Direct Displacement Based Seismic Design Compared with Force Based Design Approach

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ABSTRACT

An important advancement in earthquake engineering has been the development of performance-based concepts for the seismic design of structures. Performance-based seismic design approaches overcome the shortcoming of the traditional force-based approach. The first step in the Direct Displacement-based Seismic Design (DDBSD) procedure is the definition of the target displacement that the building should not exceed under a given seismic hazard level. Indeed, the procedures require inelastic analysis.

Direct Displacement-Based Seismic Design procedures appeared in recent seismic design codes (e.g., EC 8, etc.) in addition to the traditional forced-based approach. The design of several buildings via EC 8 Displacement Based approaches and traditional forced-based approaches will be discussed in this thesis.

Low-Rise, mid-rise, and high-rise RC has been designed based on both approaches. Seismic Performance levels of the designed buildings have been re-evaluated via the nonlinear dynamic analysis procedure. The effect of design strategy on the performance and cost of each type of building have been compared. Building performance has slightly increased in terms of displacement. Furthermore, a considerable amount of improvements has been observed in plastic hinge formations. On the other hand, the cost difference between the two methods was acceptable.

Keywords: Direct Displacement Based Seismic Design, Force Based Design, Nonlinear Time History Analyses, Pushover analysis, plastic hinges

ÖZ

Deprem mühendisliğindeki önemli bir gelişme, yapıların sismik tasarımı için performansa dayalı konseptlerin geliştirilmesi olmuştur. Performansa dayalı sismik tasarım yaklaşımları, geleneksel kuvvet temelli yaklaşımın eksikliklerinin üstesinden gelir. Doğrudan Deplasmana Dayalı Sismik Tasarım (DDBSD) prosedürünün ilk adımı, binanın belirli bir sismik tehlike seviyesi altında aşmaması gereken hedef yer değiştirmenin tanımlanmasıdır. Gerçekte, prosedürler plastik ozellikleri goz onunde bulunduran analiz gerektirir.

Doğrudan Deplasmana Dayalı Sismik Tasarım prosedürleri, geleneksel zorunlu temelli yaklaşıma ek olarak son sismik tasarım kodlarında (örneğin EC 8 vb.) ortaya çıktı. EC 8 Deplasman Temelli yaklaşımlar ve geleneksel kuvvet temelli yaklaşımlar aracılığıyla yasarlanan binanin performansi bu tezde tartışılacaktır.

Az, orta ve yüksek katlı RC, her iki yaklaşıma göre tasarlanmıştır. Tasarlanan binaların sismik performans seviyesi doğrusal olmayan dinamik analiz prosedürü ile yeniden değerlendirildi. Tasarım stratejisinin her bina türünün performansı ve maliyeti üzerindeki etkisi karşılaştırılmıştır. Tavan deplasmani açısından bina performansı biraz artmıştır. Ayrıca, plastik menteşe oluşumlarında önemli miktarda gelişme gözlenmiştir. Öte yandan, 2 yöntem arasındaki maliyet fark dikkate alinmayacak kadar ufaktir.

Anahtar Kelimeler: Doğrudan Yer Değiştirme Tabanlı Sismik Tasarım, Kuvvet Tabanlı Tasarım, Doğrusal Olmayan Zaman Geçmişi Analizleri, İtme analizi, plastik menteşeler

DEDICATION

I dedicate this study to my amazing family and husband. I have endless gratitude to my parents for their endless love, Foorozan Najafi and Amir Mostaani, who have supported me every step of the way leading to my success in life. My little brother has always supported and cheered me for my success. And my husband becoming a beacon of hope with his undying love and endless support.

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INTRODUCTION

1.1 Overview

1.1.1 Earthquakes and Seismic Risk

Earthquakes are one of the most dangerous natural disasters that can have a serious impact on the world and how it is put together. Earthquake incorporates the displacement of the crust combined with the rapid relief of energy which triggers seismic waves in the Earth's crust. The release is generated by elastic strain, gravitational force, biochemical processes, enormous bodies' movement. The discharge of elastic strain is by far the most important element, as this source of energy can be stored in great quantity in the crust to create significant forces. Earthquakes that are associated with such a source are called tectonic earthquakes. (Taranath 2005; Turer, Yakut, and Akyuz n.d.).

The elastic rebound hypothesis gives the best explanation for Tectonic earthquakes. The hypothesis was developed by Harry Fielding Reid following a seismic event at 1906 USA near Los angles, resulting in a massive earthquake. An earthquake of tectonic origin occurs theoretically if the stresses within the rock bodies build immensely that resultant strain exceeds the rock's strength, consequent in the rock's abrupt disintegration. (Minimum Design Loads for Buildings and Other Structures 2013; Taranath 2005). When these earthquakes hit structures, it creates inertia forces that may be extremely destructive, causing deformation. Hence a good awareness of the seismic effect on the structure and the impact of seismic forces applying on the structures must be understood by construction industry professionals, allowing correct protective precautions for possible failures and collapses scenarios to be taken.

As seen in Figures 1 and 2, in extensive parts of the globe, including TRNC and Turkey, high seismic risk demand can be observed, which indicates more testing and analysis in the field of seismic resistance construction should be done. Sadly, in 1999, Turkey encountered two major catastrophic earthquakes: the Izmit-Turkey event on 17/8/1999, with 7.4 magnitude, and the Duzce incident on 12/11/1999 with 7.1 magnitude; catastrophic earthquakes that country has endured in the recent past. These events occurred along with parts of the infamous North Anatolian Fault Zone (NAFZ), Turkey's most widespread active fault line, which runs through the northwest of the country for more than 1500 kilometers, covering Istanbul and other heavily populated cities, delivering 25 millimeters per year drift at Anatolia-Eurasian plate boundary located at the juncture of the Europe and Africa’s corresponding tectonic plates. Cyprus has been hit with several devastating earthquakes at it’s past. The majority of the substantial earthquakes happened in the southern part of the island, causing damage to major cities such as; Paphos, Limassol, and Famagusta.

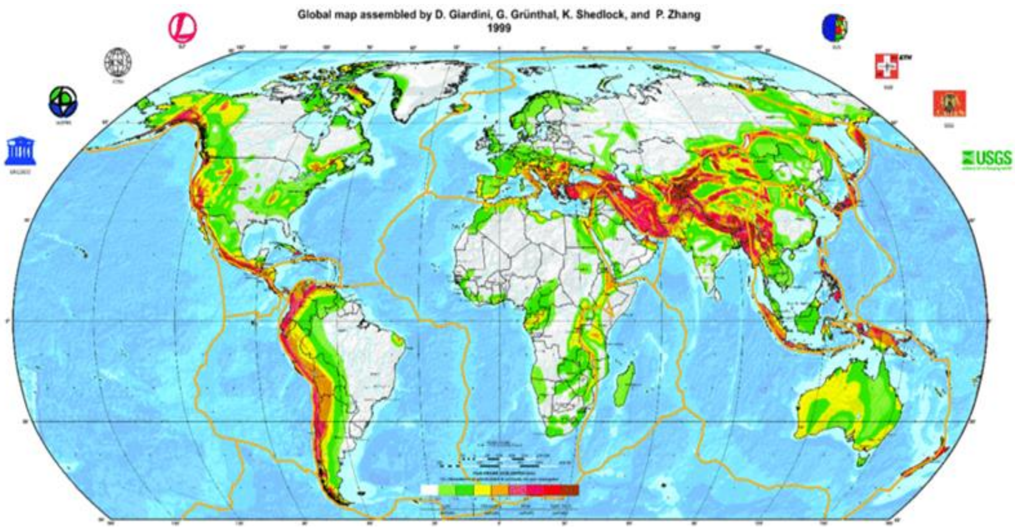

Figure 1: Global seismic hazard map (Montalvo and Reynal-Querol 2019)

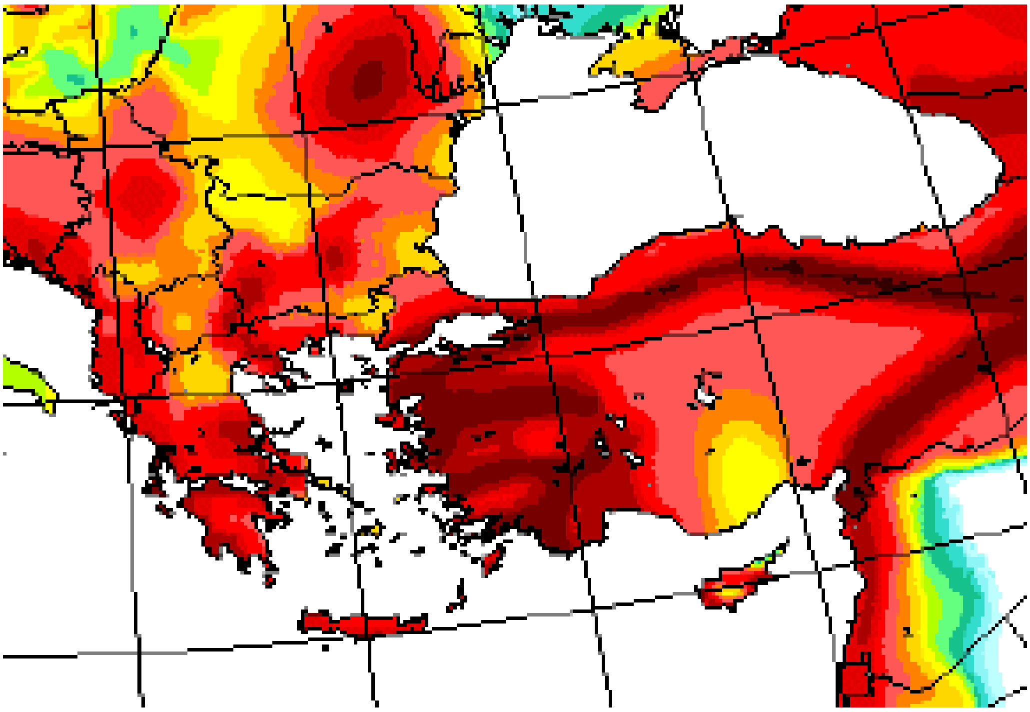
Figure 2: Seismic risk of the Mediterranean basin (D’Amico and Currà 2014)

1.2 Review of Design Methods

Over the last decade, the innovations of performance-based principles for the seismic design of structures have been an important development in seismic structural design. This method focused on the combination of several performance limit states and levels of seismic hazard, overcomes some of the shortcomings of the conventional forced-based design procedure based on strength. The initial step in the Direct Displacement-based Seismic Design (DDBSD) technique is to establish the target displacement that the building's seismic hazard level does not surpass. At present, without considering the higher mode impact, the structural target displacement curve is mostly defined by its first mode displacement (LUO, Mechanics, and 2003 n.d.; Medhekar and Kennedy 2000), so it lacks to accurately assess the efficiency of the high-rise building.

In 1990, the response spectrum idea was implemented, where responses plotted over an extensive range of a single degree of freedom. Following the calculation of the structural periods, calculated spectrum graphs are used to predict the structure's expected response under the effects of the seismic events. The base shear force is an approximation of the maximum total lateral force predicted that could occur at the base of a structure due to seismic ground motion. Base shear (V) measurements rely on several variables, such as the soil conditions at the construction site, the epicenter radius, the likelihood of seismic events, and the structure's natural vibration duration.

Displacement-based designs are now wildly accepted to be the better method for seismic analysis and design. According to most construction codes, it is still completely acceptable to use the older force-based design for seismic design when designing a new structure. In this case, a question arises, if displacement-based methods are clearly superior, then why do these national construction codes still accept the outdated force-based design like Eurocode 8 (Lubkowski and Duan 2001), Turkiye Bina Deprem Yonetmeligi 2018 (Turkish Building Earthquake Regulation 2018), American Balding and Construction code, It is important to understand the structural performance of the structures, change of structural element behaviors, and change in the overall cost of the construction with different seismic designs.

Designing a structural system can be as simple as calculating static loads and assigning a correct cross-sectional area to the structural members of the structure. But this is usually not the case due to potential hazard risks like earthquakes and extreme wind loads that might threaten the structure. In the case of earthquake areas, the construction code of the region/country/state enforces seismic checks and appropriate seismic design to be applied to the structures built in these regions. Traditionally seismic design was made using a method called Forced Based design, which provided lateral static loads to be applied to the floors of the structures. But as innovation and technology advances, more advanced methods have emerged. Unlike the forced-based method, displacement-based seismic designs are better at estimating the performance of the building as they can consider the plasticity of the elements.

To be able to understand the effects of different seismic designs on the parameters discussed previously, performance assessment of buildings with different heights will be performed against different earthquake recordings. It is important that different height structures will be used for assessment to eliminate and design methods favoring certain structural periods or any unpredictable parameters. Furthermore, performance assessment of the structures will be performed by time-history analysis with different earthquake records to eliminate any single extreme result misleading the assessment.

The structure that will be used as a testing structure is chosen to be a residential building; therefore, height scalability can be performed without changing the floor plan. Furthermore, a mostly symmetrical floor plan is used for the design, eliminating any unwanted effects caused by the shape of the structure. Although symmetry of the structure is imported, it should be noted that the building is similar between x and y axes but not exactly the same, introducing a difference in stiffness and number of structural elements in 2 different axes. This ensures that the structure has different fundamental mode periods and responses in different axes.

1.3 Organization of the Thesis

After an introduction to the topic of this study and motivation to review these methods in Chapter 1, each method is described further in the next two Chapters. In Chapter 2, the FBD method is introduced, and in Chapter 3, the DDBD method is introduced. There are various equations and approaches, such as material strengths and strains recommended for use in the calculations, that are not the same in each method, but for comparability reasons, it is important that all assumptions used in the calculations are the same. In addition, to explain the differences between these methods, capture the advantages and disadvantages of each method, a relatively simple structural model is chosen.

Chapter 2

LITERATURE REVIEW

2.1 Seismic Design Procedures

Seismic design procedures can be classified into two types: force-based or performance-based seismic design.

2.1.1 Force-Based Seismic Design

The force-based design, FBD, is based on the calculation of the base shear force occurring from the seismic activity dynamic motion, using the acceleration response spectrum and the anticipated elastic frequency of the structure. In this method, static loads are applied to a building with magnitudes and directions that strongly resemble the consequences of the dynamic loading of seismic events. Concentrated lateral forces caused by dynamic loading are applied on each floor of buildings where mass concentration is located. In addition, localized lateral forces tend to follow the fundamental mode of the structure; in other words, they are stronger at higher stories in the structure. As a result, the maximum lateral displacements and the maximum lateral forces always arise at the roof level. These effects are modeled in the equivalent static lateral force method in most design codes by imposing force at each story of the structure, where the applied force is directly proportional to the height. (Elattar, Zaghw, and Elansary n.d.)

2.1.1.1 Applicable cases

This type of analysis can be applied to the buildings where the response of the structure to the seismic activity is inconsequentially caused by the higher mode vibrations compared to fundamental mode effects. It is also not advised/allowed to use this method if the fundamental period of the structure is greater than 2 seconds or greater than 4 T_c (upper limit of the period of the constant spectral acceleration branch). Furthermore, regularity in the elevation should be verified using the following list;

- All load-bearing structural elements (like frames, structural walls, etc.) should be continuous from the foundation to the top of the building or to the relevant end of the load-bearing systems.
- Lateral stiffness and the mass of the continuous floors should remain constant or gradually decrease without abrupt differences in the foundation to roof direction.
- For framed structures, the proportion of the actual resistance capacity to the resistance demand obtained from analysis must stay within reasonably close within neighboring floors.
- Further information about regularity and setbacks can be found in the Eurocode 8 (Lubkowski and Duan 2001)chapter 4.2.3.3.

Several countries around the globe use the force-based design procedure to assess the seismic design. The force-based method steps are defined in Figure 3.

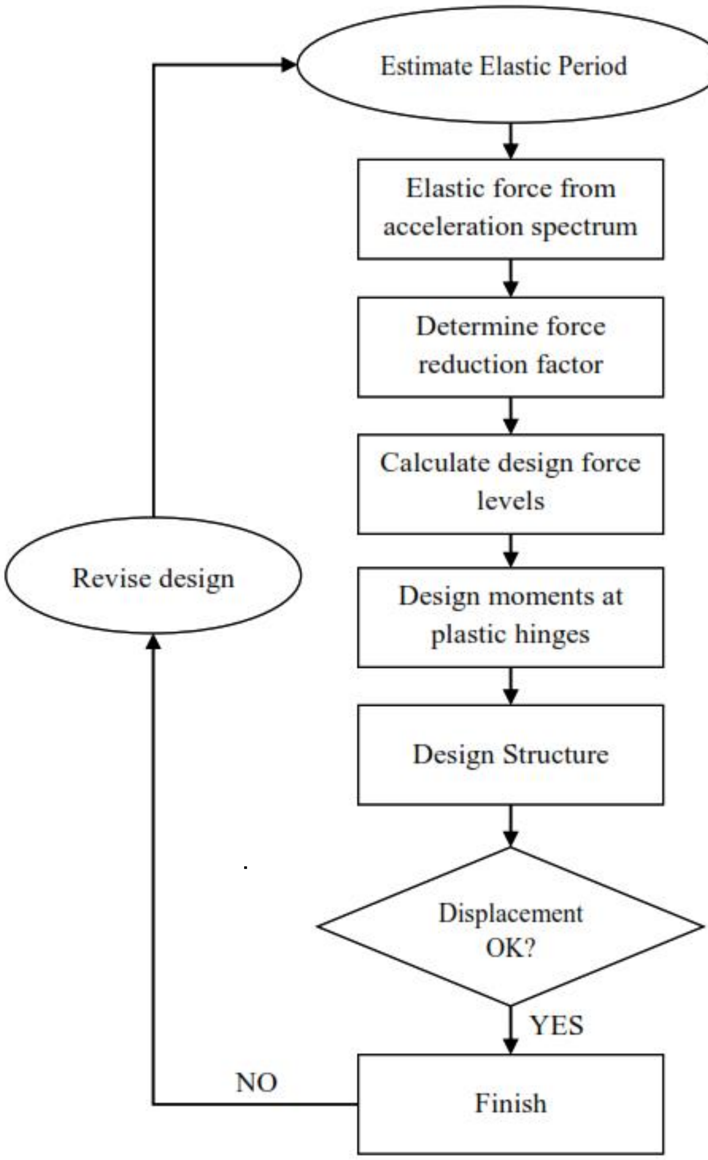

Figure 3: Force-Based Design methodology progression

The approach illustrated in Figure 3 is the most commonly used method for assessing the design base shear worldwide. A strength reduction factor (R) is determined by the structural system's assumed ductility, and a significant factor (I) is used to increase the design strength of structures of higher significance. Lateral design forces at ground level are then calculated using the specified equations to reflect the structure's dynamic properties. In order to assess a member's strength, an elastic analysis is conducted. Following the member strength’s determination, ion amplification factor for deflection, Cd, is utilized that amplify the estimated drift from the elastic analysis.

Obtained drift is checked from code-defined limitations. The technique is repeated until both the strength and drift criteria are fulfilled. In order to satisfy the required ductility conditions, member detailing provisions are practiced.

In brief, the key drawbacks of all of these existing methods currently used in codes are;

1. Taking safety into account may be accomplished by increasing the design base shear. Numerous past earthquakes have documented structural collapse as a result of local column damage.
2. Considering the design of the lateral forces acting along the height of the building that depends on elastic behavior: non-linear dynamic analysis revealed that utilizing the specified code distribution of the lateral forces, without taking into consideration that the system will reach inelasticity during a large earthquake, may be the primary cause of plentiful upper-story failures.
3. Assigning component cross-section sizes based on beginning stiffness (from elastic analysis): the severity of each element's forces is defined by the structural elements' relative elastic stiffness. Although in bigger earthquakes, the stiffness of certain parts might change drastically as a result of concrete cracking or steel yielding, the stiffness of others members can remain constant. This alters the force distribution in the structural components. Correct element size assignment is impossible without employing a more representative distribution of forces that incorporates expected inelastic behavior.
4. Trying to model inelastic displacements by using estimated variables and analytical behavior: It has been proven by several previous researchers to be

impractical, particularly for structures neither with deteriorating hysteretic behavior nor with energy dissipation mechanisms (Liao 2010)

2.1.1.2 Distribution of the Horizontal Seismic Forces

The calculated F_b is the total base shear, and in forced-based analysis, the loads should be distributed to each floor. To be able to distribute these forces, story displacements and masses should be known in fundamental mode shapes. Structural dynamic analyses are done to find these masses and displacements. Once these values are known, the horizontal force acting in each story, F_i, can be calculated using the following formula;

$$F_i = F_b \frac{s_i . m_i}{\sum s_j . m_j}$$

Where;

F_i is the horizontal force in story i;

F_b is the calculated seismic base shear;

$\boldsymbol{s_i}$, $\boldsymbol{s_j}$ are the mass displacements in the fundamental mode shapes;

m_i , m_j are the story masses obtained from dynamic analysis.

When the fundamental mode shape is modeled using a linearly increasing horizontal displacement along the height, individual horizontals of each story is calculated using the following formula, where the z_i , z_j are the height of the concentrated masses above the application of seismic load (above foundation or top of rigid foundation);

$$F_i = F_b \frac{z_i . m_i}{z_i . m_i}$$

Obtained horizontal forces F_i from this process are distributed as lateral loads to corresponding individual stories (verification of story rigidity to their planes must be done to be able to assign these loads).

2.1.1.3 Torsional Effects

Unless a more detailed methodology is used, which considers torsional effects and moments created, amplification factors might be used to increase the accuracy of the simpler methods like the force-based design. In the case where masses and lateral stiffness's of the stories are symmetrically used within the plane, without the effect of torsion caused by the accidental eccentricity of applied load, the accidental torsional effect may be counteracted by multiplying action effect (the resulting effect of applied individual load resisting elements) by the factor δ, which can be obtained by;

$$\delta = 1 + 0.6\frac{x}{L_e}$$

Where;

x is the distance of the element in which the application factor will be used to the center of mass of the structure, measured perpendicular to the seismic action direction.

L_e is the distance between two structural elements, which are most outermost, measured perpendicular to the seismic action direction.

In the case where the analysis is performed using two planar models, one of each horizontal direction, torsion effects acting on the structure can be estimated by doubling accidental eccentricity, e_a, and changing the 0.6 factor used for δ to 1.2.

2.1.2 Performance-Based Seismic Design (PBSD)

The method starts with the determination of performance objective, then defines a tentative formulation, evaluates how well the design meets its performance targets, and iteratively reevaluates it for progress before the target performance level is achieved. Figure 4 displays the key steps of the design procedure and its flow.

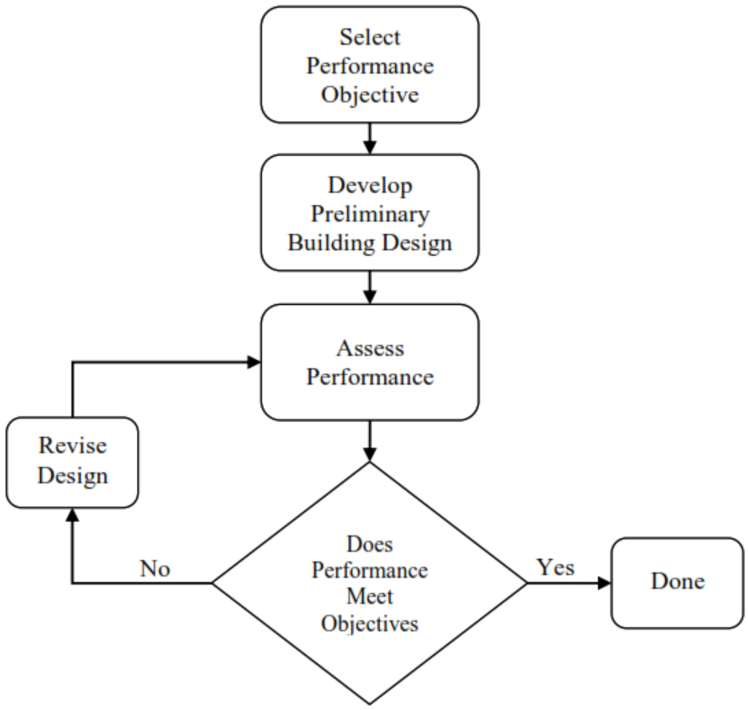

Figure 4: Performance-Based Design methodology progression

The performance-based design process begins by defining the design parameters as performance objectives. Each performance target gives an understanding of what defines a fair risk for facing varying levels of harm at varying levels of seismic hazard. If performance targets have been established, a modeling sequence (building response analysis through loading) will be performed to assess possible structure performance under various construction situations. In the event of severe packing, as when the large-scale earthquake would create, results can be obtained using non-linear methods of analysis. If the simulation performance surpasses or fulfills the performance targets, the design is acceptable. If not, the model can be modified in an iterative process before the objectives of performance are reached. With certain circumstances, the defined objective may not have been possible to achieve at a reasonable economic level, wherein these situations certain relaxing of the set performance targets may be acceptable (Fema- 2006)

2.2 Seismic Evaluation Methodologies

The structural failure of already existing structures and loss of human life during a wide range of earthquakes globally have illustrated the need for seismic assessments of existing buildings, particularly those which are not built to withstand earthquakes. Upon this basis, government bodies in different countries have established codes and new guidance for seismic assessment and retrofitting of already built structures.

(Calvi et al. 2006) The research classified the seismic assessment approaches that are relevant to categories: empirical (qualitative) approach and analytical (quantitative) approach. Qualitative seismic assessment techniques focus on determining a building's previous damage due to seismic activities to extrapolate its likelihood of potential damage due to structural resemblance. In another context, it aimed to determine damage in a given building with specific topography within a pre-modeled earthquake. For city-wide or region-based damage assessment, calculated damage is then extrapolated. The analytical seismic assessment techniques are focused on a more comprehensive seismic evaluation with an in-depth numerical assessment of the building to expose which buildings are vulnerable to different forms of seismic damage (Calvi et al. 2006).

The applicable earthquake assessment procedures were categorized by another research done (Murty et al. 2003) into two categories: configuration based; Configuration-related tests comprise a fast assessment of the structure's seismic resistance by assessing the configuration-induced deficiencies identified as the cause of poor findings, as well as essential strength checks. Atypical geometry, a vulnerability in a particular story, the influence of lumped mass, or a discontinuity in the lateral load resistance system are all examples of typical building layout flaws. The purpose of the configuration-related controls is to monitor and identify fragile systems for thorough review and assessment. (b) strength-based controls; Strength-related controls consist of thorough force and displacement assessments to determine strength at the structural and/or elemental levels. Available possible seismic assessment procedures are a mixture of configuration-based controls and strength-based controls (Rai 2005).

2.2.1 History of DDBSD

Since its inception in 1993 by M. J. Nigel Priestley(M. J.Nigel Priestley 1997), the "Direct Displacement Based Design" approach has drawn the interest of researchers from Europe as well as from New Zealand and North America. Buildings should be designed to meet a defined level of performance, measured by strain or drift limitations, within a set degree of seismic severity according to the DDBD concept. Because of this, we might call it a structure with a "uniform-risk." (M J N Priestley, Calvi, and Kowalsky 2007)

This re-examination of the DDBSD procedure was first proposed in 1993, then revised in 2003. The outcomes of this re-examination are as follows:

- Initial stiffness portrayal of structures that are anticipated to adapt inelastically to earthquake design levels. (M J N Priestley, Calvi, and Kowalsky 2007)

• Examination of seismic evidence to provide more effective feedback for displacement-based design. (M J N Priestley, Calvi, and Kowalsky 2007)

• Several concepts of inelastic time-history analysis, including elements that are significant to the prediction of elastic damping, are re-examined.(M J N Priestley, Calvi, and Kowalsky 2007)

• Development and implementation of calculations for equal viscous damping to the demand for ductility of various structural structures. (M J N Priestley, Calvi, and Kowalsky 2007)

• Development of alternative techniques for the evaluation of design moments in structural elements through the design of lateral forces. (M J N Priestley, Calvi, and Kowalsky 2007)

Examining the capacity design equations for diverse structural systems and developing new ductility-based equations. (M J N Priestley, Calvi, and Kowalsky 2007)

2.3 Qualitative Assessment Methodologies

A study done by Calvi et al. (Calvi et al. 2006) concluded that the use of macro-seismic intensity scales such as the Modified Mercalli scale (Eiby 2012) , EMS98 scale, and the MSK scale in developing an earthquake analysis methodology had enabled development of qualitative methods for the study of structural failure from earthquakes.

There are three main kinds of empirical methods to earthquake engineering: damage probability matrices (DPM), vulnerability index methodologies, and screening procedures.

Notably, the term "vulnerability" relates to how structures respond to seismic shocks. If two different sets of buildings are impacted by the same shock event and one group responds more strongly than the other, one may infer that the less damaged buildings were less vulnerable to earthquakes, and vice versa.

2.3.1 Analytical Evaluation Methods

There have been developed sophisticated methods of seismological assessment utilizing computer programs. This sort of evaluation is carried out to determine the structural ability of a building after an earthquake.

Analytical techniques must be used in situations where there is no historical record of past seismic damage. This model may be used to calculate a specific building or a particular building type. Analytical methods are being used on these bases to calculate the seismic tolerance of the structures in the analysis. Two key types of analytical techniques can be identified, spectral techniques and displacement techniques.

2.4 Collapse Mechanism

2.4.1 Drift Failure

There are three historically specified failure forms for laterally loaded columns; flexural failure, shear failure, and flexicurity-shear failure, as seen in Figure 5. A flexural column is one that has adequate shear strength to resist until it yields in flexure, while a shear column fails in shear, resulting in a moment demand that is much less than the moment strength. A column that is flexure-shear-critical fails by yielding from flexural stress at a more ductile stage without failing in shear.

Determining Flexural-to-Shear Strength Ratio (FSSR) then referring to the chart may assist in determining the failure cause. As shown in Table 1, the ASCE developed the framework for categorizing ductile columns, and therefore it is not immediately suitable for smaller ductile columns:

$$FSSR = \frac{M_n}{LV_n}$$

M_n =moment capacity

V_n =nominal shear strength

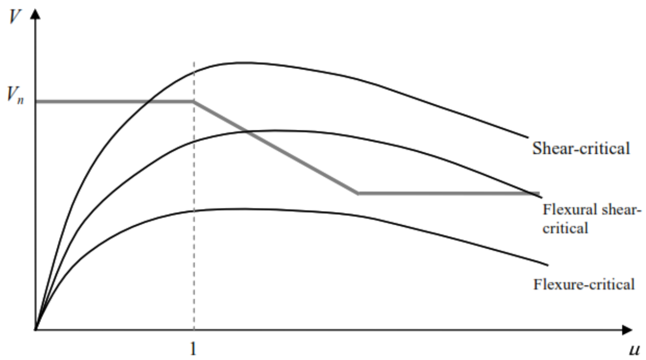

Figure 5: Deterioration of shear strength and displacement ductility

Table 1: Structural failure mechanisms as classified by ASCE/SEI41-06

	Transverse Reinforcement Details		
	ACI conforming details with 135° hooks	Closed hoops with 90° hooks	Other (including lap spliced transverse reinforcement)
$FSSR \leq 0.6$	Flexure	Flexure-shear	Flexure-shear
$0.6 < FSSR \leq 1.0$	Flexure-shear	Flexure-shear	Shear
$FSSR > 1.0$	Shear	Shear	Shear

2.4.1.1 Sezen & Moehle Model (2002)

When it comes to shear stress distribution, Sezen and Moehle created an innovative 45° truss model in 2002 that takes into account both the concrete and concrete reinforcing strength of a column, as well as its ductile displacement. (Vs)(Sezen, Moehle, and Institute 2002):

$$V_n = k(V_c + V_s) = \left[\frac{0.5\sqrt{f_c'}}{L/d}\sqrt{1 + \frac{P}{0.5\,A_g\sqrt{f_c'}}}\right]0.8A_g + k_\mu \frac{A_v f_{yh} d}{s}$$

Shear force is reduced as bending stiffness rises, as indicated by the coefficient k. For example, when severe loads are applied, the aggregate interlock system cracks and becomes weaker, while the steel element will degrade because the bond stress capacity required for an efficient truss will be reduced.

2.4.1.2 Elwood & Moehle Model (2005)

Figure 6 illustrates Elwood's introduction of a quantitative technique established by the experimental and real world results (Kenneth Elwood and Moehle Kenneth J 2005):

$$\delta_{lf} = 0.03 + 4\rho'' - 0.002\frac{v}{\sqrt{f_c'}} - \frac{1}{40}\frac{P}{A_g f_c'} \geq 0.01$$

where:

δ_{lf} = drift when collapse due to lateral loading happens

ρ'' = column transverse steel reinforcement ratio (A_{st}/bs) v = the highest shear stress that was measured in testing (V_{test}/bd)

The model may not be appropriate to columns with specifications that lie beyond the dataset's ranges, as the parameters in Equation 3 were chosen for a minimal square estimate to the research results. Additional research is required to account for demand and shear performance variations (Kenneth Elwood and Moehle Kenneth J 2005).

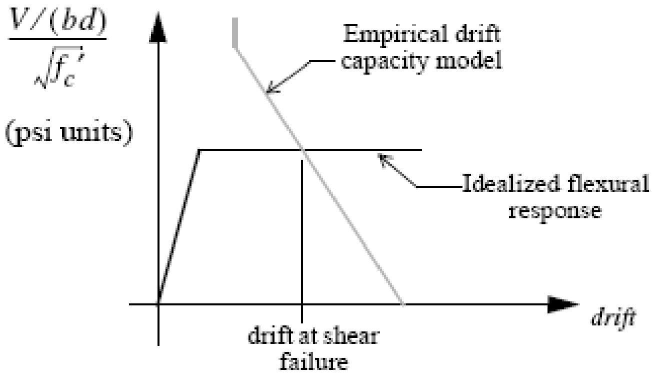

Figure 6: Drift evaluation during failure caused by loading in the lateral direction

2.4.1.3 Zhu approach (2007)

Drift was defined by Zhu et al. as the combination of different factors and ratios to include steel area, loading and other ratios. To cause lateral load failure, the model recommends that transverse reinforcement spacing and the transverse reinforcement ratio, or both, be increased, in addition to the axial load ratio. (Zhu, Elwood, and Haukaas 2007).

$$\delta_{lf} = 2.02\rho_h - \frac{0.025s}{d} + \frac{0.013L}{D} - \frac{0.031P}{A_g f_c'} \quad (4)$$

Where:

ρ_h= column stirrup ratio (A_{st}/bs)

s = stirrup distribution length

D =with of the column under consideration

L = span of shear force

2.4.2 Axial failure

2.4.2.1 Elwood and Moehle Approach

Using shear strength instead of longitudinal reinforcement strength, Elwood and Moehle have suggested the method that calculates drift ratio for compressive failure cases. Figure 7 shows how the transverse reinforcement and the axial load were used to construct this technique for shear-critical RC columns. (Elwood 2004):

$$\delta_{af} = \frac{4}{100}\frac{1+(\tan\theta)^2}{\tan\theta + P\left[\frac{s}{A_v f_{yh} d_c \tan\theta}\right]}$$

where:

d_c= distance between the column center and stirrup center

s = spacing of stirrups of column

A_v= area of columns stirrups of column

f_{yh}= yield strength of transverse reinforcement

P = column’s axial load

θ = angle between the horizontal line and critical cracking (65° assumption).

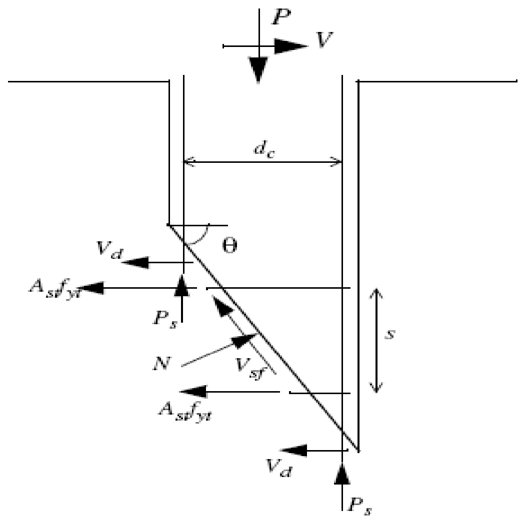

Figure 7: internal loads of the column following a failure caused by lateral forces

2.4.2.2 Ousalem et al. Model (2004)

Ousalem et al. revised Elwood and Moehle's equation by adding the transverse reinforcement rivet motion and the empiric coefficient of friction on an angled plane, as illustrated by Figure 8 and expressed by equations that follows; (Ousalem, Kabeyasawa, and Tasai 2004)

$$\mu = 0.5\,\left(k\,\delta_{af}\right)^{-0.36}$$

Where:

δ_{af}= drift when axial collapse happens

k = $\frac{\rho_h f_{yh}}{n f_c'}$

substituting ;$\left(\mu_{max} = \frac{1-0.003+1.33k}{2\sqrt{0.003+1.33k}}\right)$, equation becomes;

$$\delta_{af} = \frac{1}{k}\left(\frac{0.97-1.33k}{\sqrt{0.03+1.33k}}\right)^{\frac{1}{0.36}}$$

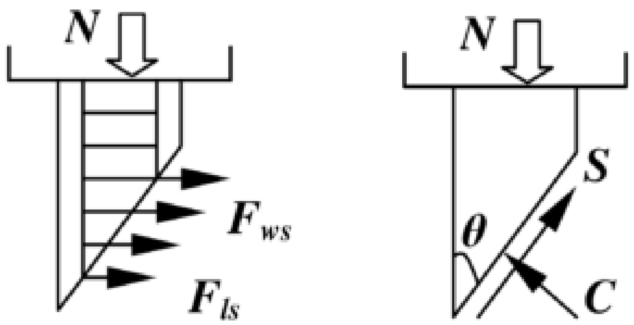

Figure 8: Column shear forces in equilibrium state

2.4.2.3 Zhu approach

Using the Elwood and Moehle(Moehle and Elwood 2003) friction formula (μ), Zhu presented an equation that explains structural drift during axial collapse, depending on 28 column studies as follows (Zhu, Elwood, and Haukaas 2007):

$$\mu=\frac{\left[\frac{P}{V_S}-1\right]}{\left[\frac{P}{2.1V_S}\frac{1}{\tan\theta}+2.1\right]}$$

Where:

$$V_s = \frac{A_v f_{yh}\, d}{s}$$

Drift of axial collapse is formulated using Zhu's findings, which state that the angle θ is equal to 65°, and by adopting this value in the simplified equation, the outcome is: (Zhu, Elwood, and Haukaas 2007);

$$\delta_{af} = 0.184e^{(-1.45\mu)}$$

where:

d_c = column core centerline to transverse reinforcement centerline

A_v = sectional area of the transverse reinforcement in the major direction perpendicular to the applied shear

2.5 Seismic Performance Assessment

It is possible to calculate the consequences of seismic events using a variety of approaches with varying levels of sophistication. As an example, there's a non-linear static approach (NSP), a non-linear dynamic approach (NDA), and a linear static approach (NDP).

2.5.1 Linear Static Procedure (LSP)

Normal or low-rise structural systems react mostly in the basic mode during seismic incidents. When defining the structure as a single-degree-of-freedom system, this technique works well. Due to a response adjustment, the elastic forces are reduced in order to compensate for the inelastic reaction. It is possible to estimate the minimal shear force Vb by using the following equation

$$V_b = \frac{Z\ C\ I}{R} W_t$$

Where:

Z = coefficient of acceleration

C = natural period T1 of normalized earthquake response factor

W_t = overall mass of structure including the proportion of live load

I = importance factor specified by the national code

R = response modification factor

It is possible to anticipate the basic natural period by applying the formula that researchers have established as the top limit of the natural frequency. This equation is derived using the Law of Conservation of Energy, stating that a vibrating structure's potential energy is equal to the kinetic energy of its motion.

$$T_1 = 2\pi\sqrt{\frac{M}{K}} = 2\pi\sqrt{\frac{\sum_{i=1}^{n} W_i\, d_i^2}{g\sum_{i=1}^{n} F_i d_i}}$$

Where:

K is the structure stiffness during lateral loading

Wi is the weight of individual story *i*

Fi is the horizontal forces of individual story *i*

di is the displacement of individual story i

Figure 9 illustrates these concepts of energy, equal displacement, and acceleration and various conclusions that have been reached as a result of Mr. Newmark's study (Veletsos and Newmark 1960) in the 1960s.

The same energy principle has been established for intermediate-period systems as the cumulative energy dampened by the inelastic system is comparable with the same-period elastic system.

Also, in the case of buildings with a greater fundamental period, a nearly identical displacement and behavior is accepted because the top displacements between inelastic systems is analogous with top displacement among elastic systems with a similar frequency.

As observed in systems with stiff SDOF's, short natural periods are comparable to inelastic and elastic systems of same fundamental frequency since the inertia force is comparable across both types of systems.

In this case, the overstrength parameter is used. R_{os} is the relationship between the model and constructed strength of a structure, and it is dependent on the characteristics and application of the material used to construct the building. R_{os} is predicted to be in the range of 1.4 (+0.1) for the majority of structures.

Even though it should be recognized that the response modification factor (R) is a significant generalization for evaluating the influence of inelastic motion and over-strength, the lateral seismic pressure profile (LSP) is an extremely useful technique for assessing lateral earthquake forces exerted on a building.

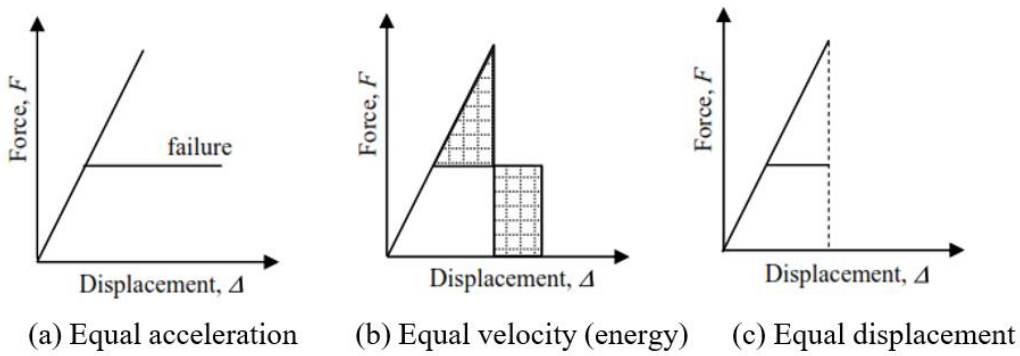

Figure 9: Equal Acceleration (a), equal velocity (b) and equal displacement (c) concepts

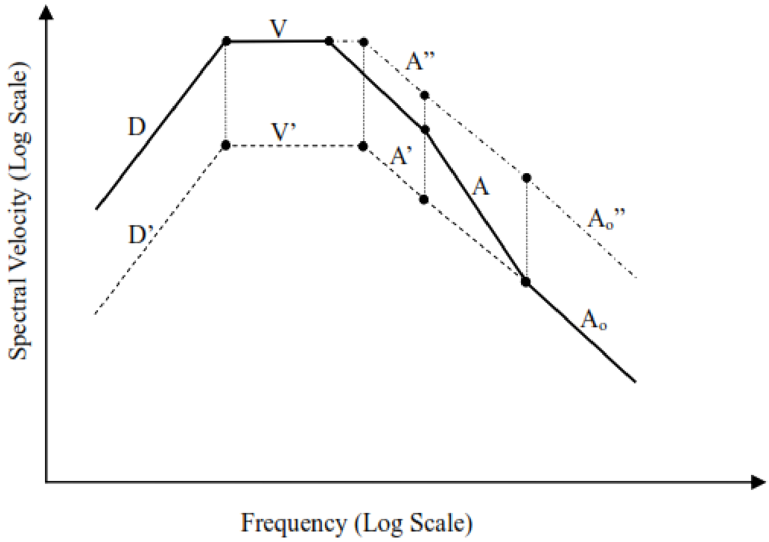

Figure 10: Spectral velocity used in inelastic methods

2.5.2 Linear Dynamic Procedure (LDP)

To be precise, the conventional SDOF system model cannot anticipate the behavior of high and uneven buildings where higher modes are involved. To compensate for the higher-mode seismic response, a dynamic analysis is required. This is done via the modeling of the structure as MDOF model and the assessment of the building utilizing the matrix of structural properties. Therefore, the equation of motion would be expressed as follows:

$$\overline{M}.\overline{\ddot{x}} + \overline{C}.\overline{\dot{x}} + \overline{K}.\overline{x} = \overline{M}.\overline{r}.\overline{\ddot{x}_g}$$

$\overline{r}$ is ground displacement's influence vector

$\overline{\ddot{x}_g}$ maximum acceleration obtained from a seismic event

A response spectrum may be used to characterize seismic motion, while modal analysis can be used to establish the equations governing dynamic motion. Inelastic behavior, such as over-strength and ductility, can properly be accounted for by the response spectrum, which is often narrowed using the response modification factor (R).

In most cases, modal responses are calculated using one of two techniques. In constructions having modes with distant frequencies, the Square Root of the Sum of Squares (SRSS) is the better choice. In contrast, Complete Quadratic Combination (CQC) is a technique that is appropriate for structures that have modes with frequencies that are closely spaced. Natural frequencies may be considered to be separate from one another if the variation between the two consecutive mode frequencies is less than 15 percent between the two consecutive modes.

2.5.3 Non-Linear Static Procedure (NSP)

When applied to systems that mainly operate in their basic mode, the NSP method has a significant advantage that inelastic behavior and overstrength are explicitly simulated, which is a major benefit. The displacement coefficient technique and the capacity spectrum method are the two most often used non-linear static procedures.

2.5.3.1 Displacement Coefficient Method

An important part of this procedure is comparing a building's capacity for movement with its required movement.

Using a static pushover analysis, in which the permanent and imposing lateral loads are introduced to the structure by means of a distribution that is descriptive of the anticipated spread of inertia forces during the seismic event, the displacement capacity may be calculated.

According to FEMA-273, the largest inelastic displacement (Δ t) is determined by the highest elastic linear system displacement (Δ e) with a set of adjustment factors(Nicoletti et al. 1997):

$$\Delta_t = \mathrm{C0C1C2C3}\Delta_e$$

$$\Delta_e = S_a \left(\frac{T_e}{2\pi}\right)^2$$

Where,

C0,C1,C2,C3 are coefficients that are used in the formula. CO is obtained from modal shapes, while C1 is enhanced displacement of the structures, hysteretic shape modifier is C2 and second-order effects are accounted with C3 coefficient. Furthermore, Sa denotes the spectral response acceleration and damping ratio for building's dominant natural period.

The natural period, which represents the frequency at which vibrations subside, Te, and may be estimated as the secant stiffness at 60% of the yield strength, with lateral stiffness equal to that value.

If the displacement capacity of the structure exceeds 150 percent of the maximum displacement requirement at the top level of the building, the structure is deemed acceptable, per the FEMA-273 (Nicoletti et al. 1997).

Whittaker et al. (Lignos, Krawinkler, and Whittaker 2011) verify the method's effectiveness by compared the results of non-linear response-history simulations, which use a yield strength value of 1.3 to a 1.5 ratio of post-yield stiffness to twenty earthquake data sets with a coefficient of friction value of 0.3 to 0.4. It was shown that the displacement coefficient technique for longer structures with higher periods would accurately predict the inelastic displacement, but for shorter structures with lower periods, the method tended to have inaccurate predictions.

2.5.3.2 Capacity Spectrum Approach

By combining the capacity curve derived from the pushover simulation with the demand curve spectrum, the capacity spectrum approach determines the performance point. It consists of three steps:

a) Capacity Curve

Using the acceleration-displacement graph Sa and Sd, the pushover curve corresponding to base shear (Vb) and roof displacement (top) is transformed into the capacity curve;

$$S_a = \frac{V_b}{\frac{\left(\sum_{j=1}^{N} m_j \emptyset_{j1}\right)^2}{\sum_{j=1}^{N} m_j \emptyset_{ji}^2}}$$

$$S_b = \frac{\Delta_{roof}}{\frac{\sum_{j=1}^{N} m_j \emptyset_{j1}}{\sum_{j=1}^{N} m_j \emptyset_{ji}^2} \emptyset_{N1}}$$

Where:

S_a = the spectral acceleration

S_b= the spectral displacement

m_j= story mass of jth story

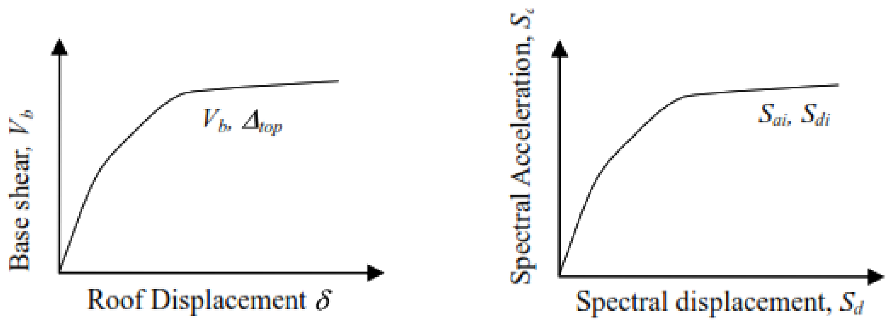

Figure 11: utilization of pushover curve to determine the spectral acceleration and displacement

b) Demand Curve

Design response spectrum (Sa vs T) was transformed to ADRS Sa against Sd with the following equation;

$$S_d = \left(\frac{T}{2\pi}\right)^2 S_2$$

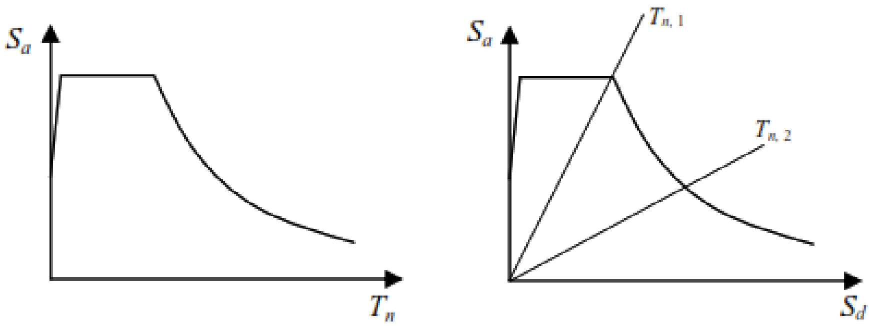

Figure 12: The conversion of the standardized elastic response spectrum to the ADRS format.

c) Performance Point

Figurine 13 shows how to construct a performance point by merging capacity with demand spectrum diagrams. Inelastic energy absorption could be taken into account in the demand curve, thus improving the performance point estimation. We consider the system collapsed if the curves for demand and capacity diverge.

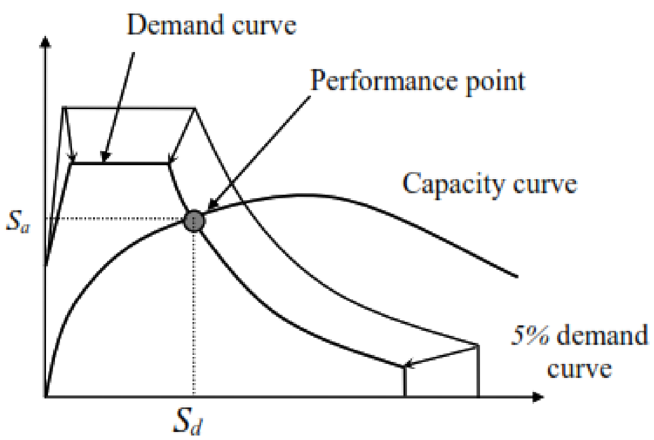

Figure 13: Determination of the performance point during a seismic event

Overstrength Factor

When utilizing a pushover analysis, the overstrength ratio ($\alpha u/\alpha 1$) derived from the analysis should be correlated, the overstrength factor extracted should be compared, and the lower value for the overstrength factor must be used for the lateral dimension in 2 unique load distributions.

Plastic Mechanism

Plastic failure modes should be recognized and described for both lateral load transmission directions. These specified plastic mechanisms must be compatible with the structural behavior factor q identified during the design process.

Target Displacement

The target displacement to be used by the analysis must be obtained from the respective seismic demand, which has been determined in accordance with the elastic response spectrum. This displacement is obtained from an equivalent single degree of freedom equivalent system.

Procedure for the Estimation of the Torsional Effects

Pushover analysis carried out in compliance with the requirements stated in Eurocode 8, which might tend to underestimation of the deformation of the rigid face of the structure, which is supposed to be torsionally flexible. The most important example of this effect can be found in the structures where first and second modes are predominant torsional. In such cases, the stiffer side displacements shall be enhanced.

This enhancement of the rigid/strong side is expected to be applied if the displacement of the rigid/strong side is derived from the elastic modal evaluation of the spatial model.

Torsional effects can be predicted using the same method for torsional effects under lateral force if the planer models used in the model are all regular in their respective axes.

2.5.4 Non-Linear Dynamic Procedure

A non-linear dynamic analysis is advantageous for irregular buildings or structures with a particular function, such as a post-disaster character, since it allows for a more precise approximate for the impact of inelastic action. Non-Linear Dynamic Procedure needs a non-linear time history analysis using a complicated detailed mathematical depiction of the building with detailed non-linear components coupled with small increments of span of time. Than the equilibrium equation of motion may be expressed as follows:

$$\bar{M} \cdot \bar{\ddot{x}} + \bar{C} \cdot \bar{\dot{x}} + S \cdot \bar{x} = \bar{M}.\bar{r}.\bar{\ddot{x}}_g(t)$$

where:

s(x)= inelastic strength matrix

F(t)= vector of forces that are not internal forces.

Formulation 17 is nearly identical to formulation 13, with the exception that it substitutes an inelastic strength term (S) for the stiffness matrix, defining the hysteretic attitude of the diverse MDOF system components. Rayleigh technique may be used to determine the structure's damping matrix (C), given the mass matrix (M) and stiffness matrix (K);

$\bar{C} = a_0\bar{M}+a_1\bar{K}$

As it can be observed from figure 14, coefficients a0 and a1 are adjustable to define the viscous damping proportionality, because the non-linear dynamic solution is very taxing, and it is very reliant to real ground interactions utilized for the mathematical model. Additionally, more than or equal to three ground movements are required to estimate a valid structural reaction. The NDP is successfully utilized as both an academic tool and to assess the operation of important structures in seismically active areas.

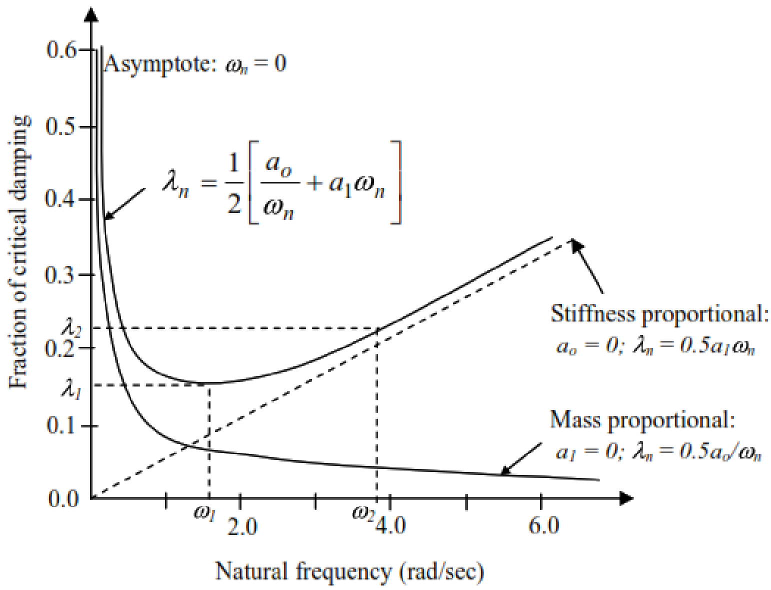

Figure 14: Damping model proposed by Rayleigh.

2.5.5 Comparison of Seismic Performance Evaluation Procedures

Models that respond primarily in the fundamental mode benefit from forced-based and displacement-based methods. The Linear static method contains formulae for mathematically modeling elastic and regular systems, while the non-linear static procedure provides details on systems' inelastic motion.

Dynamic analysis methods like LDP and NDP are suggested for systems where higher-mode effects are significant, such as irregular systems or structures for specific post-disaster purposes. The NDP methodology is the most complex and is frequently used under seismic action to evaluate the performance of critical structures.

In this research analysis, the NDP approach will be used as a benchmark test to compare the forced-based and displacement-based approaches.

Chapter 3

METHODOLOGY

3.1 Building Information

In this study, 3 RC buildings will be designed. To evaluate the differences of the direct building is to be designed using both techniques; the designs will have the same floor plan but have unique displacement-based seismic design and the equivalent lateral force-based method, a residential frame system corresponding to the design requirements of each method, and performance assessment of both methods will be made with time-history analysis (non-linear dynamic analysis). To ensure the direct displacement method or forced-based method does not favor structures with a specific period or specific height, three height variations will be designed and analyzed.

These combinations will have the same floor plan as well, but there will be a four-story, eight-stories, and a sixteen-story variant, ensuring diverse structural periods, thus increasing the accuracy of this investigation. The architectural floor plan of the structure can be seen below in figure 15;

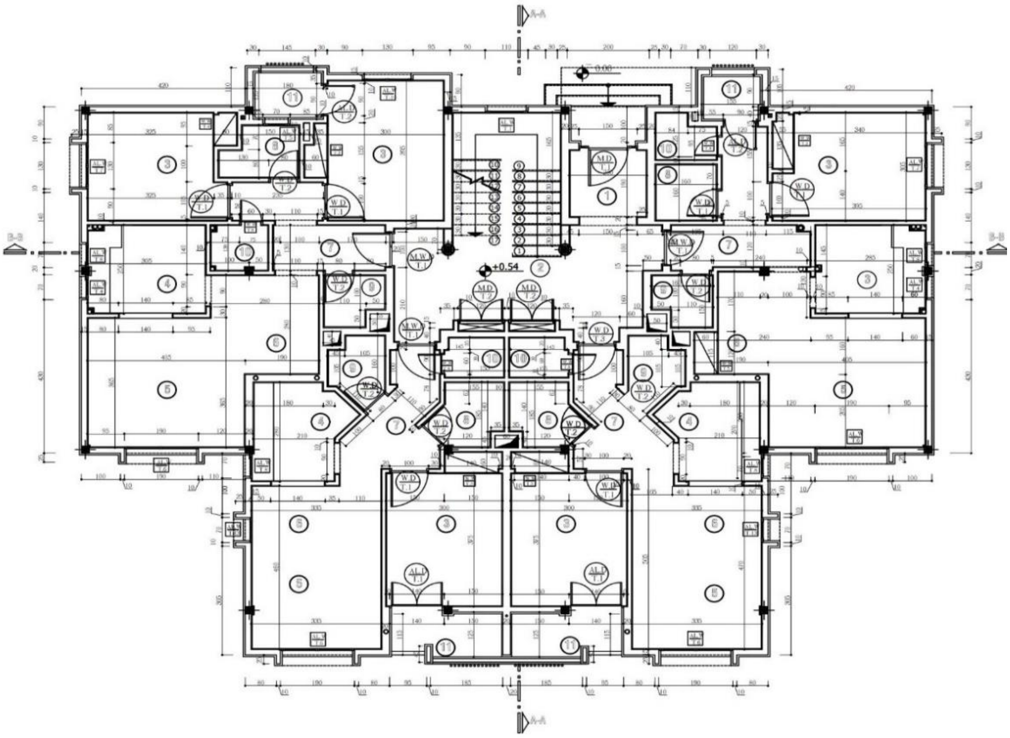

Figure 15: Architectural Floor plan of the structure

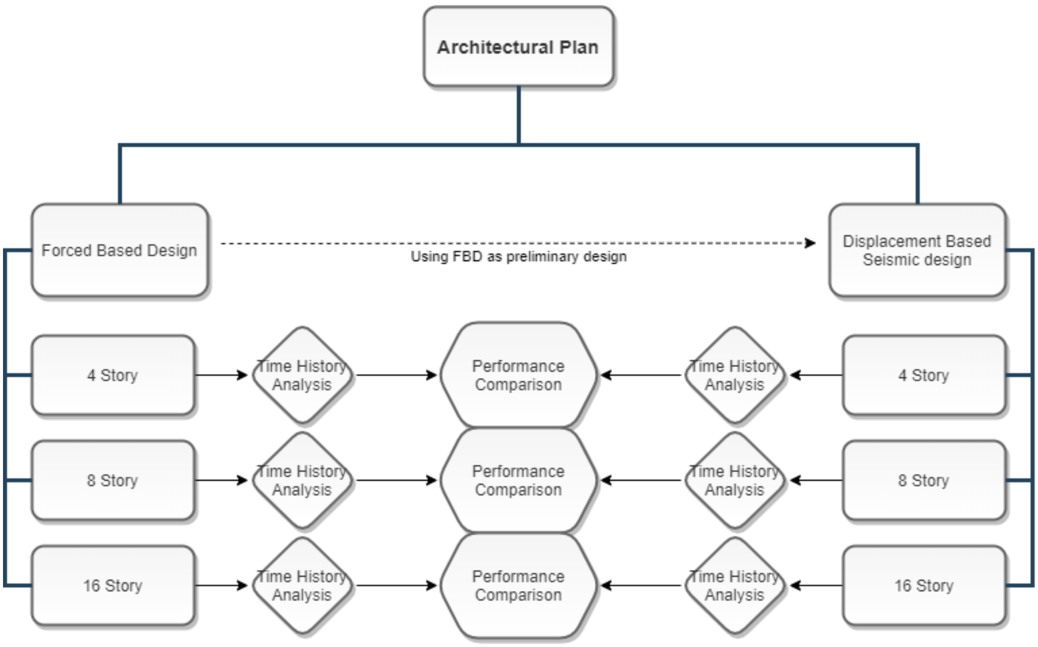

Figure 16: Design plan flowchart

Figure 16 demonstrates the planned workflow to compare the forced-based design and Direct displacement-based design methods. The architectural plan that was discussed will be used to create three height variants, which they will be designed according to the regulations of the forced-based design, as given in Eurocode 8(Lubkowski and Duan 2001). Afterward, a copy of the design will be taken, and the Direct displacement-based seismic design (DDBSD) will be applied. The changes and imposed guidelines will be applied for the Direct Displacement Based Seismic Design, according to Eurocode 8.

The completed designs will be subjected to the Time-History analysis to evaluate the performance of the building. For the performance comparison of the two methods, story drift and plastic hinge formation in the time-history analysis will be compared. Furthermore, the economic differences of the two systems will be compared using the amount of steel (in tons), concrete (in M^3), and formwork (in M2) will be compared for the frame elements. Differences in slabs will be discarded as the study mainly focuses on the design, performance, and economy of the frame system.

Furthermore, it should be noted that in EC8, the use of Forced Based design is forbidden for structures taller structures like 8 and 16 stories high, but the study will omit this restriction to investigate the differences between the two methodologies.

To model and design, and further evaluation of models and comparison, ETABS version 18 ultimate has been used.

3.2 Etabs Version 18 Ultimate

3.2.1 Introducing the Software and its Advantages

In this study, ETABS version 18 ultimate has been used for all modeling and analyzing the building. ETABS has been established as the gold industry standard for Building Assessment and Construction Applications for almost three decades. Today, following a similar tradition, ETABS has evolved as a strongly built structure analysis and design software. The framework developed around an aesthetically pleasing interface, driven by desired modern special algorithms for research and design, with services for drafting and delivering end-results is renewing levels of integration, ingenuity, and technological advancement.

A broad range of structures can be engineered with ETABS software, such as composite structures, moment frames, steel structures, reinforced concrete elements, sloped and joisted slab systems, tower designs, etc...

ETABS is used as a multi-story structural and structural modeling engineering software. Preliminary evaluation of advanced systems may be performed in either dynamic or static scenarios using ETABS. Load assignment facilitating multiple construction codes, modeling methods and tools, various analytics methods, and solution strategies; all operate grid-like geometry, particularly the type of structure. Models for seismic activities can be combined with direct integration with P-Delta to better determine earthquake performance. Non-linear connections correlated with concentrated PMM or fiber hinges will span non-linear behavior as well as monotonic or hysteretic actions. Intuitive and integrated services make it possible to add any criticality from the practical level to the project implementation. ETABS offers a well-managed and efficient application with structures ranging from the usual two-dimensional to more detailed contemporary mega structures. Software is internally manageable and intractable with more design and integration applications.

3.2.2 History of Etabs

A well-recognized and proven structural and earthquake engineering software firm, Computers and Structures Inc., headquartered in Walnut Creek, was founded in 1975. CSI is a provider of a variety of software, including CSiBridge, SAFE, CSiCOL, ETABS, and SAP2000, among others. The most valuable program developed by the Computers and Structures (C&S) Corporation is ETABS, which is used to establish the accurate mathematical model of the Burj Khalifa, the world's tallest building which has been engineered by the Chicago-based Skidmore, Owings & Merrill (SOM). Taking the role of the Design and Development of the world's highest building in their Structural Engineering magazine report on December 2009 in the Structural analysis part: The Burj Dubai renamed as Burj Khalifa. The article claimed that the gravity wind along with earthquake scenarios was evaluated utilizing ETABS. ETABS's non-linear geometric capabilities were given for predicting deformation at great depth with the consideration of P-Delta Effects on the structure. With the launch of this software, it is being trusted by the Civil Engineering community, who are providing advanced structural analysis and design.

3.3 Site Condition and Design Code

The site condition is the same for all buildings, and all the buildings are designed in accordance with European regulations.

Soil type has been selected by group C to be as same as the real condition of Famagusta, and also other factors such as q or structure behavior factor have been chosen according to the Cyprus National Annex(En 2009), and it is equal to 4.

Table 2: soil types and properties(Lubkowski and Duan 2001)

Ground Type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3: Determination of factor q according to Eurocode 8, National Annex of Cyprus (En 2009)

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section **5**	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section **7**	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_u/\alpha_1$

3.4 Force-based design according to Eurocode 8

The FBD uses an approximation of the initial stiffness and elastic damping to predict the necessary strength and final inelastic displacement. Dictated by the Eurocode 8: Design of earthquake-resistant structures is a European standard that provides guidance to the analysis and design of buildings, and structural engineering works in seismic regions. The steps in the implementation of Eurocode 8 are as follows:

1. Conceptual design
2. Determination of the behavior factor q
3. Capacity design
4. Detailing to provide adequate local strength and ductility

The aim of the conceptual design is to reduce the uncertainty of the structural response and to satisfy the fundamental requirements in Eurocode 8.

There are two fundamental requirements in Eurocode 8, which are no-collapse and damage limitation, and to satisfy the two fundamental requirements, the ultimate limit states (ULS) and damage limitation states (DLS) shall be checked. The verifications that are required in Eurocode 8, to check the satisfaction of those limit states by the structure are force-based, sometimes called force-based design (Lubkowski and Duan 2001).

FBD for ductility is based on the inelastic response spectrum of a SDOF system with an elastoperfect plastic force-displacement curve in monotonic loading. For a fixed value of viscous damping, the inelastic spectrum relates;

- Period, T, of the SDOF system
- Behavior factor, q, which is the ratio of the peak force that would have developed if the SDOF system were linear-elastic, to the yield force of the system, and

Maximum displacement demand of the inelastic SDOF system expressed as a ratio to the yield displacement an important factor that is used in the Eurocode, and in other codes as well, is the behavior factor q. The behavior factor is used to reduce the forces obtained from the linear analysis, to account for the non-linear response of the structure. The underlying mechanism of the reduction in force response due to ductility is like the effect of higher viscous damping on an elastic SDOF system, which is energy dissipation. The behavior factor also takes into account the overstrength of the system.

The definition of the behavior factor in Eurocode 8 for a structure in the intermediate-to long period range expresses Newmark's equal displacement rule well, which is an empirical observation that in the constant spectral pseudo-velocity range, the peak displacement of the inelastic and of the elastic response of the elastic SDOF system, are approximately equal. (Fardis 2008)

3.4.1 Design Procedure

The design procedure is shown graphically in Figure 17

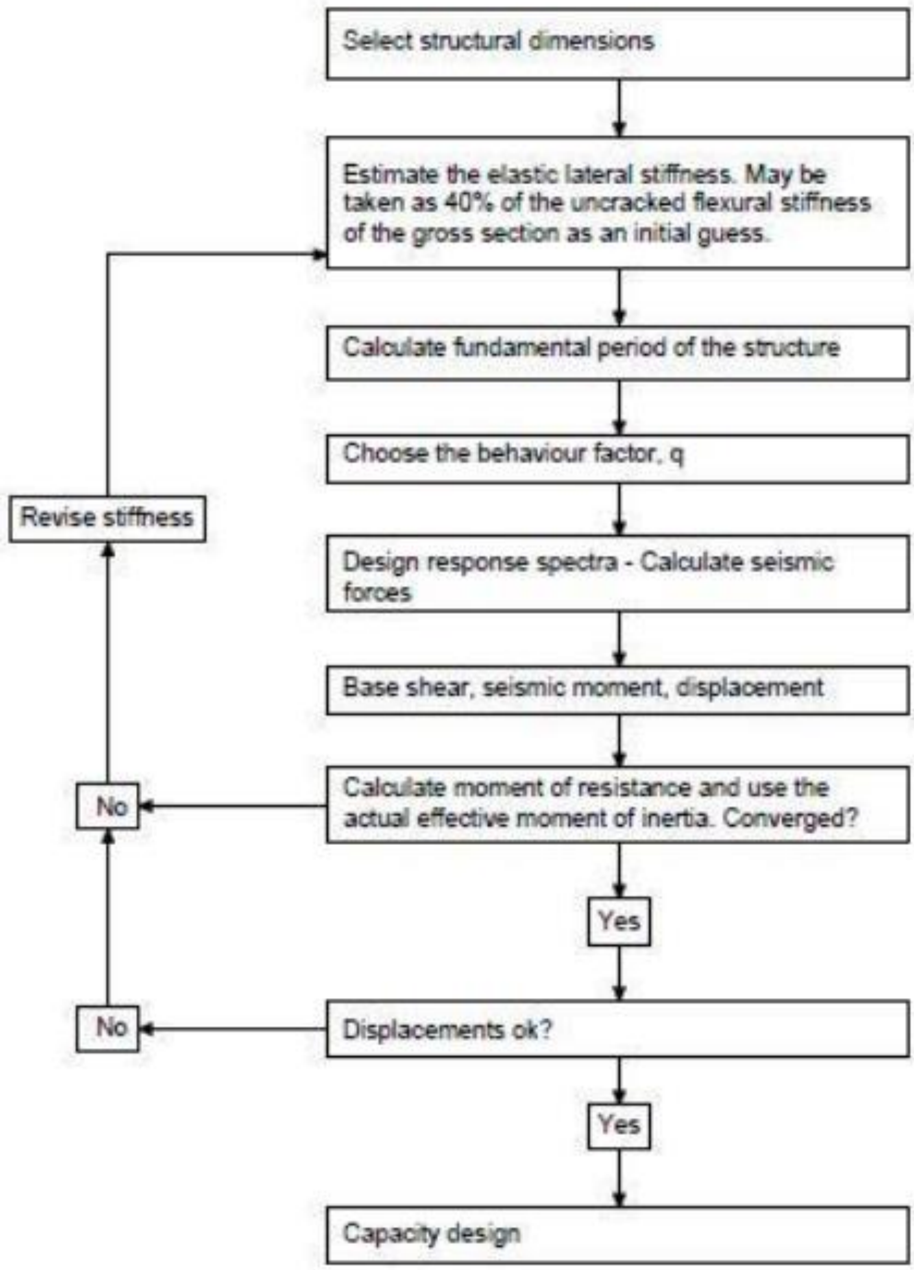

Figure 17: Force Based design procedure

The core premise driving FBD is that both the linear-elastic and non-linear response of structures implementing the same lateral resistance mechanism can be associated by a force-reduction factor (called "behavior" factor, q in EC8). The EC8 defines a q of 2 for this method. For most of the cases, the elastic spectrum is divided by the force-reduction constant to obtain the design acceleration spectrum. Eruocode8 design spectrum contains some adjustments to this method – very short period responses are enhanced, and a minimum spectral acceleration was suggested (equivalent to 0.2PGA = 7.0 percent g).

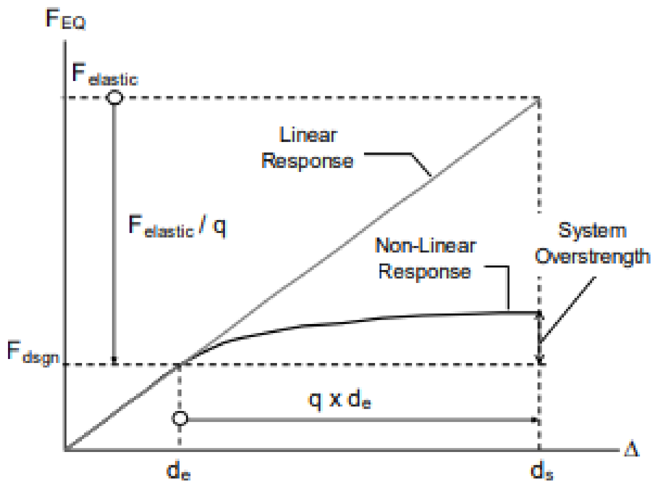

Figure 18: Estimation of behavioral factor q perEC8

3.4.2 Force-Based Design Concept

In cases where T < 4.0sec, the displacement spectrum can be derived from the elastic acceleration spectrum by using the formulation;

$$S_D(T) = S_a(T) \cdot \left(\frac{T}{2\pi}\right)^2$$

When a linear analysis is performed with the design forces, the displacement of a point in the structural system (ds) during the design basis earthquake is estimated by multiplying the analysis value de by the displacement behavior factor qd (equal to q for med-to-long period structures per EC8). Moreover, EC8 states that the value of ds does not need to be larger than the value derived from the elastic spectrum d(SD).

$$d_s = q_d \cdot d_e \leq d(S_D)$$

By substituting Eqn. (3.1) into Eqn. (3.2), the design displacement response is defined;

$$S_{Dd}(T) = q_d \cdot S_{ad}(T) \cdot \left(\frac{T}{2\pi}\right)^2 \leq S_D(T)$$

Figure 19 displays the displacement spectrum both with and without the limit value of SD(T) and the elastic displacement spectrum for the case under analysis. It was found that the minimum spectral acceleration of the design acceleration spectrum described above would result in more drastic spectral displacement for buildings with longer periods opposed to the elastic spectrum. This is not physically feasible and highlights the essential function of state ds ≤ d. (SDe). Unfortunately, we have found that in certain design standards (e.g., the Italian Building Code (T E P R I M A and Verdi 2018) and the Uniform Building Code (Paz et al. 2004), this restriction has been overlooked. On the concluding point, according to EC8 section 5.2.3.4, systems having a natural period greater than the corner period, TC, must possess a ductility capacity of minimum curvature equal to 2q-1, or 6.8 in the instance under consideration. This leads to a precise demand for peak ductility curvature of 6.8. It is important to keep in mind that this curvature ductility capacity may typically be accomplished with low transverse steel reinforcement percentages frequently seen in the designs governed by gravity.

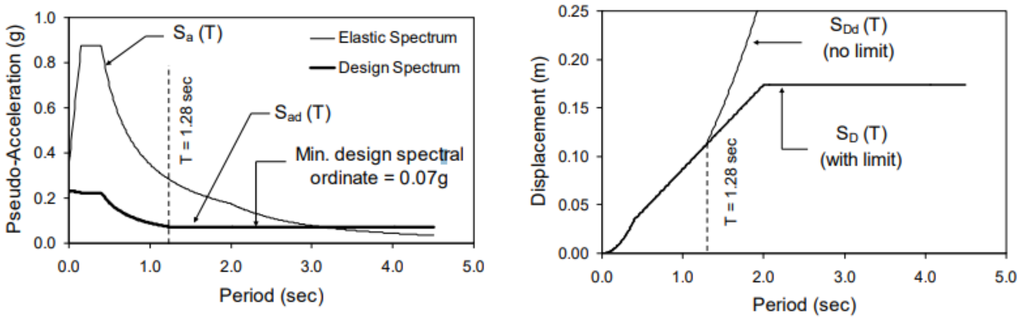

Figure 19: EC 8 Pseudo-acceleration versus time with plotted elastic and design spectra (left) and interpretation of design displacement spectra (right)

3.4.3 Analytical Models

Time-history analysis necessitated the use of "as-designed" modeling. The reaction of the structures was evaluated using the structural analysis module of the computer software RUAUMOKO (Palermo, Pampanin, and Carr 2005). The analytical techniques included many key aspects, including the utilization of mean material properties, minimum concrete responses, reinforcing steel strain hardening, and hysteretic actions in the manner of localized plasticity at frame beam and column element connections. We predicted the flexural response for frame element plastic hinges using a bilinearized moment-curvature response. To describe the plastic-hinge region's cyclically debilitating behavior, the modified Takeda hysteresis rule (Engineering and 1981 n.d.) was utilized. Rayleigh tangent stiffness proportional damping of 5% was developed for structural variations of 4, 8, and 16 stories.

3.4.4 Element Check and Iteration

Once the shear forces acting on the structure are determined, structural element checks and reinforcement detailing is done. If all the elements can sufficiently carry the loads acting on them without exceeding the minimum and maximum steel reinforcement ratios, the design is accepted to be completed. But usually this is not the case, and element sizes predetermined before force based design needs to be altered and usually increased for some members.

Once a change in the dimensions of a member occurs, the Force-Based design procedure should be reapplied as mass, period, stiffness and other properties of the structure changes. Therefore, this iteration is a must. Once a design is completed and all the members pass the design and detailing phase, no further iteration is needed, and structure design is completed.

3.5 Direct-Displacement-Based-Seismic Design

‘’The characteristic design displacement of the substitute structure depends on the limit state displacement or drift of the most critical member of the real structure, and an assumed displacement shape for the structure.’’(Calvi, Priestley, and Kowalsky 2008). The displaced geometry is related to the first mode inelastic seismic loading at the design stage. Therefore, alterations done to the elastic first mode is achieved by altering the stiffness of members locally caused by plastic hinges from inelastic actions, which are considered at the early design phase. However, inelastic and elastic first-mode shapes tend to be akin, using inelastic displacement and characterizing the engineering structure using secant stiffness results in maximum response.

3.5.1 Design Displacement

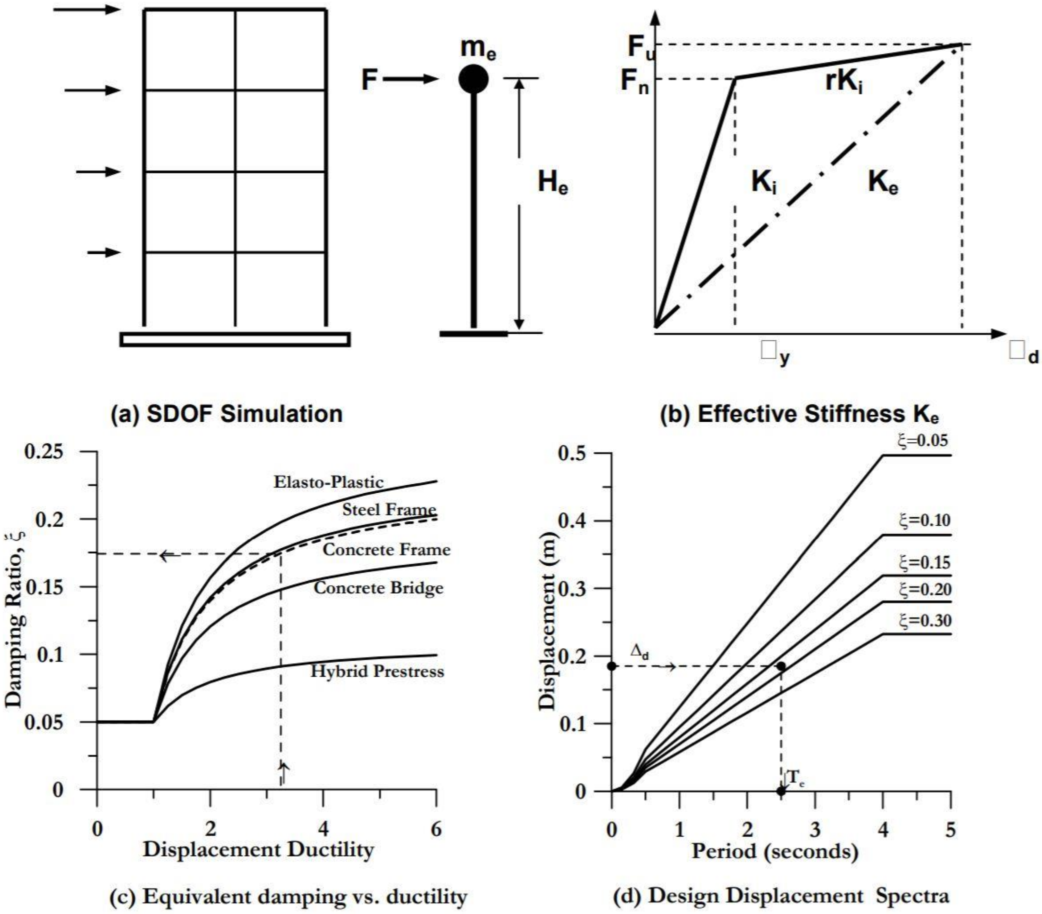

Figure 20: Fundamentals of DDBSD

To be able to perform DDBSD for framed structures, the displacement shape through the first inelastic mode shape should be obtained. For structures with four stories or less, the displacement shape can be obtained from the following formula.

$$\delta_i = \frac{H_i}{H_n}$$

And for structures with more than four stories;

$$\delta_i = \frac{4}{3}\frac{H_i}{H_n}(1 - \frac{H_i}{4H_n})$$

Where;

H_i, is the height of the story under consideration

H_n, is the total height of the building.

Where strain limitations apply, the crucial member's design displacement can be computed by integrating the curvatures associated with the limit strains. Similarly, when code drift constraints are in effect, the same findings administer. To give an instance, the design displacement of the framed building is typically constrained by drift limitations in the building's lower levels. Then the critical displacement, Δ_c, of the structure should be calculated. Usually, the ground floor is where the critical displacement occurs. The critical displacement of the structure can be calculated by multiplying the story height of the critical story by the performance level given in the national build code. After calculating the critical displacement, displacement of the individual stories, Δ_i, can be calculated using the following formula;

$$\Delta_i = \delta_i \left(\frac{\Delta_c}{\delta_c}\right)$$

Where;

δ_i is the calculated displacement shape of the story i

δ_c is the calculated displacement shape of the critical displacement story.

It is worth noting displacements and drifts of the structure due to the effect of higher modes are typically negligible and is therefore ignored early stages during design phase. Dynamic drift may be excessive for tall buildings, therefore requiring a design drift limit reduction. After getting the displaced form and individual displacements for each story, the corresponding SDFO system's design displacement may be computed utilizing the mass of individual stories (m_i).

$$\Delta_d = \sum_{i=1}^{n}(m_i \Delta_i^2) \Big/ \sum_{i=1}^{n}(m_i \Delta_i)$$

Obtaining the equivalent effective displacement Δ_d, allows us to calculate the effective mass of the structure. Typically, the effective mass of multi-story cantilever walls ranges from roughly 70% to more than 85% of the overall mass of frame buildings with more than 20 stories. The remainder of the mass vibrates at a greater frequency. While the elastic base shear force increases significantly compared to the inelastic mode, modal combination methods such as square-root sum-of-squares (SRSS) or complete quadratic combination (CQC) have a much lesser effect on the design base overturning moment. Elastic analyses do not accurately portray the effects of higher modes, which are targeted with higher accuracy at the capacity design phase rather compared to the initial phase of design.

$$m_e = \frac{\sum_{i=1}^{n}(m_i - \Delta_i)}{\Delta_d}$$

Furthermore, the effective height of the structure that participates in the first inelastic mode can be calculated;

$$H_e = \frac{\sum_{i=1}^{n}(m_i \, \Delta_i H_i)}{\sum_{i=1}^{n}(m_i \Delta_i)}$$

3.5.2 Structure Ductility Demand

To determine optimum level of equivalent viscous damping for the structure, it is necessary to know the structural ductility. This is trivial as the design displacement has already been determined, and the yield displacement is geometry-dependent and not strength-dependent. According to Priestley et al. (Calvi, Priestley, and Kowalsky 2008), the following relationships have been discovered for yield drift slope of structural elements framed structures can be expressed as:

$$\theta_y = C_2 \varepsilon_y \frac{L_b}{h_b}$$

Where;

C_2 is 0.5 for concrete frames (dictated by the local structural code)

ε_y is the yield strain of flexural reinforcements within structural elements

L_b is the beam span in the critical floor

h_b is critical story's beam depth

L_b and h_b should be selected from the beam that will reflect the most unfavorable scenario. After obtaining the yield drift slope, yield displacement Δ_y, can be calculated by multiplying the yield drift slope with the effective height of the structure. Furthermore, now the displacement ductility of the structure can easily be calculated using yield displacement and the design displacement.

$$\mu = \frac{\Delta_d}{\Delta_y}$$

3.5.3 Equivalent Viscous Damping and Effective Stiffness

A crucial component of DDBD is that equivalent viscous damping is used to represent hysteretic damping (EVD). Equivalent damping is calculated as a total of elastic, ξ_{el}, and hysteretic, ξ_{hyst} damping:

$$\xi_{eq} = \xi_{el} + \xi_{hyst}$$

Using inelastic time-history analysis, the hysteretic component approach was used to use EVD values that were calibrated for different hysteresis rules to yield the same peak displacements as the hysteretic response.

As a substitute for hysteretic damping, elastic damping is utilized in inelastic time-history analysis. As a result of many different variables, the most notable of which is the hysteretic model's often employed simplifying assumption of completely linear response inside the elastic range. Additionally, foundation compliance and radiation damping, as well as the connectivity of structural and non-structural components, produce additional damping. The chosen stiffness determines the damping coefficient and force. This has been chosen to be the initial stiffness in the majority of inelastic analyses. However, when the response is inelastic, this results in excessive and erroneous damping forces, which is misleading. When tangent stiffness is used, the damping coefficient changes correspondingly when the stiffness changes according to yield, unloading, or reloading. As the structural stiffness weakens as a result of yield, this leads to a decrease in damping force and a decrease in the energy dampened by the elastic system. It is possible to incorporate the elastic damping's ductility dependence within the fundamental form of the equivalent viscous damping equations. The damping–ductility equations assume the following basic form with the frequently used presumed of 5% elastic damping is applied:

$$\xi_{eq} = 0.05 + C_3(\frac{\mu - 1}{\mu\pi})$$

Calculated Equivalent Viscous damping is used to drive the displacement spectra for the structure from the earthquake data for the precisely calculated damping, which will be used to read the equivalent damping of the structure. Derivation of the displacement spectra is discussed in its own chapter. The derived spectral displacement graph is then used to read the equivalent SDFO period of the structure, T_e. The equivalent SDFO period is read according to the design displacement, Δ_d, of the structure. The following graph shows an example of reading the displacement spectra for 15% viscous damping.

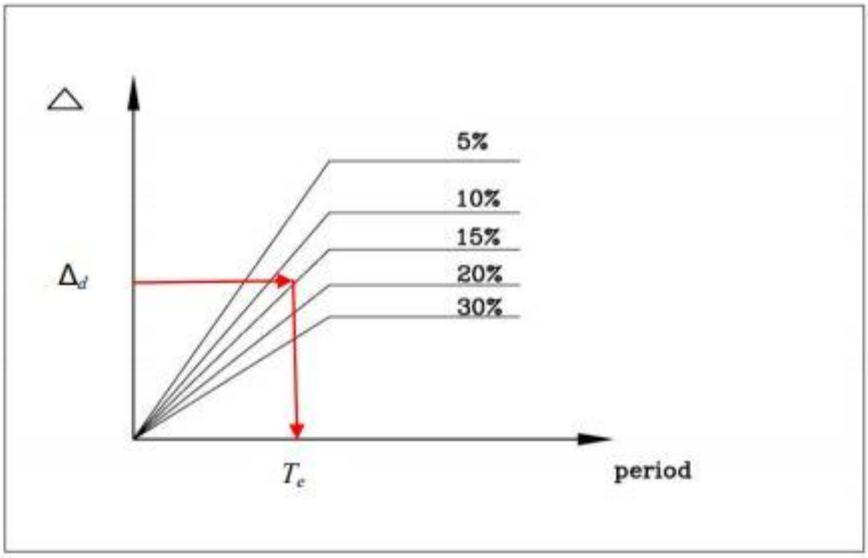

Figure 21: Reading the period of the SDFO system's period from the derived displacement spectra

After obtaining the period of the structure, the equivalent stiffness of SDFO, K_e, can be calculated using the participating mass, Me, and the period obtained from displacement spectra. Using the design displacement at the maximum response as described above and the appropriate damping obtained from the anticipated ductility requirement, we can calculate the effective time Te at maximum displacement at the effective height He. The frequency of an SDOF oscillator may be expressed in the equation for an analogous SDOF system's maximum displacement, and the system's effective stiffness at this maximum displacement can be determined by simply reversing the equation.

$$K_e = 4\pi M_e / T_e^2$$

As the last step, the design base shear force is calculated by multiplying the analogous stiffness of a single degree of freedom with design displacement. Then these forces are distributed to each floor according to the mass percentage of individual stories to the total mass.

$$V_{base} = K_e \Delta_d$$

3.5.4 Modified Direct Displacement Based Seismic Design

As proposed by Mousavi et al. (Fard Mousavi and Sensoy 2019), a first-story-single-degree-of-freedom system is used for obtaining the structural period. As Mousavi demonstrates, in contrast to the original MDOF system, the FSSDOF system is not intended to mimic system's dynamic characteristics; rather, it is a useful tool for determining the base-shear of MDOF systems and for advanced evaluation of drift-ratio constraints.

With the modification to DDBSD, most of the procedure is unchanged, with few modifications. The procedure is unchanged until the calculation of equivalent viscous damping and the creation of displacement spectra. After this point, instead of using the design displacement of the equivalent SDFO system, displacement of the first story is used to find the equivalent period, which is also the critical displacement for all the structures that are under examination. Furthermore, calculation of the equivalent stiffness of the SDFO system, the modified period is used with total mass instead of participating mass. This modification to DDBSD procedure creates a better estimation of design Shear Force as the first story of a multi-degree-of-freedom system has unique properties where;

- First story shear force is the sum of all the shear force acting on the structure, making it equal to the base-shear.
- The axial load on the first story is equal to the structure's overall seismic load.
- The first story has a negligible height change during a dynamic loading.
- The first story has a known exact initial excitation.

As a result of the aforementioned features, the first story may be considered a distinct SDOF system, called the FSSDOF system, which has additional practical benefits.

3.5.5 DDBSD Design Tables

With the implication of the discussed methodology, the design of the three variant structures can begin. To increase the accuracy of the estimation of design base-shear, instead of using minimum element dimensions, the element dimensions from Force-Based Design are used for calculating the mass of the structure. After obtaining the base-shear, elements are designed with the newly obtained forces. After obtaining the new element dimensions, iteration of the DDBSD procedure can be done until the used element dimensions do not change for the newly calculated base-shear. The following tables show the last iteration of the design stages for the 4-story, 8-story, and 16-story structures.

Table 4: DDBSD Design Table for 4- story variant

Story	**Mass X**	hi	δi	Δc	Δi	mi*Δi^2	mi*Δi	mi*Δi*Hi	Vi
#	ton	m	1	m	m	ton.m2	ton.m	ton.m2	kN
Story4	168.8	12	1	0.06	0.24	9.7	40.5	486.1	571.6
Story3	208.4	9	0.75		0.18	6.8	37.5	337.6	529.2
Story2	209.0	6	0.5		0.12	3.0	25.1	150.5	353.8
Story1	210.9	3	0.25		0.06	0.8	12.7	38.0	178.6
					Σ	20.2	115.8	1012.1	1633.1

Δd	0.1749	m
me	81.251	KN.sec2/m
He	8.744	m
lb	5	m
ϴy	0.0125	
Δy	0.1093	m
ϻ	1.6	
ζ	0.1174	%
Te	0.556	sec
Ke	10376	KN/m
Vbase	1814.6	KN

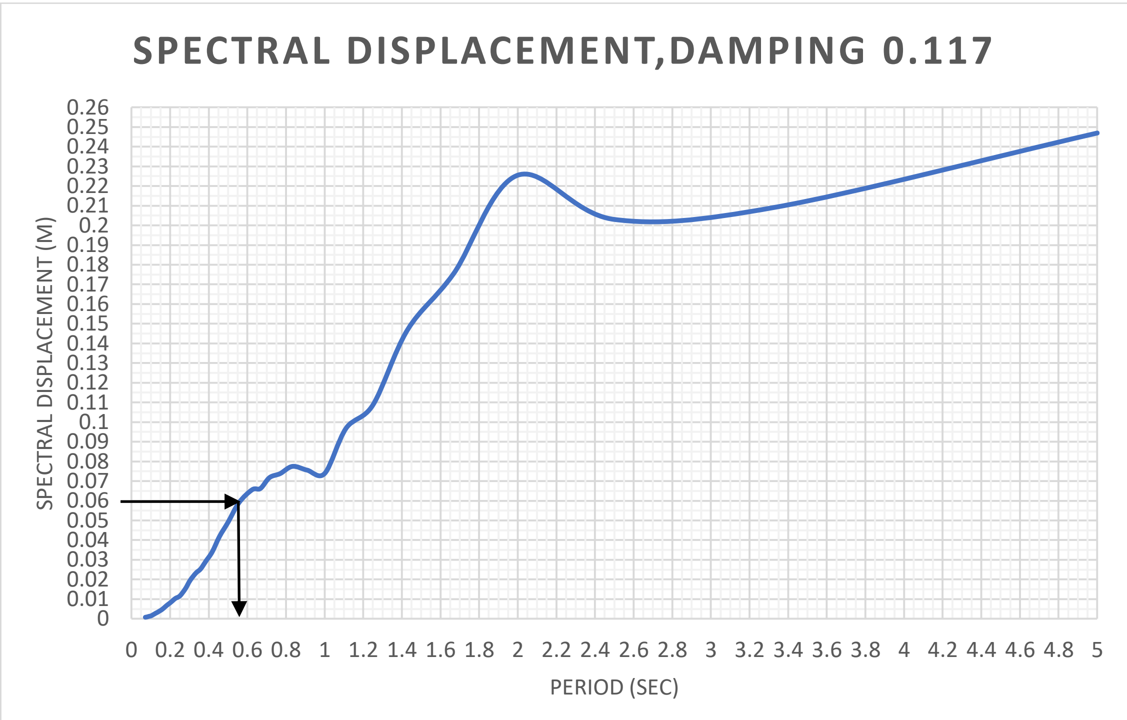

Figure 22: Spectral Displacement for 11.7% Viscous Damping

Table 5: DDBSD Design Table for 8- story variant

Story	Mass X	hi	δi	Δc	Δi	mi*Δi^2	mi*Δi	mi*Δi*Hi	F
#	ton	m	1	m	m	ton.m2	ton.m	ton.m2	kN
Story8	168.65	24	1	0.06	0.3716	23.29	62.67	1504.14	915.69
Story7	175.11	21	0.9115	0.06	0.3387	20.09	59.31	1245.55	866.59
Story6	175.24	18	0.8125	0.06	0.3019	15.98	52.91	952.43	773.09
Story5	175.74	15	0.7031	0.06	0.2613	12.00	45.92	688.79	670.92
Story4	176.61	12	0.5833	0.06	0.2168	8.30	38.29	459.43	559.38
Story3	210.07	9	0.4531	0.06	0.1684	5.96	35.37	318.36	516.84
Story2	211.35	6	0.3125	0.06	0.1161	2.85	24.54	147.26	358.60
Story1	212.71	3	0.1615	0.06	0.0600	0.77	12.76	38.29	186.47
Σ	1505.50				Σ	89.23	331.78	5354.25	4847.58

Δd	0.2689	m
me	153.47	KN.sec2/m
He	16.1379	m
lb	5	m
Өy	0.0125	
Δy	0.2017	m
μ	1.3332	
ζ	0.0949	%
Te	0.55	sec
Ke	20028.35	KN/m
Vbase	5386.20	KN

Modified DDBSD Method		
Δd	0.06	m

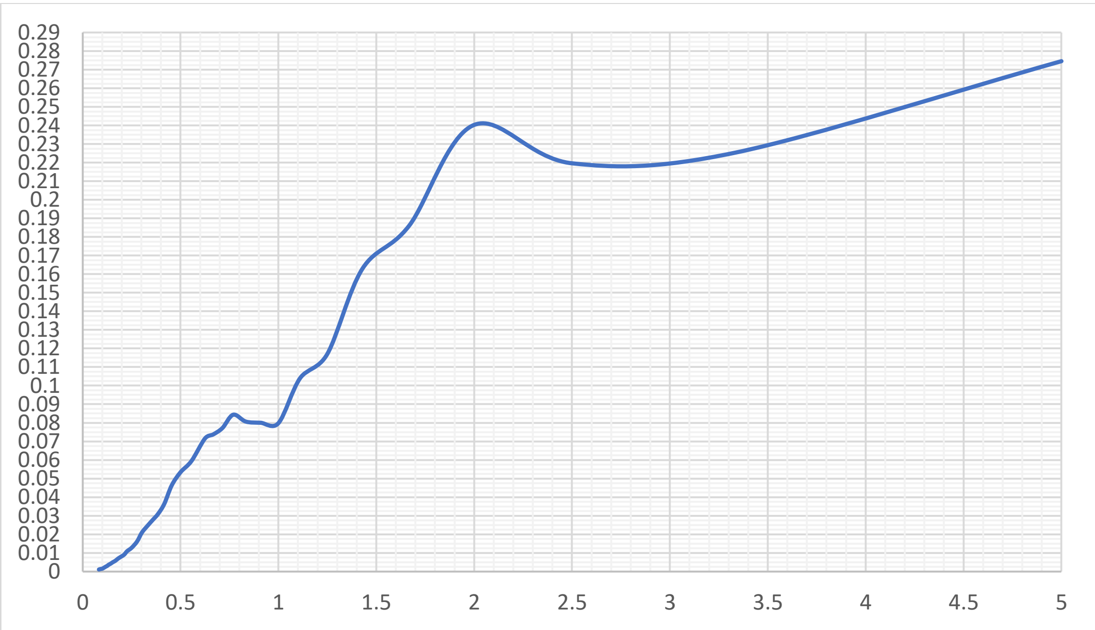

Figure 23: Spectral Displacement for 9.49% Viscous Damping

Table 6: DDBSD Design Table for 16- story variant

Story	Mass X	hi	δi	Δc	Δi	mi*Δi^2	mi*Δi	mi*Δi*Hi	F
#	ton	m	1	m	m	ton.m2	ton.m	ton.m2	kN
16	168.43	48	1.0000	0.06	0.7314	90.11	123.20	5913.47	1763.53
15	174.63	45	0.9570	0.06	0.7000	85.57	122.24	5500.74	1749.81
14	174.72	42	0.9115	0.06	0.6667	77.65	116.48	4892.05	1667.33
13	174.88	39	0.8633	0.06	0.6314	69.73	110.43	4306.59	1580.70
12	175.13	36	0.8125	0.06	0.5943	61.85	104.08	3746.79	1489.83
11	175.82	33	0.7591	0.06	0.5552	54.20	97.62	3221.49	1397.41
10	177.04	30	0.7031	0.06	0.5143	46.83	91.05	2731.50	1303.35
9	177.84	27	0.6445	0.06	0.4714	39.52	83.84	2263.70	1200.15
8	178.75	24	0.5833	0.06	0.4267	32.54	76.27	1830.41	1091.74
7	180.78	21	0.5195	0.06	0.3800	26.11	68.70	1442.66	983.39
6	183.05	18	0.4531	0.06	0.3314	20.11	60.67	1092.02	868.44
5	186.04	15	0.3841	0.06	0.2810	14.69	52.27	784.03	748.21
4	188.15	12	0.3125	0.06	0.2286	9.83	43.01	516.07	615.62
3	221.60	9	0.2383	0.06	0.1743	6.73	38.62	347.59	552.85
2	222.65	6	0.1615	0.06	0.1181	3.11	26.29	157.76	376.39
1	229.04	3	0.0820	0.06	0.0600	0.82	13.74	41.23	196.72
Σ	2988.56				Σ	639.39	1228.49	38788.11	17585.48

Δd	0.5205	m
me	304.64	KN.sec2/m
He	31.5737	m
lb	5	m
ϴy	0.01	
Δy	0.3157	m
ƙ	1.6484	
ζ	0.1207	%
Te	0.566	sec
Ke	37542.21	KN/m
Vbase	19539.43	KN

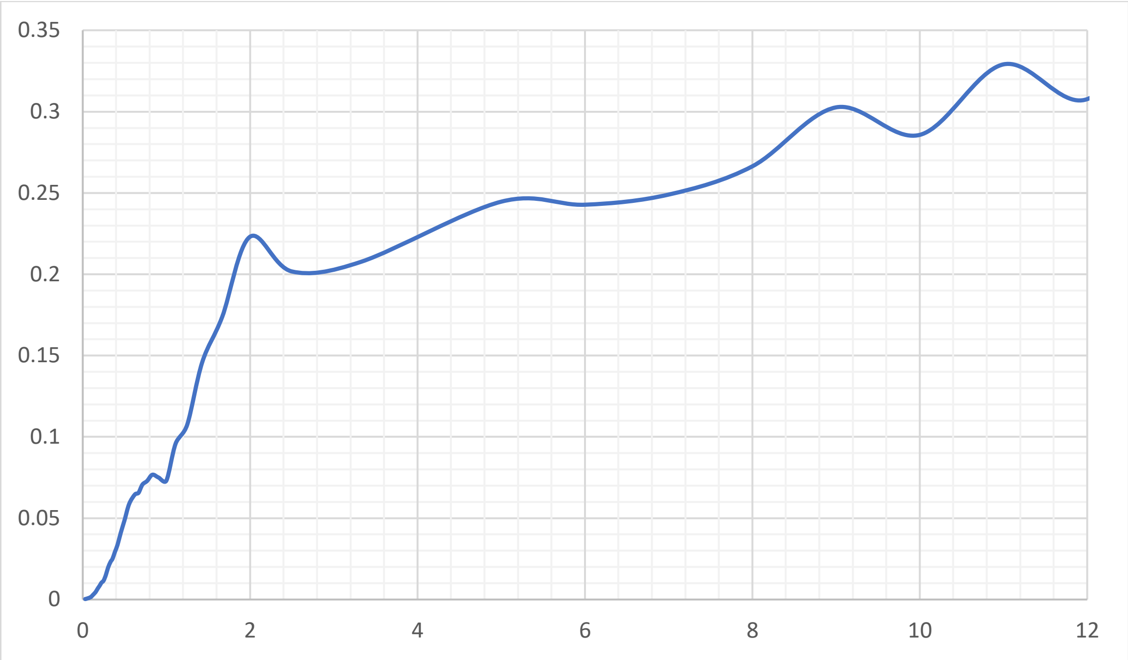

Figure 24: Spectral Displacement for 12.07% Viscous Damping

3.6 Non-Linear Dynamic (Response Time-History) Analysis

3.6.1 Applicable Cases

The response of the structure against time can be obtained using a direct numerical integration of its differential equation of motion, utilizing the ground acceleration.

The structural element model should be within the rules of behavior of the material under post-elastic loading and unloading cycles. The defined rules and specifications of the material should be realistic and model proper energy dissipation of the element over the displacement value ranges in the seismic instances.

When applying different time-history analysis, the worst response should be used for the design as Ed. But if a minimum of 3 different time histories is utilized, the average of the response quantities can be used as Ed in the design phase.

3.6.2 Combination of the Effects of the Components of the Seismic Action

3.6.2.1 Horizontal Components of the Seismic Action

Horizontal components of the seismic action must be accepted as acting simultaneously. Furthermore, the horizontal component should account for;

The response of each structural element should be considered individually, utilizing the combination rule of modal response.

The maximum structural effects of both the horizontal planes then may be estimated using the square root of the sums if the squared values of the values obtained in each direction.

The SSRS method for estimating the action lies on the safe side and gives an overestimate. Therefore, if wished, a more accurate method can be utilized for the estimation of the probable simultaneous values of multiple components.

As an alternative to the previous two methods following formulations can be used;

EEdx "+" 0,30EEdy

0,30EEdx "+" EEdy

For the sign of each component in the calculation of the seismic effects, the most unfavorable response from each axis must be chosen.

When utilizing pushover analysis and a spatial model, the SSRS method must be utilized, considered deformations, and forces resulting from target displacement in x and y direction will be used as EEdx and EEdy, respectively. Furthermore, the combination of the internal forces must not exceed corresponding capacities.

When non-linear time-history is being utilized with a spatial model of the building, ground acceleration recordings acting at the same time must be acting in both horizontal directions.

3.6.2.2 The Vertical Component of the Seismic Action

If the a_{vg} is greater than 0.25 g, the vertical component must be taken into consideration for the cases that follow;

- Horizontal members with a height greater than 25m
- Horizontal cantilevers longer than 5 meters
- Horizontal members with pre-stressed components
- For any beams which support columns

- Base-isolating structures

A partial model of the structure might be used to determining the vertical components of the seismic actions if the model includes the vertical component acting elements and the stiffness of the adjacent elements.

Vertical components of the seismic action are taken into account only for listed elements above and elements directly supporting those elements.

Furthermore, if horizontal components of the listed elements are relevant, all their components must be considered.

$$E_{Edx} \text{"+"}\ 0,30\ E_{Edy} \text{"+"}\ 0,30\ E_{Edz}$$

$$0,30\ E_{Edx} \text{"+"}\ E_{Edy} \text{"+"}\ 0,30\ E_{Edz}$$

$$0,30\ E_{Edx}\ \ 0,30\ E_{Edy} \text{"+"}\ E_{Edz}$$

Where "+" implies combination with and $E_{Edx,}$ $E_{Edy,}$ E_{Edz} is the action effect in the x, y, z axis respectively, due to representative forces along the axis due to seismic action.

Seismic actions vertical effect may be neglected if a non-; linear static analysis is to be performed.

3.6.3 Displacement Calculation

Elastic deformation of the system must be calculated if a linear analysis of the seismic actions is to be performed. To calculate the elastic deformation, a simplified method can be utilized;

$$d_s = q_d d_e$$

Where;

d_s refers to the displacement of a point under examination caused by the seismic action

q_d refers to displacement behavior factor

d_e refers to the displacement of the same point under linear loading obtained from the design response spectrum

When calculating de the effect of l torsional effect caused by the seismic action can be considered to improve the accuracy of the analysis. For static as well as non-linear dynamic analysis, the displacement values were obtained from the analysis without any amplification or modification.

3.6.4 Earthquake Data

Ground acceleration data that will be used for the time-history analysis are gathered from Mr. Safkan's research (Safkan 2018). According to his research, the dataset is gathered from historical earthquakes and represents similar possible seismic activities that might occur in Cyprus. The gathered seismic data is concerted to the pseudo-spectral acceleration as well to provide better insight for the selection of earthquakes that will be used for the analysis. According to fundamental periods of 4-story and 8-story structures, earthquakes GM1, GM2, and GM3 are selected. Furthermore, for structures with 16-stories, additional GM4 and GM14 earthquakes are added. This addition is due to the fact that the fundamental period of the structure is far apart from the other two structures; thus, it is expected not to be excited by the earthquakes chosen for the low-rise and mid-rise structures. Following table 7 displays the earthquake data's origins;

Table 7: Ground Motion Dataset that will be used for Time-History Analysis and determination of PSA (Safkan, Sensoy, and Cagnan 2017)

#	Name	Year	Location	Mag.	RJB	VS	Fault
1	PEER531	1986	Puerta La Cruz	6.1	67.5	442	Reverse
2	PEER686	1987	Whitter	5.9	40.9	390	Reverse
3	PEER	1989	Loma Prieta	6.9	79	623	Reverse
4	PEER208	2002	Alaska	6.7	106	341	Strike Slip
5	PEER016	1975	Oroville	5.7	9.8	590	Normal
6	PEER450	2009	L`Aquila Italy	6.3	60.8	535	Normal
7	PEER112	1995	Kozani, Greece	6.4	72.8	650	Normal
8	PEER026	1980	Sahop Casa	6.3	19.0	242	Strike Slip
9	PEER026	1980	Sahop Casa	6.3	39.1	260	Strike Slip
10	PEER031	1981	Corinth, Greece	6.6	10.3	361	Normal
11	PEER046	1981	Taiwan	5.9	26.4	309	Reverse
12	PEER053	1986	San Jacinto	6.1	30.7	331	Reverse
13	PEER054	1986	Chalfant Valley	6.2	21.6	371	Strike Slip
14	PEER071	1987	Imperial Valley	6.2	17.6	179	Strike Slip
15	PEER330	1999	Chi Chi	6.3	27.6	553	Reverse
16	PEER272	1999	Chi Chi	6.2	76.3	247	Strike Slip
17	PEER441	1983	Borah Peak	6.9	80	324	Normal

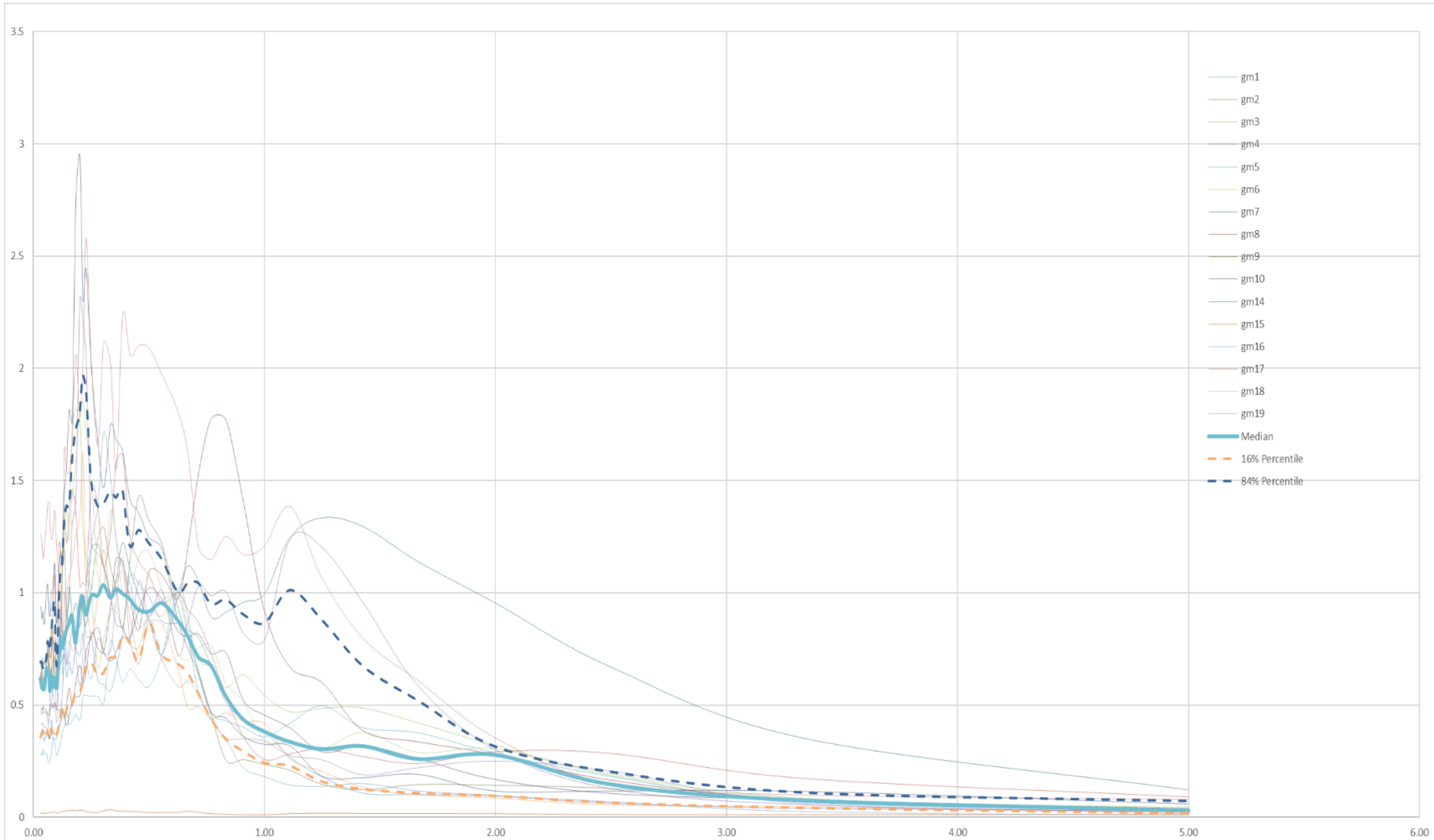

Figure 25: Pseudo spectral acceleration data of individual earthquakes and their median, illustrated visually.

Chapter 4

RESULTS AND DISCUSSIONS

Structures with four-story, eight-story, and sixteen-story have been designed with both forced-based design and direct-displacement-based seismic design. After the completion of both designs, the designed structures have been analyzed by non-linear non-elastic time history analysis for benchmark tests as discussed previously. To be able to compare the performance of the two different design methods, the following results from the design inputs and time history analysis results will be investigated in detail;

- Design Base-Shear
- Story Drift of Diaphragm Center of Mass Displacements from Time history analysis
- Hinge formation and hinge stages from time history analysis
- Structure Economy

After completing both designs, design data and performance data of the building have been extracted to excel, and numerical investigation and analysis of the data are performed. From the data, story drift and hinge statues of the designs are extracted and moved to a new excel sheet with their design counterpart. Then the floors or hinge numbers are synchronized with the counterpart design to be able to compare the performance of the methods. Furthermore, as discussed previously, the economy of the different methodologies has been compared by calculating the amount of steel, formwork, and concrete required for the frame system of the whole building.

4.1 Design Base-Shear

Although design base-shear is not a result obtained from any analysis, it is an input that is used during the design phase. However, design input can be an essential factor to investigate and better understand the difference between the two different design methodologies. The following table 8 shows the design base shear for three height variations of both design methods.

Table 8: Design Base-Shear of two different design methodologies

	Force-Based Design (kN)	Direct Displacement Based Seismic Design (kN)
4	1821	1815
8	2533	5386
16	4304	19539

Table 8 shows that both methods result in a similar base shear for the 4 story structure variant and expected to perform similarly during a seismic event. Nevertheless, it is impossible to comment on the performance level of these structures just with base design shear. Moving to the eight-story variant, it can clearly be observed that there is an increase in design shear for both methodologies. In the eight-story variant, it can be observed that the higher base shear is used for DDBSD design. Force-based design's shear force is approximately 50% of the DDBSD's design shear force. A similar pattern can be observed for 16 story variant but with a much more significant difference between the two methodologies. DDBSD design base shear for 16 story variant is approximately five times greater than the force-based design. Such a difference indicates that one of the methods use will result in either very overdesigned (if DDBSD is not the suitable method) or a severely under-designed and weak structure (if Force-based design is not suitable).

To better understand the difference between two methodologies and resulting base-design forces, investigation of structure masses can provide valuable insights. It is a known fact from structural dynamics, with the increasing lumped mass, the lateral forces acting on the body will increase. Therefore, it can be commented that the more the mass of the structure, the greater the effects of the earthquake will be on the structure. Considering this, the increase in the base shear force should be able to cover up for the increase in mass of the structure to compensate for extra dynamic loads that the structure sustains.

Table 9: Base Shear and Structure Mass increase as a percentage

	Force-Based Design		Direct Displacement Based Seismic Design	
	Increase in Base Shear (%)	Increase in Mass (%)	Increase in Base Shear (%)	Increase in Mass (%)
4	0.00%	0.0%	0.0%	0.0%
8	39.1%	92.2%	196.7%	107.8%
16	136.6%	294.8%	976.5%	467.1%

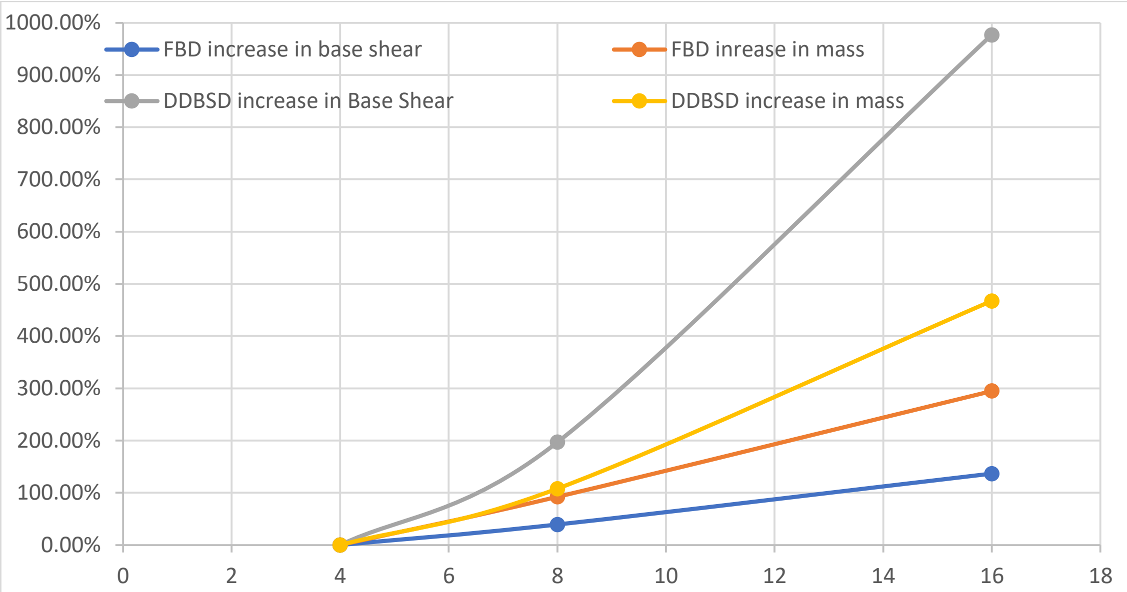

Figure 26: Increase in Base Shear and structural mass

Figure 26 and Table 9 display the mass and the base shear increase as a percentage to 4 story variant. Investigation of mass and design base sheer is a vital coupling that needs to be investigated in detail to understand the working fundamentals of two different methods. Furthermore, it should be noted that the height increase is linear and equal for both cases (three meters for every story). As it can be observed, for the eight-story variant, there is a drastic difference in the increased base shear between the two different methods. While there is a 40% increase in the force-based design, there is approximately a 200% increase in the DDBSD method. Although the starting reference point is not the same for both methods, the increase in mass is approximately 100% higher, which puts the increase in base sheer in perspective. The increase in force-based design is less than half of the mass increase in percentage, while the DDBSD method's increase in base shear is double the mass increase. As discussed before, an increase in mass directly affects the dynamic loads on the structure, and for this case, an increase in force-based design base shear seems insufficient.

Furthermore, a similar pattern can be observed for 16 story variant, with differences being even more prominent. There is only a 136% increase to base shear, while there is a 295% increase in mass. The increase in base shear compared to an increase in mass is inconsequentially, and the earthquake performance of the structure will most probably fall short of what will be necessary for the structure to stay within desired performance level. On the other hand, the increase in base-design shear is more than double the increase in structure mass, which indicates that the DDBSD procedure took into consideration of mass and height increase better than force-based design.

Furthermore, the increase in the masses when compared between two methods indicates a pattern that the DDBSD method results in heavier structures as the height of the structure increase. This pattern is caused by the greater base-shear design dictating the building's structural elements. As the loads on the structure increases, the cross-section of the elements increases as well.

4.2 Story Drift

Story drift is one of the most important outcomes that can be measured during seismic events and further analyzed to understand the structural behavior. To improve the structure performance, the DDBSD method checks the critical displacement to ensure the story drift does not exceed the safe limits. On the other hand, the force-based design does not take story drift into consideration when assigning lateral loads.

Table 10: Maximum story drifts of four-story variants during time-history analysis

	GM 1		GM 2		GM 3	
	FBD	DDBSD	FBD	DDBSD	FBD	DDBSD
	%	%	%	%	%	%
Story4	0.44%	0.42	0.54%	0.56	0.60%	0.60
Story3	0.74%	0.77	0.89%	1.05	1.00%	1.09
Story2	0.85%	0.90	1.09%	1.26	1.15%	1.25
Story1	0.50%	0.59	0.65%	0.85	0.65%	0.80

Table 10 shows the story drifts for four-story variants during three different time-history analyses. As it can be seen from the investigation of the data, it can be deduced that force-based design has and the DDBSD methods have performed similarly. Investigating the story drifts in detail shows that the drift in the critical stories (first and second) is within the acceptable performance level for DDBSD approaches while Forced-Based design have exceeded 1% in GM2 and Gm3 earthquakes at the second floor. Furthermore, there is no vast difference between consecutive floors, which might create plastic hinges and create unwanted forces because of drift and axial loads combining to create moment forces.

Table 11: Story drifts during time-history analysis for eight-story variant structure

	GM 1		GM 2		GM 3	
	DDBSD	**FB**	**DDBSD**	**FB**	**DDBSD**	**FB**
Story	**Drift (%)**	**Drift (%)**	**Drift (%)**	**Drift (%)**	**Drift (%)**	**Drift (%)**
Story8	0.3551	0.36	0.5513	0.67	0.4323	0.80
Story7	0.4809	0.60	0.7657	1.15	0.6114	1.36
Story6	0.4934	0.67	0.8228	1.37	0.6928	1.59
Story5	0.5167	0.64	0.8751	1.42	0.7808	1.61
Story4	0.5139	0.55	0.8952	1.34	0.847	1.51
Story3	0.4972	0.54	0.8857	1.40	0.8773	1.57
Story2	0.476	0.55	0.8551	1.42	0.8882	1.61
Story1	0.287	0.37	0.5117	0.98	0.555	1.12

Table 11 displays the story drifts as percentages for both designs during non-linear non-elastic time-history analysis. Unlike 4-story story drifts, the DDBSD approach is the better performer, where nearly all story drifts are lower than force-based design. Furthermore, it can be observed that the difference between consecutive story drifts are closer to each other and more balance in the DDBSD approach. Furthermore, no single story in the DDBSD method has reached 1% story drift, unlike the force-based method, where drifts exceeding nearly 1.5% have been recorded.

Table 12: sixteen story structure variant's story drifts during time history analysis

Story	GM 1		GM 2		GM 3		GM 4		GM 14	
	DDBSD	**FBD**	**DDBSD**	**FBD**	**DDBSD**	**FBD**	**DDBSD**	**FBD**	**DDBSD**	**FBD**
	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)	Drift (%)
Story16	0.24	0.28	0.33	0.71	0.36	0.50	1.08	0.76	0.79	1.63
Story15	0.33	0.42	0.46	1.05	0.49	0.73	1.50	1.14	1.09	2.47
Story14	0.35	0.47	0.50	1.23	0.52	0.86	1.61	1.24	1.19	2.92
Story13	0.34	0.45	0.51	1.28	0.54	0.90	1.63	1.22	1.25	3.11
Story12	0.31	0.38	0.49	1.27	0.51	0.90	1.54	1.09	1.21	3.17
Story11	0.29	0.33	0.48	1.30	0.51	0.89	1.49	0.90	1.22	3.14
Story10	0.30	0.32	0.49	1.47	0.53	1.02	1.54	1.00	1.29	3.41
Story9	0.30	0.28	0.48	1.56	0.52	1.11	1.50	1.06	1.29	3.47
Story8	0.30	0.27	0.48	1.60	0.54	1.16	1.50	1.11	1.33	3.41
Story7	0.29	0.28	0.47	1.62	0.54	1.20	1.46	1.13	1.33	3.35
Story6	0.26	0.29	0.45	1.56	0.52	1.15	1.37	1.06	1.30	3.05
Story5	0.24	0.33	0.44	1.61	0.51	1.18	1.32	1.06	1.30	3.08
Story4	0.24	0.29	0.43	1.46	0.50	1.08	1.26	0.94	1.30	2.69
Story3	0.24	0.28	0.41	1.34	0.48	0.99	1.15	0.83	1.24	2.39
Story2	0.19	0.28	0.32	1.13	0.38	0.82	0.87	0.66	0.98	2.00
Story1	0.07	0.15	0.11	0.55	0.13	0.39	0.29	0.36	0.34	1.01

Table 12 displays the story drifts for sixteen-story structures during non-elastic non-linear time-history analysis. Examining the story drifts exhibited by both procedures, it can be interpreted that the DDBSD method results in lower story drifts, as expected from the design base-shear. Furthermore, lower design base-shear of force-based design has severely impacted the structure's performance, specifically in GM 14 case, where individual story drifts exceeding 3.4% (more than 10 cm for a 3-meter-tall story) maximum allowed story drift dictated by Eurocode 8. In comparison, the DDBSD method maxed around 1.5% story drift, which is within the immediate occupancy performance level requirements.

Furthermore, when consecutive story drifts are investigated, it can be seen that transition from one story to another, the transition of story drifts is smoother for the DDBSD approach than the FBD approach. Additionally, looking only at first and second story drifts, the forced-based design has less gradual changes, which might lead to a phenomenon called soft-story formation. Although the soft-story phenomenon does not apply to this structure because of structure geometry, this might be a bigger problem which force-based design does not consider when designing.

4.3 Hinge Formation

Story drift is one of the most important aspects to be monitored while evaluating the performance of the building during a seismic event. Structural elements during dynamic loading and might lead to severe damage and even the collapse of the structure. Under this section, both the formation of the hinges and the stage of the hinge will be examined, as the formation of a hinge might not be too damaging to the structure if the hinge state remains within the designed structural limit (life safety, immediate occupancy, collapse prevention, limit damage).

For the four-story variant, as it was evident from the design base shear of the two methodologies, there were failing hinges for both design methodologies. There was slightly more hinge formation for Force Based Seismic Design. For the Direct Displacement Based Seismic Design, there were 6 plastic hinges for the earthquake case GM3. For the earthquake cases GM1 and GM2, no hinge formation passed immediate occupancy performance level for the DDBSD method, while 3 hinges have left the collapse prevention safety level during GM3 earthquake in the structure design with Forced-Based design. A single hinge for forced based design also left the collapse prevention safety level during GM1 earthquake. Although it is a single hinge, structural collapse can be caused by this single weak point that has occurred in the structure.

Table 13: Hinge formation of 4-story variant structure with both DDBSD and FBD Design approaches

	Forced Based Design			Direct-Displacement Based Seismic Design		
	GM1	GM2	GM3	GM1	GM2	GM3
Hinges formed	15	0	44	0	0	7
Hinges Failed	1	0	3	0	0	6

Further investigation of the formed hinges for 4-story variant structure cam can be seen in table 13. As it can be seen, 6 hinges have formed during GM3 earthquake for DDBSD approach. These hinges were all in the ground floor of the structure while the other hinge formed was in the second story. Moreover, all of hinges in the analysis which surpassed pre collapse prevention level were formed in the columns of the structure, at the bottom of the columns. It is an unexpected result, as both methodologies have created a failure scenario in the time-history analysis. The structures are designed with life safety performance levels, and for the GM3 earthquake, the structure has moved past even collapse prevention performance levels. The reason for this extensive hinge formation might be that earthquake gm3 has a nearly resonating frequency with the natural frequency of the structure, leading to extensive hinge formation.

On the other hand, force-based design has created a single hinge in GM1 earthquake time-history analyses that have been performed. This behavior compared to DDBSD in GM3 earthquake, tells that the FBD method leads to structures with lower safety. It was unexpected to see that there was a failure mechanism for FBD but nor for DDBSD method, as both of the approaches had nearly identical design base shear. When designing a structure for a particular performance level, when the structure behaves better than expected, over-design might lead to a considerable increase in cost and effects the economics. Therefore, a structure is expected to be damaged during a seismic event where the earthquake is more damaging than the design expectations.

For the eight-story variant building, as expected from design base shear, formation and statues of hinges have differed for the two methods. DDBSD method variant resulted in some hinge formation between all methods, but none of the formed hinges have exceeded the immediate occupancy performance level, making the structure very safe during the earthquake and dynamic loading. On the other hand, in force-based design, 7 hinges for GM3 and 8 hinges for GM2 earthquakes that have passed the collapse prevention stage, making the structure extremely unsafe, as it can be seen from table 14. Furthermore, it can be seen that all the hinges that formed during GM3 earthquake have formed in the ground floor of the structure, while the hinges formed in the GM2 earthquake are all located on the higher stories. This shows us that column cross-sections of the Forced-Based method is not sufficient and fails to meet the expected safety levels.

Table 14: Hinge formation of Force-Based design of 8 story variant

Story	Output Case	Hinge	Hinge Location	Hinge Status
Story6	gm2	Column	Top	CP
Story6	gm2	Column	Bottom	CP
Story6	gm3	Column	Top	CP
Story6	gm2	Column	Top	CP
Story6	gm3	Column	Bottom	CP
Story5	gm2	Column	Bottom	CP
Story4	gm2	Column	Bottom	CP
Story3	gm2	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP
Story1	gm3	Column	Bottom	CP

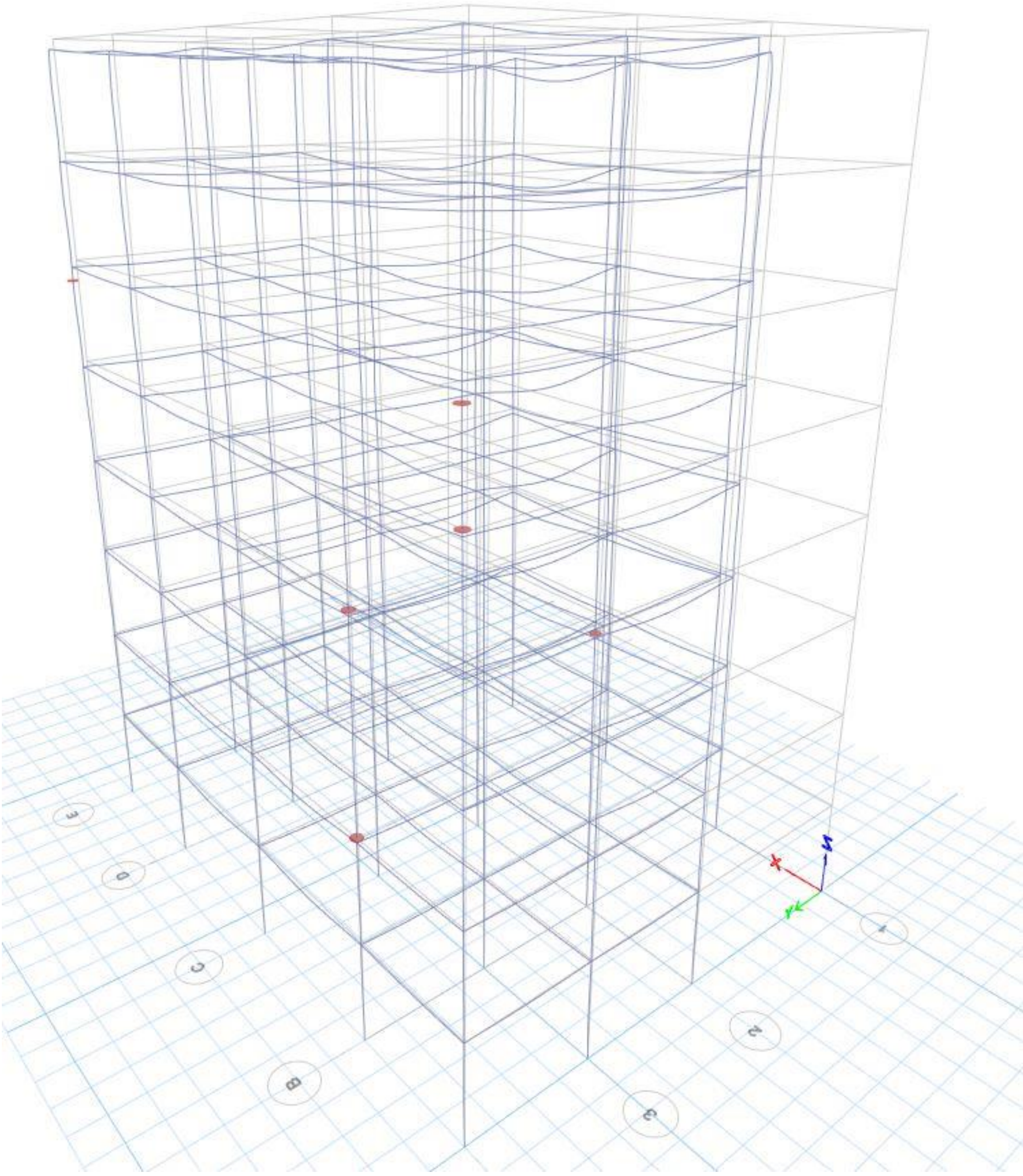

Figure 27: Hinge formation of force-based design in GM2 earthquake

Figure 27 shows the hinge formation for force-based design for GM2 earthquake time history analysis, and as it can be seen, all 6 hinges that formed are all outside acceptable performance limit (Immediate Occupancy to Life Safety performance level). This shows that these elements might be problematic and weaker than they are supposed to be. This might be an indicator that force-based design might be providing an insufficient design base-shear which dictates element sizes.

Table 15: Hinge formation and statues for 16-story structure variants

	GM 14		GM 4		GM 2	
	DDBSD	FBD	DDBSD	FBD	DDBSD	FBD
Total	32	147	2	4	1	1
IO to LS	3	4	1	1	0	0
LS to CP	12	1	0	1	0	0
>CP	17	142	1	2	1	1

Table 15 displays the hinge formation and statues of formed hinges after the seismic activity has taken action. As seen from the table, added earthquake GM 14 to Time-History analysis had the most significant effect on the structure. As it can be seen from the data, there is extensive damage for both design methods. Investigating the hinge numbers and states, the structure designed with DDBSD had 32 hinges formed in the structure, while forced-based design had nearly five times the number of hinges. Further investigation of the hinge statues shows that half of the hinges that formed have passed the collapse prevention level, which does not satisfy the pre-determined performance level of the structure and might lead to collapse or destruction of the building. On the other hand, nearly all of the hinges formed in force-based design have surpassed the collapse prevention stage. Thus, for this earthquake, DDBSD has drastically improved the structure's performance and increased the chances of the structure's survival but fails to stay between the pre-determined performance level.

The earthquake case GM 4 shows that hinge formation for the Force-Based Design is higher than DDBSD by double. Nevertheless, investigating the hinge statutes shows that half of the hinges that formed in force-based design have passed the collapse prevention performance, while the DDBSD method only had a single hinge that passed the collapse prevention method. The rest of the hinges are within Life Safety and Immediate occupancy boundaries. So instead of looking at the total number of hinges, if we look at the number of hinges that passed the collapse prevention stage, we can say that the DDBSD method has decreased the number of danger imposing hinges by half. Therefore, it can be said that the performance of the structure is doubled from this perspective.

Last but not least, the GM 2 earthquake analysis of the structures shows that there a single hinge formation for the DDBSD method, and also there was a single hinge formation that passed the collapse prevention requirements for the forced-based design method and put the structure in grave danger. For the GM 2 case of earthquake, like all the other cases within the sixteen-story variants, the structure's performance has improved with the utilization of the DDBSD method.

4.4 Structural Economy

Although the economy is not a part of performance analysis, it is one of the most important factors for the construction industry in the real world. Therefore, it is important to compare and understand the economic difference between these two methods. To see the cost difference of structural framing members, columns and beams are compared between different methodologies. Slabs are not considered in this calculation, as the economic impact of the change in the slab is negligible. The unit prices for steel (including workmanship), concrete, and formwork (workmanship included) are obtained from the unit price list published by the local contractors association(Cyprus Turkish Building Contractors Association 2020). The prices that are published are Turkish Lira, which can fluctuate or change due to global economics; for this reason, the prices are converted to Euro using the exchange rate of the day where this document is published.

To calculate the financial differences between the Force-Based method and the Direct Displacement Based Seismic Design, the amount of concrete, steel, and formwork is calculated for beams and columns for each structural variant. Then the cost of each material is calculated.

Table 16: Structural frame cost variance of 4-story buildings

	Column			Beam			Total Cost
	Steel	Concrete	Formwork	Steel	Concrete	Formwork	
FBD	€ 5,885	€ 1,334	€ 2,141	€ 8,120	€ 2,602	€ 3,926	€ 24,007
DDBSD	€ 5,374	€ 1,202	€ 2,023	€ 7,294	€ 2,646	€ 3,978	€ 22,517

Table 16 displays the cost of structural elements for a 4-story variant building for both force-based design and direct-displacement-based seismic design. As the force-based design had a greater design base-shear for the four-story variation structure, the total cost is surprisingly lower for this approach. Although the force-based structure performed worse, cost of an additional €1500 compared to Direct-Displacement-Based-Seismic-Design was observed. DDBSD was both cheaper and performed better than forced based design.

Table 17: Structural frame cost variance of 8-story buildings

	Column			Beam			
	Steel	Concrete	Formwork	Steel	Concrete	Formwork	Total
FBD	€ 16,525	€ 3,177	€ 4,850	€ 23,461	€ 6,124	€ 9,091	€ 63,227
DDBSD	€ 22,421	€ 4,218	€ 5,239	€ 31,573	€ 6,783	€ 9,189	€ 79,423

Table 17 shows the economic difference of the 8-story variant structure for the methods under investigation. A quick glance shows that the force-based design is the cheaper option when compared to the DDBSD approach by more than 16000 euros. But choosing force-based design because it would be 20% cheaper might lead to unwanted situations, as discussed for the performance level differences. Considering the performance gain, an increase in the structural steel and concrete makes enough difference to promote the structure to higher levels of performance level. The main reason for the increase in cost is the DDBSD method forcing larger structural cross-sections on the building.

Table 18: Structural frame cost variance of 16-story buildings

	Column			Beam			
	Steel	Concrete	Formwork	Steel	Concrete	Formwork	Total
FBD	€ 42,621	€ 9,288	€ 10,897	€ 69,429	€ 13,798	€ 18,097	€ 164,131
DDBSD	€ 109,459	€ 23,587	€ 16,266	€ 144,632	€ 25,727	€ 24,430	€ 344,102

Table 18 displays the cost of structural framing elements for force-based and direct displacement-based seismic design. There is a huge economic difference between the two methods. The DDBSD method costs more than double of force-based design. But for structures this tall, using the force-based design is completely unacceptable. The 16-story variant designed with a force-based structure cannot survive any considerable dynamic loading, while the DDBSD method has improved the chances of structural survivability drastically.

4.5 Performance Level

Performance level of each structure variant and design methodology is determined for every different time-history analysis that was performed. Performance levels are determined according to the given criteria in Eurocode 8. IT should be noted that all of the structures have been designed by Life Safety performance level. The following table shows the performance level of each design for each earthquake analysis performed.

Table 19: Performance level of each structure in different time-history analysis

	GM1	GM2	GM3	GM4	GM14
FBD 4 Story	CP	LS	CP	-	-
DDBSD 4 Story	IO	LS	CP	-	-
FBD 8 Story	IO	CP	CP	-	-
DDBSD 8 Story	IO	IO	IO	-	-
FBD 16 Story	IO	CP	LS	CP	CP
DDBSD 16 Story	IO	CP	IO	CP	CP

Where;

IO is Immediate Occupancy

LS is Life Safety

CP is collapse prevention

Chapter 5

CONCLUSION AND FURTHER STUDY RECOMMENDATIONS

5.1 Conclusion and Summary

The seismic design has always been one of the most important aspects of structural engineering in the modern world for countries with seismic risk. In this study, the seismic frame system, which is a significantly used load-bearing system in the seismic regions, has been analyzed and compared using both the legacy Force-Based approach and Direct Displacement Based Seismic Design. Even though most seismic codes allow the usage of Force-Based design only for designing multi-story structures, the performance difference of the structures differs according to the method used according to the findings of this study. To evaluate the performance of the different methodology non-linear dynamic, time-history analysis has been performed on the designed models. According to the results of the analysis, structures that have been designed with both the Force-Based and Direct Displacement Based Seismic Design are compared for the performance of the structure and economic differences caused by the methods.

Comparing the design base-shear forces for both designs, an interesting result was discovered. Unlike what was expected for the four-story structure, the design base-shear of the DDBSD approach was very similar to the force-based design. But even with identical total base shear, DDBSD method have performed slightly better for the

4-story structure variant. This results indicates that the distribution of the story shear forces is more accurate for DDBSD method, which allows for better earthquake resistant design. But when structure height and mass increased, an increase in design base shear of force based-design was insufficient, while the DDBSD method, as expected, created much higher forces, generating safer structures during dynamic events. For the mid-and high-rise structures, the increase in base-shear of the force-based design was considerably lower than the mass increase in the structure.

It was observed that even though both design methods were followed according to the specification, the formation of plastic hinges that surpass the allowed limits have formed during the time-history analysis at the taller structures designed with forced-based design. On the other hand, direct-displacement-based have performed flawlessly for mid-rise structure analysis. Although high-rise buildings designed with DDBSD have performed much better than the FBD method, the structure performance was still outside the required limits. Dissipating the seismic loads might be beneficial for the taller structures where higher modes might be dominant. These loads can be dissipated using alternative methods like base isolators, shear walls san structural dampeners.

5.2 Recommendations for Future Studies

To further assess the advantages and disadvantages of the Direct Displacement Based seismic design, the following studies can be conducted;

- Performance assessment of the structures with dampeners designed with Force-Based Design and Direct Displacement Based Seismic Design
- Performance assessment of steel structures designed with Force-Based Design and Direct Displacement Based Seismic Design

- Performance assessment of structures designed to Immediate Occupancy state with Force-Based Design and Direct Displacement Based Seismic Design
- Performance assessment of irregular structures designed with Force-Based Design and Direct Displacement Based Seismic Design
- Performance assessment of structures with shear walls designed with Force-Based Design and Direct Displacement Based Seismic Design
- Effectiveness of Direct Displacement Based Seismic Design under extreme wind loads.

REFERENCES

ASCE. (2013, October). Minimum design loads for buildings and other structures. American Society of Civil Engineers.

Calvi, G. M., Pinho, R., Magenes, G., Bommer, J. J., Restrepo-Vélez, L. F., & Crowley, H. (2006). Development of seismic vulnerability assessment methodologies over the past 30 years. *ISET journal of Earthquake Technology*, *43*(3), 75-104.

Cyprus Turkish Building Contractors Association. 2020. *Yapı İşleri Birim Fiyatları / Construction Works Unit Prices*. Nisosia-Cyprus. Retrived from; https://www.ktimb.org/yapi-isleri-birimi-fiyatlari-m2-fiyatlari/

D'Amico, A., & Currà, E. (2014). Paper 20: Urban Resilience and Urban Structure: Vulnerability assessment of historical Italian towns. *Proceedings ANDROID Residential Doctoral School*.

Eiby, G. A. (1966). The Modified Mercalli scale of earthquake intensity and its use in New Zealand. *New Zealand journal of geology and geophysics*, *9*(1-2), 122-129.

ElAttar, A., Zaghw, A., & Elansary, A. (2014). Comparison between the direct displacement based design and the force based design methods in reinforced concrete framed structures. In *Second European conference on earthquake engineering, Istanbul*.

Elwood, K. J. (2002). *Shake table tests and analytical studies on the gravity load collapse of reinforced concrete frames*. University of California, Berkeley.

Elwood, K. J., & Moehle, J. P. (2005). Axial capacity model for shear-damaged columns. *ACI Structural Journal*, *102*(4), 578.

Eurocode, C. E. N. (2004). 8: Design of structures for earthquake resistance. *Part*, *1*, 1998-1.

Fardis, M. N. (2008). Earthquake-Resistant Design of Concrete Buildings According to EN1998-1 (Eurocode 8). *European Commission: Brussels, Belgium*.

FEMA, F. (2006). 445. Next-Generation Performance-Based Seismic Design Guidelines Program Plan for New and Existing Buildings. *Redwood City*.

Liao, W. C. (2010). *Performance-based plastic design of earthquake resistant reinforced concrete moment frames* (Doctoral dissertation, University of Michigan).

Lignos, D. G., Krawinkler, H., & Whittaker, A. S. (2011). Prediction and validation of sidesway collapse of two scale models of a 4-story steel moment frame. *Earthquake Engineering & Structural Dynamics*, *40*(7), 807-825.

Lubkowski, Z. A., & Duan, X. (2001, November). EN1998 Eurocode 8: Design of structures for earthquake resistance. In *Proceedings of the Institution of Civil Engineers-Civil Engineering* (Vol. 144, No. 6, pp. 55-60). Thomas Telford Ltd.

LUO, W. B., & QIAN, J. R. (2003). RULES OF ELASTOPLASTIC DRIFT DECOMPOSITION OF RC FRAMES AND THE TARGET DRIFT OF MEMBERS [J]. *Engineering Mechanics*, *5*.

Medhekar, M. S., & Kennedy, D. J. L. (2000). Displacement-based seismic design of buildings—theory. *Engineering structures*, *22*(3), 201-209.

Montalvo, J. G., & Reynal-Querol, M. (2019). Earthquakes and terrorism: the long lasting effect of seismic shocks. *Journal of Comparative Economics*, *47*(3), 541-561.

Mousavi, S. M. F., & Sensoy, S. (2019). Direct estimation of the P-delta effect through the “stability-coefficient-response-spectra” by introducing the “first-storey-single-degree-of-freedom” system. *Bulletin of Earthquake Engineering*, *17*(6), 3495-3516.

Murty, C. V. R., Rai, D. C., Bajpai, K. K., & Jain, S. K. (2003). Effectiveness of reinforcement details in exterior reinforced concrete beam-column joints for earthquake resistance. *Structural Journal*, *100*(2), 149-156.

Nicoletti, Joseph, John A. Blume, San Francisco, and Richard Wright. 1997. *Nehrp Guidelines For The Seismic Rehabilitation Of Buildings*.

Otani, S. (1981). Hysteresis models of reinforced concrete for earthquake response analysis. *Journal of the Faculty of Engineering*, 125-159.

Ousalem, H., Kabeyasawa, T., & Tasai, A. (2004, August). Evaluation of ultimate deformation capacity at axial load collapse of reinforced concrete columns. In *Proceedings of 13th world conference on earthquake engineering*.

Palermo, A., Pampanin, S., & Carr, A. J. (2005). Efficiency of simplified alternative modelling approaches to predict the seismic response of precast concrete hybrid systems.

Paz, M., & Leigh, W. (2004). Uniform Building Code 1997: Equivalent Lateral Force Method. In *Structural Dynamics* (pp. 699-729). Springer, Boston, MA.

Priestley, M. J. N., Calvi, G. M., & Kowalsky, M. J. (2005). Direct displacement-based seismic design of structures. *Journal of Earthquake Engineering*, *9*(2), 257-278.

Priestley, M. N. (1997). Myths and fallacies in Earthquake Engineering. *Concrete International*, *19*(2), 54-63.

Priestley, M. N., Calvi, G. M., & Kowalsky, M. J. (2007). *Displacement based seismic design of structures*. Iuss.

Rai, D. C. (2005). Review of documents on seismic evaluation of existing buildings. *Department of Civil Engineering, Indian Institute of Technology Kanpur India*.

Safkan, I., Sensoy, S., & Cagnan, Z. (2017). Seismic behaviour of the old-type gravity load designed deteriorated RC buildings in Cyprus. *Engineering Failure Analysis*, *82*, 198-207.

Safkan, Ismail. 2018. "Influence of Material Variation and Corrosion Deterioration on Seismic Vulnerability Assessment of RC Buildings in North Cyprus."

Sezen, H., & Moehle, J. P. (2002, July). Seismic behavior of shear-critical reinforced concrete building columns. In *Seventh US National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Boston, MA*.

Taranath, B. S. (2004). *Wind and earthquake resistant buildings: Structural analysis and design*. CRC press.

Teprima, Pa. R., and Piazza G. Verdi. 2018. 20 *Gazzetta Ufficiale Della Repubblica Italiana Direzione E Redazione Presso Il Ministero Della Giustizia-Ufficio Pubblicazione Leggi E Decreti-Via Arenula, 70-00186 Roma Amministrazione Presso L'istituto Poligrafico E Zecca Dello Stato-Via Salaria, 691-00138 Roma-Centralino 06-85081-Libreria Dello Stato Aggiornamento Delle «Norme Tecniche per Le Costruzioni»*.

Turer Ahmet, Ahmet Yakut, and Ugurhan Akyuz. *3 Th World Conference on Earthquake Engineering Building Damage Patterns In Bingol-Turkey After The May 1st, 2003 Earthquake*.

Veletsos, A. S., & Newmark, N. M. (1960, July). Effect of inelastic behavior on the response of simple systems to earthquake motions. Department of Civil Engineering, University of Illinois.

Walsh, K. Q., Elwood, K. J., & Ingham, J. M. (2015). Seismic considerations for the Art Deco interwar reinforced-concrete buildings of Napier, New Zealand. *Natural Hazards Review*, *16*(4), 04014035.

Zhu, L., Elwood, K. J., & Haukaas, T. (2007). Classification and seismic safety evaluation of existing reinforced concrete columns. *Journal of Structural*