsen's damping was determined using the secant stiffness rather than initial stiffness. Such a technique was initially presented by Rosenblueth [47], which was then extensively used in other numerous studies. Therefore, this study is compatible with the underlying concepts of characterizing the structure by viscous damping and stiffness at the peak response level. The following expression is used for the estimation of the equivalent hysteretic damping:

$$\xi_{hys} = \frac{E_D(\Delta)}{4\pi E_{so}(\Delta)} = \frac{E_D(\Delta)}{2\pi F_o \Delta_o} \tag{3.7}$$

Here E_D is energy dissipated within one full cycle of stabilized force-displacement response (area of that cycle) and F_o & Δ_o are the maximum inelastic forces and peak displacements respectively, that can be obtained in each hysteretic cycle. And $E_{so}(\Delta)$ represents strain energy corresponding to an equivalent linear system, which is determined as the area of the triangle of the force-deformation curve. It should be noted that this equivalent damping ξ_{eq} or $E_{so}(\Delta)$ is proportional to the secant stiffness K_e at the peak response. To ensure equal energy dissipation, a higher assumed equivalent stiffness leads to lower equivalent damping, whereas a lower choice of equivalent stiffness provides a higher level of damping. It implies that the viscous damping of a system is influenced by the choice of stiffness. Because nonlinear and equivalent linear systems are expected to have the same damping force, the inherent initial damping employed in any EL approach involving the secant stiffness should preferably be modified to make the results compatible with the NLTHA procedure results. However, this modification is not taken into account in this study, and equivalent damping is estimated by the addition of initial elastic damping ξ_{el} used in time history analysis, and the hysteretic damping estimated from hysteretic loops. This adopted methodology may not be the most accurate to estimate the equivalent linear characteristics amongst the other known EL approaches. However, it keeps the theoretical base and conceptual clarity; thus, it may be considered suitable for the evaluation of the proposed simplified displacement-based design (SDBD) procedure for infill frame buildings.

The development of general relations estimating the equivalent linear characteristics with an appropriate deformation parameter is of prime importance to ensure the practical implication of the SDBD procedure in design offices. In the DDBD method, general relations to determine the equivalent linear characteristics are established as a function of ductility ratio ($\mu_\Delta = \Delta_d / \Delta_y$, where Δ_y is the yielding displacement) for different structural systems. But this study is aimed at developing the generalized relationships to determine the equivalent linear properties as a function of roof drift ratio instead of ductility ratio for RC infill frames. Current studies ([52], [53] have indicated that significant nonlinearity can be induced during the cyclic response of high-rise RC structures due to tensile cracking of shear walls before the yielding of steel reinforcement in shear walls. Tensile cracking of RC shear walls resulted in considerable stiffness softening at relatively lower displacements than the yield displacement. In such situations where there's no well-defined yield point of buildings, the ductility ratio may not be an appropriate parameter to represent the nonlinear structural state. Therefore, the roof drift ratio can be used as a more appropriate and meaningful index to represent the nonlinearity of a structure.

The fundamentals of the SDBD procedure are illustrated in Figure 3-10 for the infill RC frame structures, while the primary concept implements on any structural configuration. The RD_{el} is the roof drift ratio of the building when it is in the linear elastic range, and T_{el} & ξ_{el} are corresponding elastic natural period and damping characteristics of the building. The initial natural period T_o of the building can be determined from the modal analysis of the linear elastic model. The period T_o is used as an input to the acceleration response spectrum developed for ξ_{el} damping to estimate the seismic demands.

The acceleration response spectrum can be converted into the displacement response spectrum using the following equation:

$$\boldsymbol{S_d = \frac{1}{\omega^2} S_a = \frac{T^2 S_a}{4\pi^2}} \tag{3.8}$$

Here S_d represents the spectral displacement, S_a represents the spectral acceleration, and T shows the natural period of the building.

The elastic roof drift RD_{el} can be determined from spectral parameters by the use of this equation as follows:

$$\boldsymbol{RD_{el}} = \frac{\boldsymbol{\Gamma_1}}{\boldsymbol{H}}\boldsymbol{S_d} = \frac{\boldsymbol{\Gamma_1.T^2.S_a}}{\boldsymbol{4\pi^2.H}} \tag{3.9}$$

Here Γ_1 represents the modal participation factor of first vibration mode and H is the total height of the building.

The trial roof drift, usually RD_{el}, is used to get a trial pair of $T_{eq,i}$ & $\xi_{eq,i}$ from the corresponding relationships. The response spectrum is reduced for the pre-determined $\xi_{eq,i}$, and the roof drift RD_i corresponding to $T_{eq,i}$ is obtained. The roof drift ratio RD_i is updated by again estimating the spectral displacement S_d associated with the trial $T_{eq,i}$ & $\xi_{eq,i}$. This process is repeated until the initial value of RD_i for the iteration converges to the resulting RD_i yielding the final equivalent linear characteristics of the building. The convergence has occurred in a maximum of three trials for the case study buildings. Equation (2.4) is employed to obtain the secant stiffness K_e from the equivalent natural period T_{eq} . The final roof drift ratio RD is converted into design displacement Δ_d using the total height of the building H as follows:

$$\Delta_d = RD \times H \tag{3.10}$$

The design lateral force on the building is determined using equation (2.5). The determined design force is distributed to different floor levels as per the distribution method based on the first mode of vibrations as follows:

$$F_x = V_{base}\frac{(w_x h_x)}{\sum_{i=1}^{n}(w_i h_i)} \tag{3.11}$$

Here, F_x is the lateral force at level x of the building, w_x & w_i are effective weights of the building at level x or i, and h_x & h_i are heights of the building at level x or i. The structure is then analyzed using the force vector characterized by equation (2.6) to evaluate its performance.

In a nutshell, this proposed procedure can be easily carried out without developing the nonlinear model and performing the cyclic MPA to get the hysteretic responses for RC infill frame buildings.

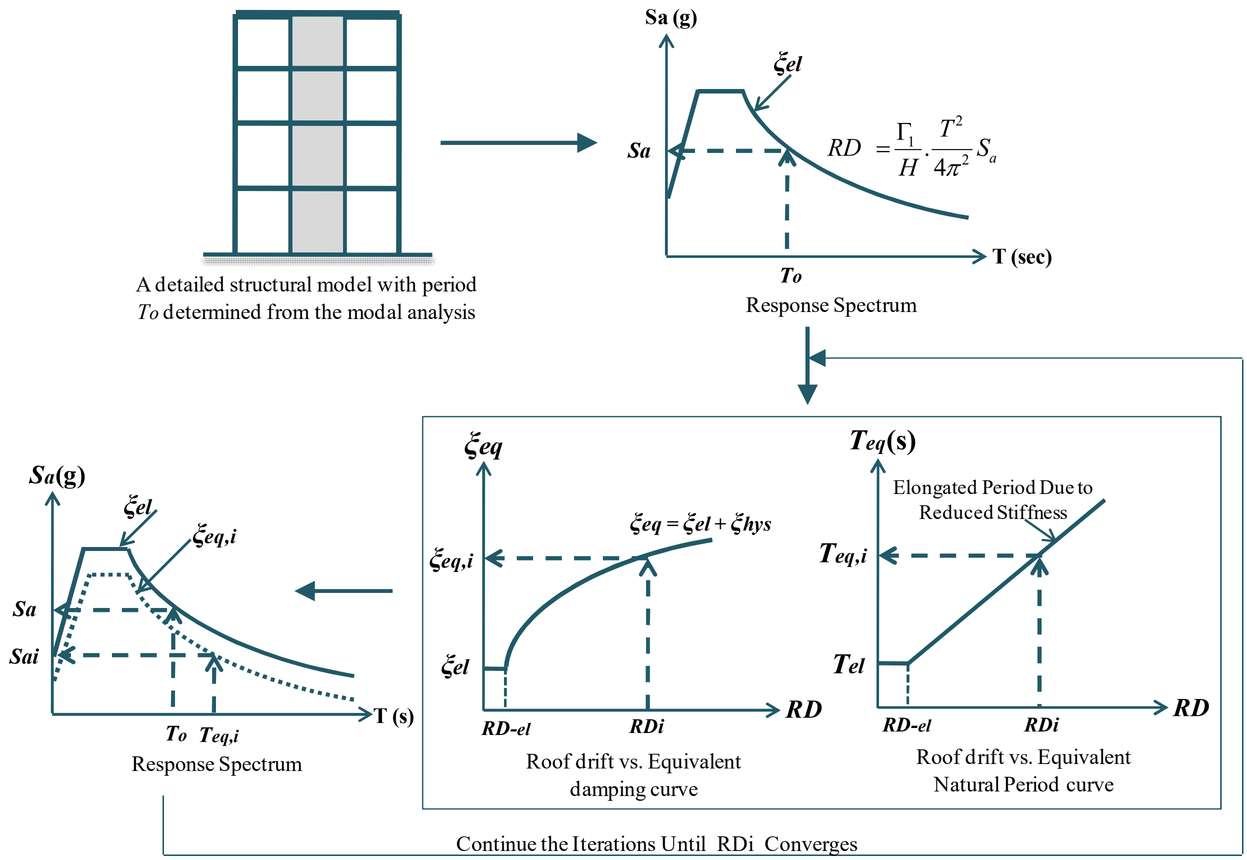


Figure 3-10: Basic concepts of the SDBD procedure

Chapter 4

Results & Discussions

The effects of the masonry infill panels on the behaviour of buildings are shown & discussed which supports the literature. The equivalent linear properties of case study buildings are determined using the equal energy principle and point-by-point conversion of the stiffness of the envelope curve into the natural period. The equivalent linearization approach gives the convenience of the linear static analysis plus the determination of equivalent properties of the building under consideration.

The detailed elastic and inelastic models of all the case study buildings are subjected to the earthquake demands using the different seismic analysis & design procedures under consideration. The results of the standard code-prescribed RSA method, the proposed SDBD procedure, and the detailed NLTHA procedure are compared to evaluate the accuracy of the proposed SDBD procedure. This study does not focus on the flaws of the standard RSA method rather it focuses on the investigation of the proposed design & analysis scheme. Both local and global responses of the case study building (B4) are included in the comparison and insightful discussion is made on them. The following global responses are determined and compared with seismic approaches:

- Story Displacements
- Inter-story Drift Ratios
- Story Shears
- Story Moments

The following local responses are estimated and compared:

- Exterior column shears
- Interior column shears
- Exterior column moments
- Interior column moments

4.1 Effects of Masonry Infill Walls

4.1.1 Effect of Infill Panels on Natural Period of Buildings

To examine the effects of masonry infill walls on the behavior of frame structures, 3-story (B1) and 4-story (B2) case study buildings are selected for this investigation. Table 4-1 shows the comparison of B1 & B2 case study buildings with and without masonry infill walls. It is clearly visible that infill walls can reduce the fundamental natural period of buildings significantly, as supported by the literature. When natural period of the building is reduced then the forces on the building change significantly in any code-based procedures.

Table 4-1: Effects of infill walls on the natural period of buildings

Building		3-Story	4-Story
Natural Periods (sec)	No Masonry	0.6464	0.8228
	Masonry	0.2287	03702

4.1.2 Effect of Infill Panels on Monotonic Pushover Analysis

Figure 4-1 shows the comparison of the normalized base shear vs. roof drift ratio including and excluding the masonry infill walls. It can be seen that the presence of infill panels has significantly increased the base shear of the B1 & B2 case study buildings. The distribution of infill panels in the RC frames is of prime importance to avoid additional torsion on the building.

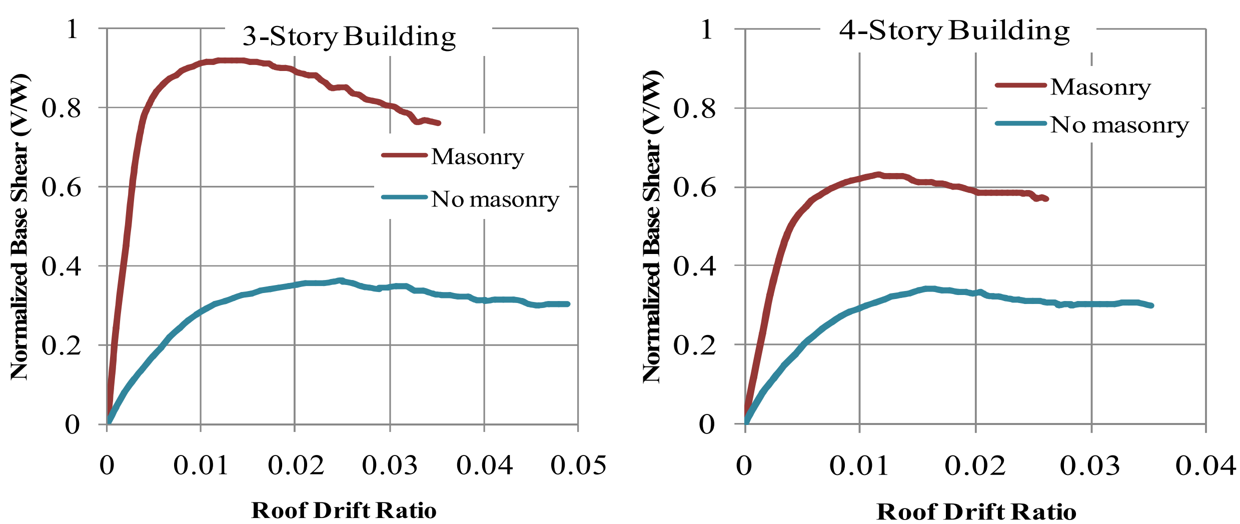


Figure 4-1: Effects of infill walls on the base shear of buildings

4.1.3 Effect of Infill Panels on Cyclic Pushover Analysis

Previous studies showed that the masonry infill panels can improve the initial stiffness, lateral strength, and energy dissipation properties of RC frame structures at low drift levels. The placement of the masonry infill walls should be regular in the structural plan & elevation and they do not cause brittle shear failures of column elements. Masonry infill panels act as a damping source in the frame buildings. The damping in concrete buildings arises due to the energy dissipation through various mechanisms such as cracking of concrete and sliding between non-structural & structural members. The masonry infill panels can act as active dampers when strengthened with fiber-reinforced polymers. The polymers increase the frictional forces between the infill walls which results in additional damping. Figure 4-2 shows the cyclic modal pushover curves of the 3-story building (B1) with and without infill walls. It is expressed that infill frames have high energy dissipation capacity as compared to bare frames.

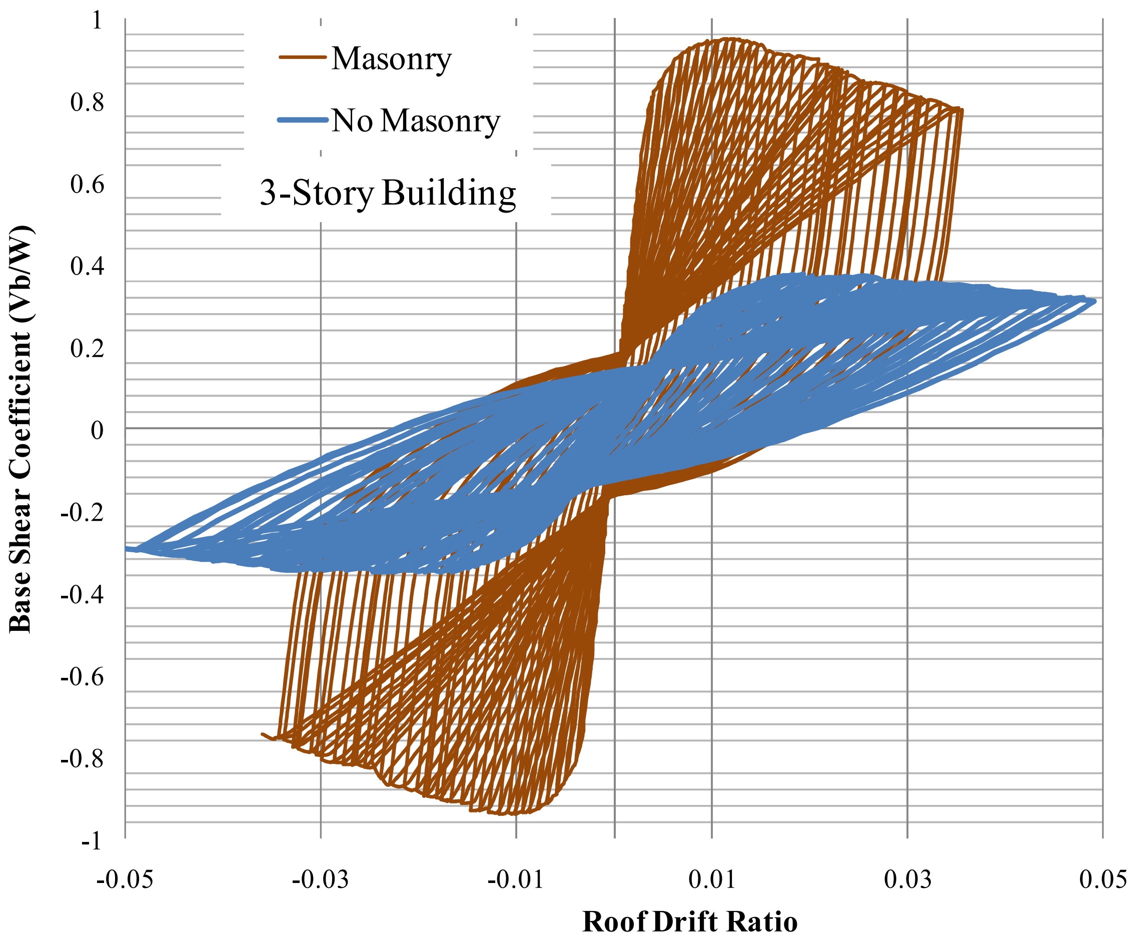


Figure 4-2: Cyclic modal pushover analysis of B1 with / without infill walls

4.2 Computation of Equivalent Properties & Modeling Recommendations

Using the adopted EL scheme, any nonlinear SDOF system governed by the modal hysteretic response can be converted into an equivalent linear system. To determine the equivalent linear properties as a function of roof drift ratio, nonlinear models of the case study buildings are subjected to cyclic modal pushover analysis (MPA) for the 1st mode in the X-direction. In cyclic MPA, each incremental load cycle was implemented on case study buildings with the stiffness characteristic of the previous load cycle end. The resulting cyclic pushover curves (base shear vs. roof drift ratio) are used to develop the T_{sec}/T_o versus roof drift ratio relations as shown in Figure 4-4 (where T_o is the initial period). Similarly, hysteretic damping ξ_{hys} vs. roof drift ratio relations are also developed using the cyclic pushover curves to estimate the equivalent characteristics in this proposed SDBD procedure. Even though these buildings have different total heights, footprint areas and, structural arrangements, the sequence of damage accumulation and global hysteretic responses of these buildings are highly comparable. This suggests that comprehensive relationships to determine the equivalent linear characteristics as a function of suitable deformation measure (e.g. roof drift ratio) can be developed.

In the X-direction of case study buildings, Figure 4-3 illustrates the base shear coefficient vs. roof drift ratio relations for their first mode. The monotonic pushover curves (base shear vs. roof drift) of buildings are converted into force-displacement curves. The slope of these force-displacement curves is secant stiffness K_{sec} which is converted into T_{sec} by using the effective modal mass and modal participation factor of that mode. This resulted in T_{sec}/T_o versus roof drift ratio relationships as depicted in Figure 4-4 for all three case study buildings. It can be observed that all these buildings have a comparable trend of effective period normalized to the initial period. Therefore, a generalized equation can be developed to determine the equivalent period for masonry infill RC frames. The following equation is developed in this regard:

$$T_{eq} = T_o\left[\frac{15(RD_{Total} - RD') + 0.14}{(RD_{Total} - RD') + 0.127}\right] \tag{4.1}$$

Here, RD_{Total} is the total roof drift of the building where the equivalent period is required to be determined. RD' is the roof drift ratio beyond which nonlinearity starts in the building and T_o is the initial natural period of the building. This equation is developed using the average curve of T_{sec}/T_o of three case buildings and is valid to calculate the equivalent period of infill frames when this condition satisfies:

$$RD_{Total} - RD' \geq 0 \tag{4.2}$$

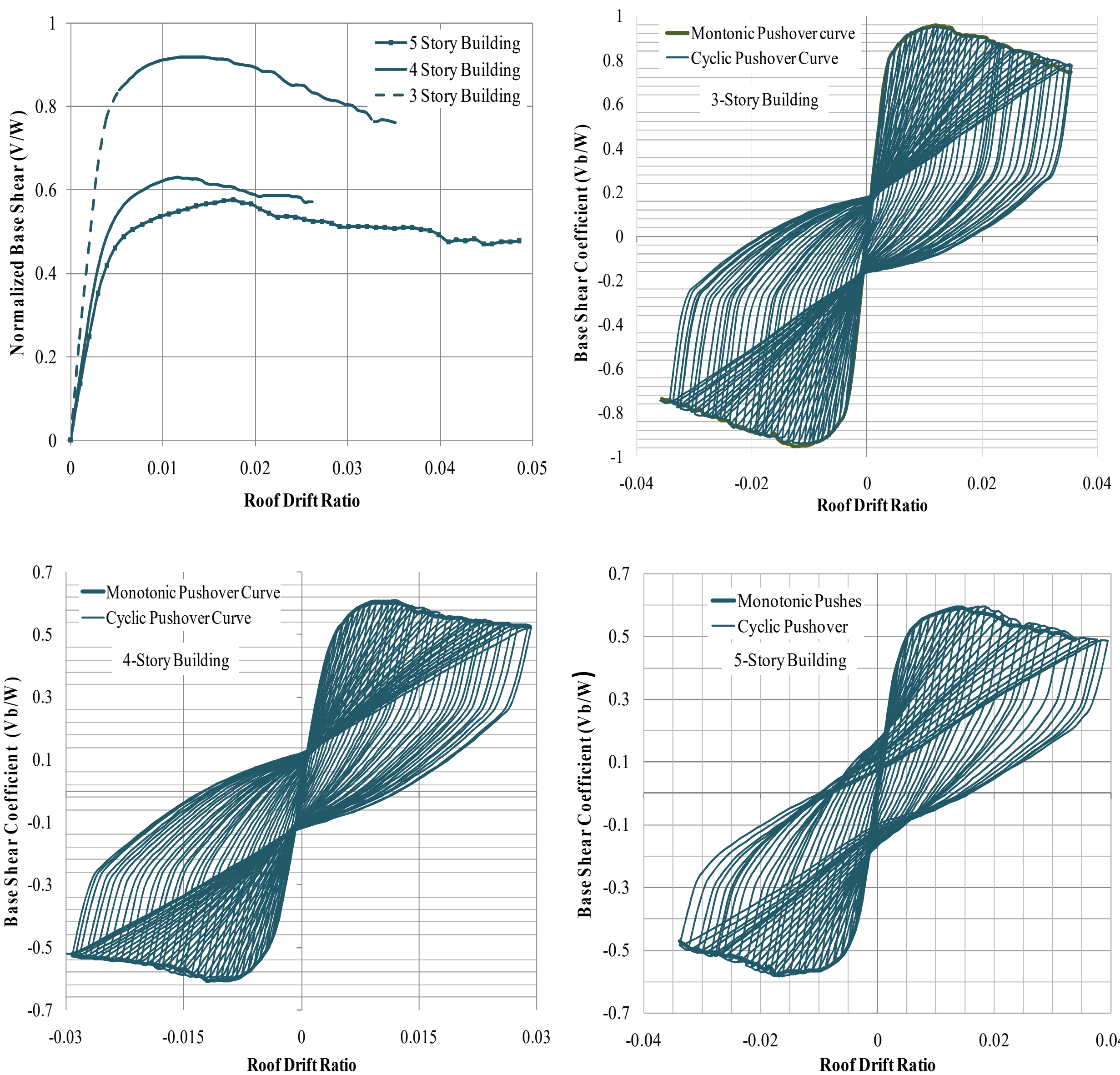


Figure 4-3: The monotonic and cyclic pushover curves of the case study buildings

The equal energy principle is applied to the cyclic pushover curves of buildings to determine the hysteretic damping ξ_{hys}. The actual area $E_D(\Delta)$ of each cyclic loop is computed at its peak displacement response. For all points of envelop pushover curve, the strain energy $E_{so}(\Delta)$ associated with the equivalent linear system (at secant stiffness) is also determined. The point by point application of equation (3.7) yielded hysteretic damping vs. roof drift ratio relations. The equivalent viscous damping ξ_{eq} is determined by adding the initial elastic damping ξ_{el} to the hysteretic damping ξ_{hys} following the equation (3.6). The relationships of equivalent damping vs. roof drift ratio are displayed in Figure 4-4 for the first mode in X-directions of three case study buildings. Similarly, a representative equation of the average equivalent damping of RC frame buildings is developed to approximately estimate the equivalent damping for low-rise infill frames. The equivalent damping is given as:

$$\xi_{eq} = \xi_{el} + \frac{0.2(RD_{Total} - RD')}{(RD_{Total} - RD') + 0.01} \tag{4.3}$$

Where, ξ_{eq} represents the total equivalent viscous damping and ξ_{el} is the initial elastic damping of the building. Similarly, RD_{Total} is the total roof drift ratio and RD' is the roof drift ratio after which nonlinearity starts developing in the building. As a result, it will not be necessary to create a nonlinear model to employ the SDBD procedure to determine the accurate estimations of the seismic demands for masonry infill frame buildings.

To establish the equivalent linear properties of any frame building into its computer structural model, the equivalent natural period or period shift ratio of the building determined using the developed relationships is required to be the same as the fundamental natural period of the building. This is achieved by reducing the stiffness of the column elements and masonry infill panels simultaneously in this study. But it is up to the reader how he ensures that the natural period of the computer structural model is the same as determined from the generalized relationships for infill RC frames. When both the periods are compatible then the seismic forces applied to the computer model of the elongated period will yield the responses in comparison to the detailed NLTHA procedure for local & global responses.

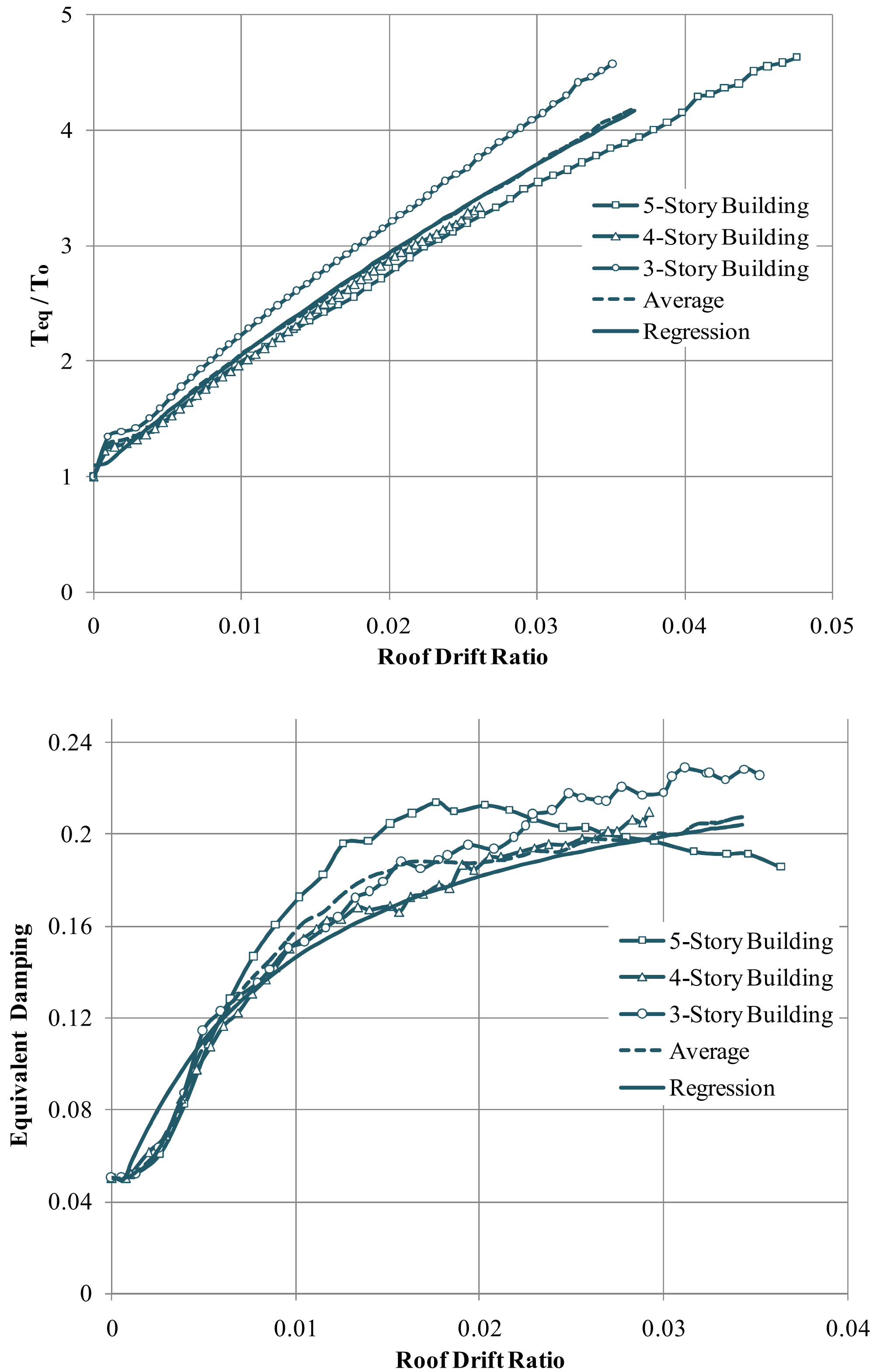


Figure 4-4: Equivalent linear characteristics and generalized relationships of case study buildings

The linear elastic modeling of the building is done using the ETABS software. To get the results of the SDBD procedure, the elongated period and the additional damping is introduced in the structural model of the case study building. The masonry infill walls are modeled using the links to get only realistic estimates of the stiffness in linear elastic modeling. At higher drift levels, the equivalent diagonal struts of the concrete can be removed. Figure 4-5 shows the recommendation to model the masonry infill walls as equivalent diagonal struts.

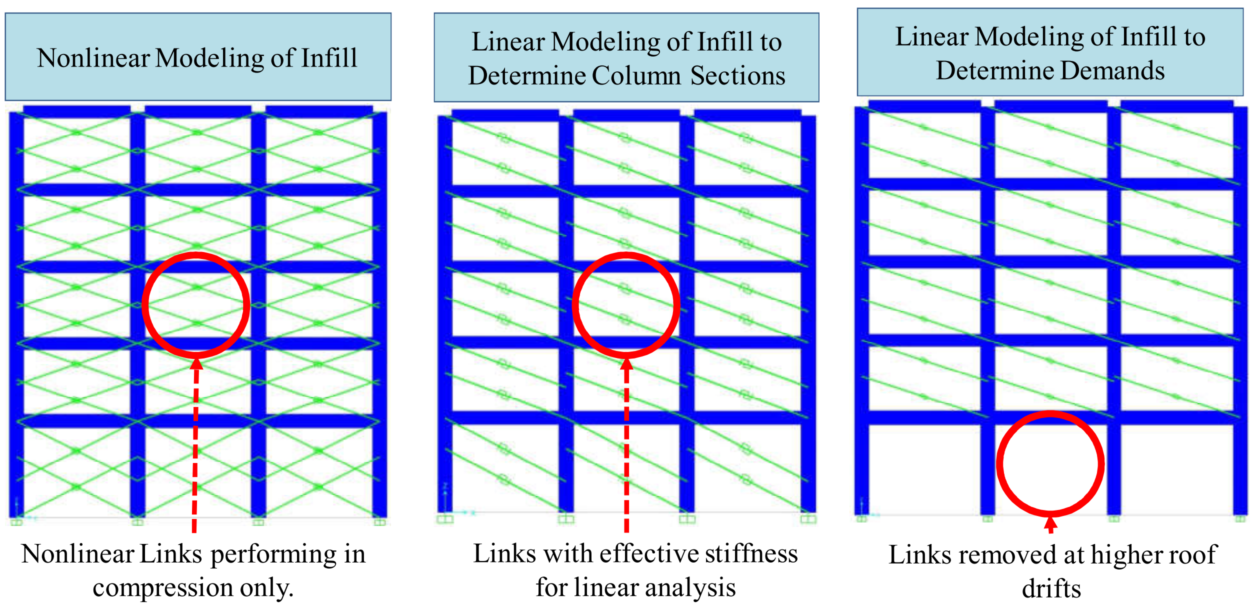


Figure 4-5: Modeling recommendations for the masonry infill walls in different modeling techniques

4.3 Evaluation of the SDBD Procedure using Nonlinear Time History

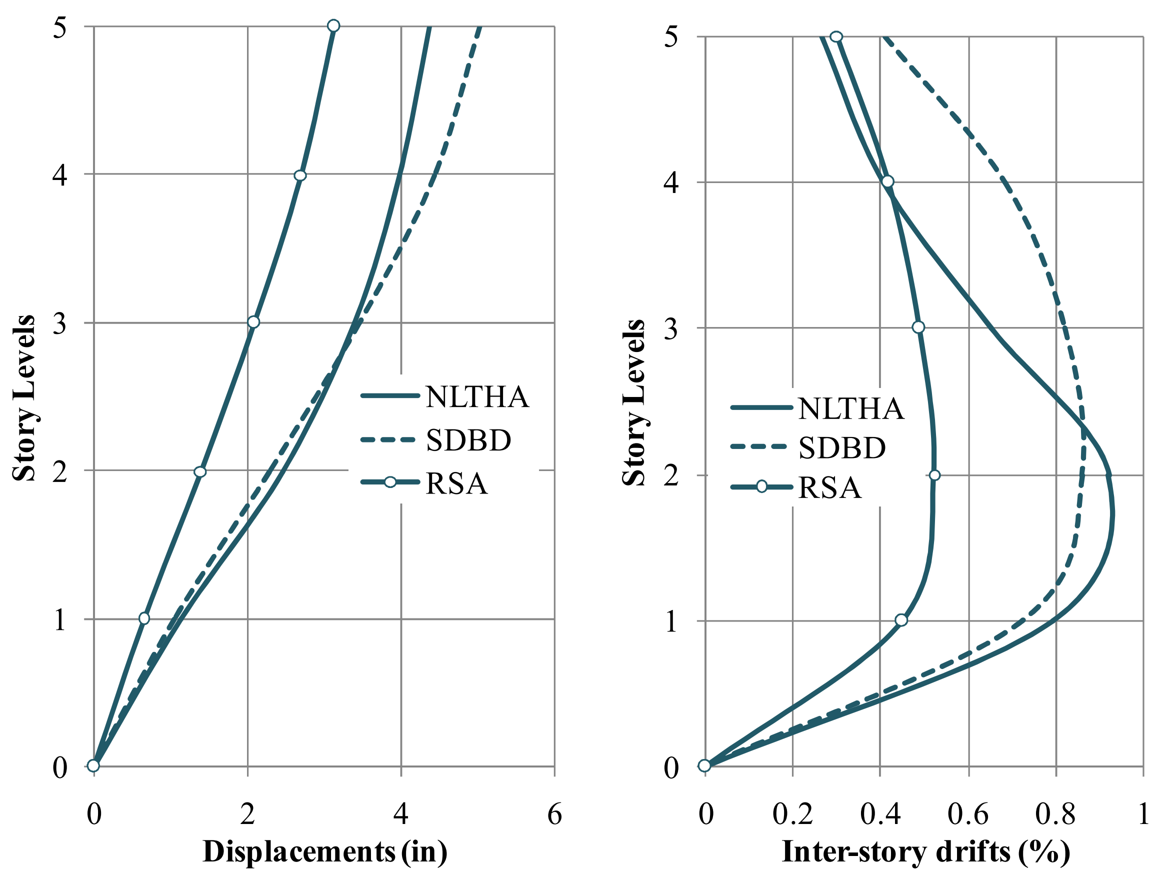
The SDBD procedure is used to estimate the seismic demands imposed by the selected ground motions on a 5-story high building (B4). In this method, a building structural model with an elongated period T_{eq} is subjected to the displacement response spectrum, developed for the additional damping ratio ξ_{eq}, to obtain the seismic responses of the building. The most important step in this proposed SDBD procedure is to accurately determine the equivalent linear properties of the building under consideration. Once the relationships of the equivalent period T_{sec}/T_o versus the roof drift RD and the equivalent damping ξ_{eq} versus the roof drift RD have been established, the equivalent linear characteristics of structures can be found using the iterative scheme.

The initial natural period T_o and the modal participation factor (1st mode) of the building can be obtained from the linear elastic model. This initial period T_o can be used in the displacement response spectra, developed for the initial elastic damping ratio ξ_i, to determine the initial elastic roof drift ratio RD. This initial roof drift RD is used to determine the trial values of the equivalent period T_{sec}/T_o and equivalent damping ξ_{eq} from the developed relationships. The displacement response spectrum is then adjusted for trial equivalent damping ξ_{eq} and the roof drift ratio RD is updated corresponding to the trial equivalent period T_{sec}/T_o. This process is repeated until the roof drift ratio RD value converges for a particular iteration, yielding the final equivalent linear characteristics of that building. This convergence is obtained in a maximum of three trials for the case study building. In this suggested iterative scheme of determining the equivalent properties, the initial roof drift RD can be very small in some scenarios resulting in little additional damping and period elongation. In such cases, the seismic demands can be determined using the initial characteristic of the building.

4.3.1 Comparison of Global Responses

In the next stage, the seismic demands of the building corresponding to the equivalent linear characteristics are estimated. The final roof drift ratio RD can be converted into the roof displacement by multiplying it with the total height of the building and the secant stiffness K_{sec} can be achieved following the equation (2.4). The base shear force V_{base} of that building is then obtained from the equation (2.5). This base shear force V_{base} is distributed along the height of the building using the ELF vertical force distribution approach based on the 1st mode distribution. Then the global responses (story shears and story moments) are computed by following the simple equilibrium rules after the application of these story forces on the relative floor levels. A structural model of the building under consideration is needed to be developed of the computed equivalent linear period, which can be achieved by reducing the stiffness of frame elements and masonry infill panels in a similar manner. The computed story forces are applied to that model of the elongated period at the respective floor levels. The elastic analysis in any available commercial software like ETABS will give you the other global responses (IDR profile and displacement profile) and simultaneously the local responses of structural elements of the building for the proposed SDBD procedure.

Figure 4-6 depicts the comparison of the global responses computed using the standard RSA method, the SDBD procedure, and the detailed NTLHA procedure for the case study building (B4). The results obtained from the NTLHA procedure are considered as a benchmark here. The seismic responses of low-rise buildings are generally governed by the 1st mode of vibration, therefore, only the 1st mode of the case study building (B4) is considered here. In the standard RSA method, the response modification factor R with a value of 3.5 corresponding to the ordinary moment resisting concrete frame as per $UBC-97$ is applied to reduce the elastic demands of the building due to the nonlinearity effects. The inter-story drift ratios and displacements of the building are multiplied by $0.7R$ to take care of the inelastic behavior of the building.



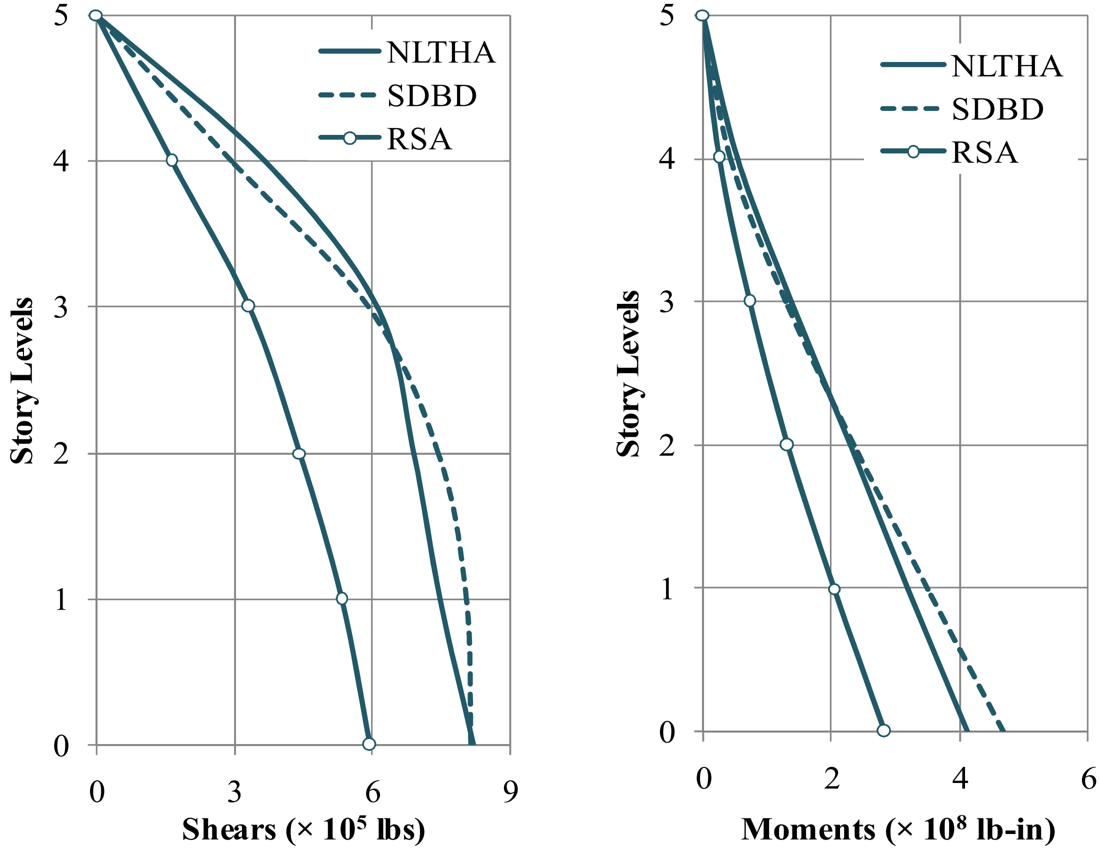


Figure 4-6: The comparison of global responses of the RSA, the SDBD, and the detailed NLTHA procedures

Figure 4-6 indicates that the story shears and the story moments determined from the RSA method are considerably lower than the results of the NLTHA procedure for the overall height of the building. The inter-story drift ratio and the story displacement profiles are also undervalued as compared to the benchmark results of the NLTHA procedure. The reason for the underestimation may lie in the selection process of the response modification factor *R* value. This factor *R* is the same for both systems, one system with low over-strength and high ductility while the other system with high over-strength and low ductility. However, several studies [54] have revealed that the uncertainty in determining the seismic responses is primarily contributed by the ground motion histories because of their random nature. Therefore, a system with high ductility capacity is preferred over a system with high strength. This implies that the high strength system should have a lower reduction factor to be assigned than the high ductile system. More studies and detailed research is needed in this regard because systems have become progressively more complex in terms of their design, arrangements, and configurations.

On the other hand, the SDBD procedure is providing fairly accurate estimations of global seismic responses of the building in comparison to the true demands predicted by the NLTHA procedure. The spectral displacement S_d corresponding to the elongated natural period T_{sec}/T_o is greater than S_d at the initial period T_o, for the target response spectrum under consideration. The global responses corresponding to this equivalent natural period T_{sec}/T_o are reasonably accurate as compared to the responses of the standard RSA method. The respectable performance of the SDBD procedure demonstrates that it can be considered and developed further for the determination of the seismic demands of masonry infill RC frames. The SDBD procedure offers computational effort and convenience approximately similar to the linear elastic analysis with an additional step of determining the appropriate equivalent linear characteristics of the building.

4.3.2 Comparison of Local Responses

The member-level forces and the deformations also play an important role in the design and performance assessment of structural members besides the global responses of the building. The SDBD procedure can predict the local responses of frame members in any generalized structural analysis software due to its convenient application, apart from the global responses. Figure 4-7(a) displays the shear forces of two columns of the 5-story high building (B4), determined using the SDBD procedure, the standard RSA method, and the detailed NLTHA procedure. One exterior column and the other interior column, continued along the total height of the building, are adopted in this evaluation. It is apparent that the shear force demands determined from the SDBD procedure are in good comparison with the demands determined using the NLTHA procedure, while the standard RSA method displays the enormous undervaluation of results. A similar utterance may be derived from the bending moments of both columns, displayed in Figure 4-7(b). It is evident that the standard RSA method considerably underestimates the seismic demands, while the SDBD procedure captures the nonlinear demands closer to the actual demands determined by the NLTHA procedure.

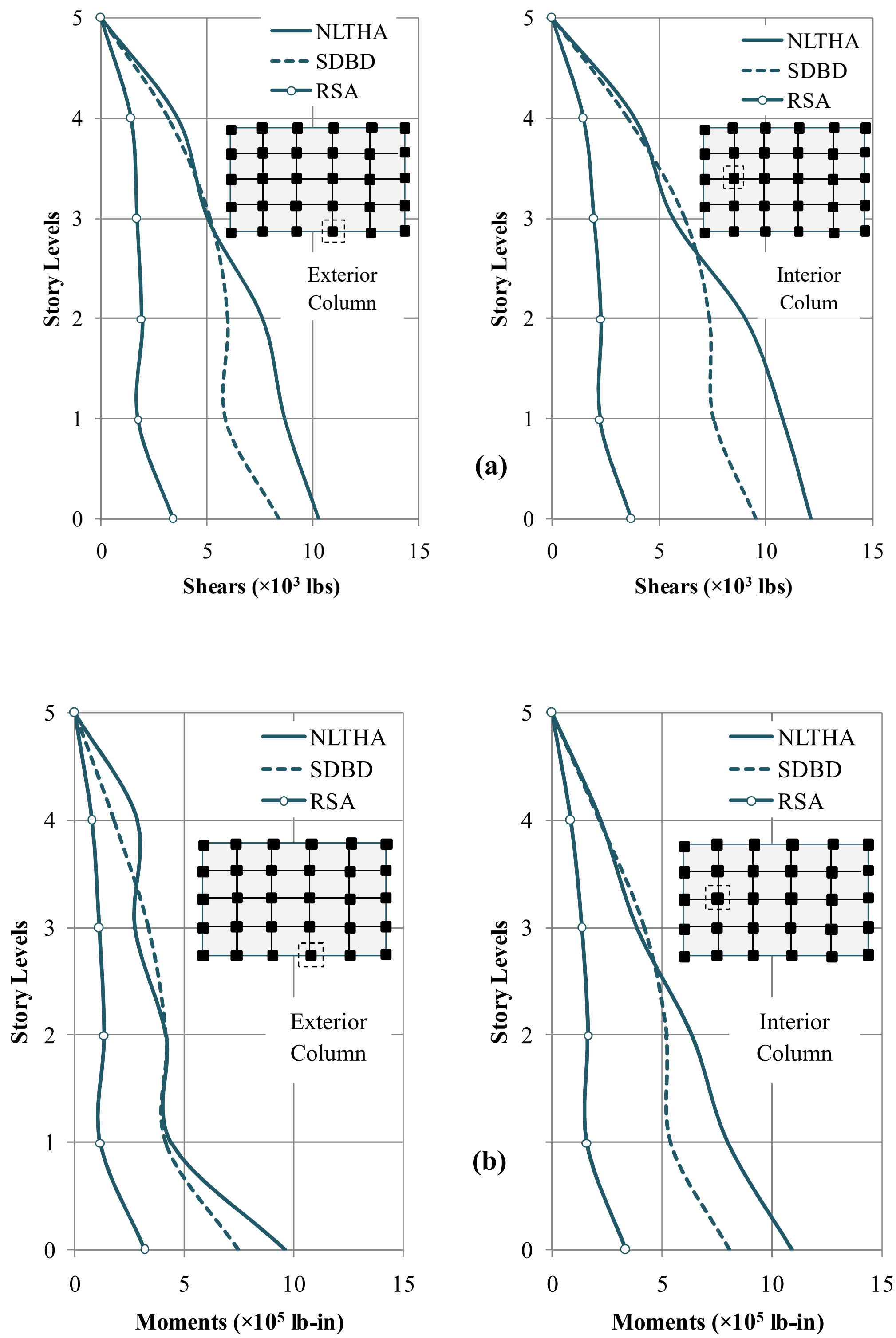


Figure 4-7: The comparison of local responses among RSA, SDBD, and NLTHA procedures. (a) The shear forces of exterior and interior column along the total height of building. (b) The bending moments of exterior and interior column along the total height of building.

The global seismic demands estimated by the SDBD procedure are satisfactory but the member level demands are showing notable differences compared to the true nonlinear demands of the NLTHA procedure. This difference may be due to the non-identical distribution of global seismic demands among the structural members. The comprehensive NLTHA procedure takes into account the exact distribution of seismic demands in structural members, while the SDBD procedure assumes the elastic distribution of demands. Nevertheless, the member-level demands estimated by the SDBD procedure are considerably more appropriate than the demands determined by the standard RSA method. Therefore, the SDBD procedure can be used to design new structures and evaluate the seismic performance of existing structures by providing the local and global seismic demands with a reasonable level of accuracy.

To implement the SDBD procedure in practical situations, there’s a necessity to develop the generalized relationships between the equivalent linear characteristics of the system and appropriate deformation measure (e.g. roof drift ratio) for an extensive variety of structural systems and hysteretic behaviors. Several graphical aids (e.g. family of displacement response spectra developed for different damping ratios) can be provided to help the practicing designers in finalizing the equivalent characteristics quickly through iterations to adopt the proposed SDBD procedure for buildings.

Chapter 5

Conclusions & Recommendations

5.1 Conclusions

This study proposed a simplified displacement-based design (SDBD) procedure, using the framework of the DDBD method that is based on the EL approach. The fundamental philosophy is that a suitably tuned linear elastic SDOF system having additional damping and elongated period can almost mimic the response of a nonlinear system; therefore, an equivalent linear system can be appropriately used to determine the true seismic demands of a nonlinear system. The best-known EL approach (determination of additional damping using equal energy principle at secant stiffness related to maximum displacement response) is employed to propose and evaluate the SDBD procedure for masonry infill RC frames. The results of the 5-story high case study building predicted by the SDBD procedure showed that this procedure can estimate the seismic demands of infill frames effectively in comparison to the true nonlinear demands of the NLTHA procedure. The results obtained from the SDBD procedure are in good comparison to the actual results than the results of the standard RSA method as the latter method underestimates the seismic responses significantly. The SDBD procedure neither requires nonlinear modeling nor nonlinear analysis to obtain the demands and hence it keeps the simplicity offered by the linear elastic analysis. This study is merely an initiative in the direction of advancement in a more resourceful SDBD procedure. The implementation of other recognized EL approaches (i.e. instead of adopting Jacobson's damping at secant stiffness) and proper distribution of global forces in structural members can enhance the accuracy of the SDBD procedure to determine the seismic responses of various structural systems. Considering the implementation of this scheme in general design offices, further improvement is needed in this method to make it suitable for building structures of different materials, designs, and configurations.

5.2 Recommendations

The generalized relationships for determining the equivalent linear characteristics can be developed using the various structural systems & different hysteretic behaviors of buildings. Different distribution of the masonry infill walls (i.e. soft story) can be considered for future studies. The other modeling approaches of masonry infill panels can be used to obtain the results rather than using the equivalent concrete diagonal strut model approach only. The brittle failure of structural elements due to the presence of infill panels may be considered to propose a more resourceful procedure. The out-of-plane contribution provided by the masonry infill panels may be considered for further investigation.

The proposed equations are developed using only the X-direction of case study buildings while other direction of buildings can be considered for further suitability. A family of generalized curves for representative building's stock of Pakistan can be developed and handed over to the designers to implement this proposed scheme. The other EL approaches instead of additional damping at secant stiffness can be considered to improve the accuracy of the SDBD procedure. The accuracy to determine the seismic responses of the building can be increased by incorporating the soil-structure interaction effects in this scheme. The proposed procedure is based on the 1^{st} mode of the building and it can be improved by taking into account the higher mode effects in its formulation.

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