ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

STRUCTURAL PERFORMANCE EVALUATION OF AN EXISTING RC BUILDING BY NON-LINEAR DYNAMIC ANALYSIS

M.Sc. THESIS

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

MEVCUT BETONARME BİR BİNANIN SİSMİK PERFORMANSININ DOĞRUSAL OLMAYAN DİNAMİK ANALİZE DEĞERLENDİRMESİ

YÜKSEK LİSANS TEZİ

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To my parents and my sisters
who help me in all situations and have given me all of things that I need.
To Assoc.Dr.Beyza Taskin
Who has given me a chance to learn more about structures and understanding of civil science.
To Research Asst.Ülgen Mert Tuğsal
Who help me to understand and solve my thesis problems with patience.

FOREWORD

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April 2015

Mohammadreza Khajehpour
(Civil Engineer)

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ABBREVIATIONS

AIJ	**:** Architectural Institute of Japan
ASCE	**:** American Society of Civil Engineers
BSSC	**:** the Building Seismic Safety Council
CIESN	**:** Centre for International Earth Science Information Network
EPP	**:** Elastic-Perfectly-Plastic
IDA	**:** Incremental Dynamic Analysis
NEHRP	**:** National Earthquake Hazards Reduction Program
RC	**:** Reinforced Concrete
SEAOC	**:** Structural Engineers Association of California
USD	: Ultimate Strength Design

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STRUCTURAL PERFORMANCE EVALUATION OF AN EXISTING RC BUILDING BY NON-LINEAR DYNAMIC ANALYSIS

SUMMARY

Annually more than 17000 individuals are losing their life due to earthquake disaster in the world. For decreasing this number, structures must be performed by earthquake code to be designed for withstanding against ground motion. Aim of this earthquake codes are to minimize loss of life due to the collapsing the yielding building.

Every country in the world has an earthquake code which analyzed and check structures with that but all of them are intend that structures resist large earthquake loading without life-threatening damage and specially without structure collapse or creation of large, heavy failing debris hazard. First earthquake code that had published building regulation enacted in Lisbon, Portugal.

Turkey is one of the most hazard locations accordance to earthquake in the word. Most of Turkey settles on the active faults same as Anatolian tectonic plate. Since 80 years, 12 important earthquake was happened in Turkey that took lots of human life. Turkey is a historically place and most of structures are old so this structures must be analyzed and check that this building can be tolerance ground motion force.

For analyzing the structure there are two methods. First is linear analysis that is divided to linear static analysis and linear dynamic analysis. In linear analysis, structural response is assumed that is linear and stiffness matrix does not change over the time. But with improving technology, it is cleared that structural behavior is not linear and go to nonlinearity against ground motion forces. Second method is nonlinear analysis that divided to nonlinear static analysis which a pattern of force is applied to a structural model that includes nonlinear properties. Total force against target displacement variation to define capacity curve-combined with a demand curve (typically in form ADRS) which cause the problem to single degree of freedom and nonlinear dynamic analysis that is a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual represented by ground motion time histories.

Nonlinear dynamic analysis is the most precise method but it is a complex method too because it needs a lots of information as input and take lots hours for analyzing so this method is used just for very important structures. For nonlinear dynamic analysis ground motion records are need but because we predict earthquake acceleration and we want to check this structures against future ground motion records, artificial ground motion records are used. This records are tougher that real earthquake records.

In this thesis, different analysis methods are explained and then Kasimpasa building that is located in Istanbul is performed by nonlinear dynamic analysis. For analyzing of this

structure, seven different ground motion records are used and then elements damage zone were defined by 2007 Turkish earthquake code and shear force and displacement of this building had been checked with 1975 earthquake code because this building was built in 1979 and it must be analyzed with this code. In conclusion, because some of elements (columns and beams) are located in collapse damage zone so this building must be retrofitting before usage.

MEVCUT BETONARME BİR BİNANIN SİSMİK PERFORMANSININ DOĞRUSAL OLMAYAN DİNAMİK ANALİZE DEĞERLENDİRMESİ

ÖZET

Dünyada, yılda yaklaşık 17.000 kişi deprem felaketi nedeniyle hayatını kaydetmektedir. Bu sayıyı azaltmak yapıların, yer hareketlerine karşı dayanmaları için, deprem koduyla yapılmalarını gerektirir. Bu deprem kodlarının amacı zayıf binaların çökmesi nedeniyle oluşan yaşam kayıplarını en aza indirmektir.

Dünyadaki her ülkenin, yapıları bu konuyla ilgili olarak analiz ve kontrol etmek üzere, bir deprem kodu vardır fakat bunların hepsi de yalnızca yapıların büyük deprem yüküne, yaşama kasteden bir hasar ve özellikle yapı çökmesi ya da büyük enkaz olmadan, direnç gösterebilmesi doğrultusundadır. Deprem yönetmeliği yayınlatan ilk deprem kodu Lizbon-Portekiz'de yasallaşmıştır.

Sismik olarak Türkiye, dünyada deprem karşısında en tehlikeli olan ülkelerden biridir. Türkiye'nin çoğu, devasa Avrasya, Arabistan ve Afrika plakaları arasında ezilen bir alt-plaka, Anadolu tektonik plakası üzerinde oturmaktadır. Arap plakası Türkiye'yi kuzeye doğru iterken, Anadolu plakası saat yönünün tersi yönde hareket eder fakat Avrasya plakasına bağlı olarak, kuzeye hareket etmesi engellenir. Bu durum sismik sürtünmeye ve dağ oluşumuna neden olur ve Türkiye'nin en kalabalık büyük şehri ve birçok aktif faya komşu olan İstanbul'da depremler oluşturur. Kolombiya Üniversite'sindeki The Centre for International Earth Science Information Network (CIESN) (Uluslararası Yer Bilimi Bilgi Ağı Merkezi) İstanbul'u bir "çok yüksek" sismik hasar yerleşimi olarak kategorize eder. Ek olarak CIESN, İstanbul'u depremden doğacak ekonomik kayıp ve ölüm oranlarının en yüksek seviyesine koyar.

Yapısal dinamikler, dinamik denge içindeki bir gövde ya da yapının araştırılması olarak düşünülebilir. Bu denge halinin matematiksel ifadesi hareket denklemidir. Statik denge denklemi yapının iç kuvvetleriyle dışarıdan uygulanan kuvvetler arasında dengeyi ifade ederken, hareket denklemi iç ve dış kuvvet terimlerinin eşitlik noktasını (ki bunlar statik denge denklemindekilerin aynısıdır) ve kütle eylemsizliğini ve sönümlenme etkisini ifade eder. Eylemsizlik terimi yer değiştirmenin zamana göre ikinci türevini, sönümlenme terimi ise birinci türevini kapsarken, hareket denklemi, sabit katsayılarla, ikinci dereceden diferansiyel bir denklemdir.

Yapıyı analiz etmek için iki yöntem vardır. Birincisi, lineer statik analiz ve lineer dinamik analiz olarak bölünmüş olan, lineer analizdir. Lineer analizde, yapısal tepkinin lineer olduğu ve katılık matrisinin zaman boyunca değişmediği kabul edilir.

Diğer yanda, çelik, zorlamadaki değişimlere rağmen gerilimin hemen hemen sabit kaldığı esneklik sınırına ulaştıktan sonra, belirli bir gerilime kadar ("orantılı limit" olarak adlandırılır) lineer olarak esnektir. Bu esneklik sınırının ötesinde, gerilim, akma

sınırındaki dayanıklılığa oldukça yakın gerilimdeki bozulmaya kadar azalırken, zorlamayla (pekleşme) beraber tekrar maksimuma (nihai dayanıklılık, f_{ult}) yükselir. Çelik için bu çalışmada kullanılan elastik-kusursuz-plastik (EPP: elastik-perfectly-plastic) modeli gerilimin zorlamayla birlikte bozulma noktasına kadar lineer olarak değişeceğini ve bunun ötesinde sabit kalacağını kabul eder. Beton ve çeliğin ikisi de lineer olmayan materyaller olduklarından, RC'nin maddesel doğrusal olmayışı her ikisinin de karmaşık bir kombinasyonudur. Bunlar yaklaşık olarak RC tasarımı için Nihai Dayanıklılık Tasarımı (USD: Ultimate Strength Design) yöntemi içinde düşünülürler. Bu çalışmada, bunlar, seksiyonun parçalanmamış olduğu bir aşamayla (artan eğrilikle beraber seksiyonun bükülme momentinin azaldığı bir aşama) başlayıp bozulmaya doğru giden, rastgele bir RC seksiyonu için moment-eğrisi ilişkisini geliştirmek için kullanılmışlardır.

Son on yıllardaki artışla, yüksek binaların ve gökdelenlerin inşası sismik tasarım süreçlerinde sismik direnç ve emniyet konuları bağlamında anlamlı gelişmeler ortaya koymuştur. Sismik analiz yapısal analizin bir alt seti ve depremlere maruz kalmış bir bina yapısının tepkisinin hesaplanmasıdır. Depremlerin hüküm sürdüğü bölgelerde, yapısal tasarım sürecinin bir parçasıdır. Depremin bir artçı dalgası vardır ve bu yüzden, binalar her yönden emniyete alınmalı ve sismik yüklere karşı ayrı ayrı analiz edilmelidir. İkinci yöntem ise deprem hızlanmasının ya da zorlamasının tüm deprem zamanlarında aynı olduğunun kabul edildiği lineer olmayan statik analize bölünmüş olan lineer olmayan analizdir. Bir yapının maddesel ve geometrik doğrusalsızlık ile incelenmesi lineer olmayan statik analizle yapılabilir. Bu yöntemde, depremin neden olduğu ve yapılara, belirli bir noktanın (hedef noktası) yer değiştirmesinin yanal zorlama altında belli bir miktara ulaşmasına ya da yapının yıkılmasına kadar, statik ve dereceli şekilde artarak uygulanan yanal yük, yanal kuvvet altında ya da yapının yıkılmasıyla belli bir miktara ulaşır. Bu yöntem sismik yük altındaki yapısal davranışın iyi bir tahminini sağlar ve çantada keklik yöntemi olarak bilinir.

Lineer olmayan dinamik analiz yer hareketleri kayıtlarının kombinasyonunu ile ayrıntılı bir yapısal modelden yararlanır ve böylece, görece olarak düşük belirsizlikteki sonuçlar üretebilir. Lineer olmayan dinamik analizde, yer hareketi kaydına tabi tutulan ayrıntılı yapısal model, modeldeki her bir özgürlük derecesi için bileşen deformasyonlarının tahminlerini üretir ve model tepkiler karelerin-kare-kök-toplamı gibi planlar kullanılarak birleştirilir.

Lineer olmayan analiz en kesin fakat aynı zamanda karmaşık bir yöntemdir çünkü girdi olarak çok fazla bilgiye ihtiyaç duyar ve analiz saatler alır. Bu yüzden, bu yöntem çok önemli yapılar için kullanılır. Lineer olmayan dinamik analiz için yer hareketi kayıtları ihtiyaçtır çünkü biz deprem hızlanmasını tahmin ediyoruz ve bu yapıları gelecekteki yer hareketi kayıtlarına karşı kontrol etmek istiyoruz. Bunun için yapay yer hareketi kayıtları kullanılmaktadır. Bu kayıtlar gerçek deprem kayıtlarından daha zorludur.

Bir yapının lineer olmayan tepkisi deprem yer hareketi, rüzgâr basıncı, dalga hareketi, patlama, makine titreşimi ve trafikteki hareketler gibi farklı yükleme şartlarından kaynaklanıyor olabilir. Fakat bunların içinde deprem hareketi, yapıların tepkisi üzerinde en çok etkisi olandır ve bu nedenle, deprem sorunlarıyla bağlantılı olarak, lineer olmayan davranış üzerine daha fazla araştırma yapılmıştır.

Yapıların lineer olmayan dinamik analizi eylemsiz kuvvetler, sönümlenme, zorlama oranı etkisi ve osilasyon (salınım) gibi bazı noktalarda lineer olmayan statik analizden farklıdır.

Bu tezde, DRAIN-2DX programından faydalanan yedi farklı deprem kaydına konu olmuş ve kuvvetlendirilmiş bir beton yapının (KASIMPAŞA) lineer olmayan dinamik tepkisini değerlendirmek için bir çaba gösterilmiştir. Çalışmada, binanın yapısal performansı da araştırılmıştır.

İkinci bölümde, ikisi de dinamik ve statik analizler için sınıflandırılmış olan lineer analizden lineer olmayan analize kadar farklı analiz yöntemleri tartışılmıştır. Her bir yöntemin, ikinci bölümde açıklanan, avantajları da dezavantajları da vardır.

Üçüncü bölümde, lineer olmayan dinamik analiz metodolojisi tanımlanmıştır. Hareketin dinamik denkleminden başlayarak, bir binanın sismik yükler altında nasıl yer değiştirdiği gösterilmiş ve yapısal davranış modelleri tarif edilmiştir. Daha ötesi, üçüncü bölümde, RC unsurları için farklı histeretik davranış modelleri gösterilmiştir.

Bu tezde kullanılmış olan lineer olmayan dinamik analiz bilgisayar programı DRAIN-2DX giriş bilgisinden başlayarak programın çıktısına ve gerekli bilgilerin kaydedilmesine kadar programın nasıl kullanılacağını tarif eder.

Son bölümde, dört katlı bir binanın yapısal analizleri sonuçlarla birlikte gösterilmiştir. Lineer olmayan dinamik analiz boyunca tasarım spektrumu ile uyumlu 7 adet simüle edilmiş deprem hareketi kullanılmıştır. Yapısal tepkiler hesaplanmış ve yapısal sistemin performansı değerlendirilmiştir.

Kasımpaşa binası İstanbul'dadır ve lineer olmayan dinamik analize tabi tutulmuştur. Bu yapıyı analiz etmek için yedi farklı yer hareketi kaydı kullanılmış ve sonra 2007 Türkiye kodu ile, elemanlar hasar zonu tanımlanmış ve kesme kuvveti ve binanın yer değiştirmesi 1975 deprem koduyla kontrol edilmiştir çünkü bina 1979 yılında inşa edilmiştir ve bu kodla analiz edilmesi gerekmektedir. Sonuç olarak, bazı unsurlar (kolon ve kirişler) yıkılma hasar zonunda olduğundan bu binanın kullanılmadan önce güçlendirilmesi gerekmektedir.

1 INTRODUCTION

In reinforced concrete (RC) frame structures designed according to current specifications of earthquake resistant design, displacements are expected to exceed those induced by the equivalent static lateral loads stipulated in codes. When these structures are subjected to severe earthquake excitations, they are expected to deform well into the inelastic range and dissipate the large seismic energy input into the structure through large but controllable inelastic deformations at critical regions. In order to predict the distribution of forces and deformations in these structures under the maximum considered earthquake that can occur at the site, accurate models of the hysteretic behavior of the different critical regions of the structural elements are necessary.

For analyzing a structure precisely, it is more realistic to analyze structures with nonlinear analysis methods because of geometric and material nonlinearities.

Linear structural analysis is based on the assumption of small deformations and linear elastic behavior of materials. The analysis is performed on the initial undeformed shape of the structure. As the applied loads increase, this assumption is no longer accurate, because the deformations may cause significant changes in the structural shape. Geometric nonlinearity is the change in the elastic load-deformation characteristics of the structure caused by the change in the structural shape due to large deformations. While this requires complicated formulation, reasonable accuracy can be achieved by suitable approximation of the problem.

For example, in one-dimensional flexural members modeled by the ‘Euler-Bernouli beam, the geometric nonlinearity can be reasonably represented by approximating the strains up to second order terms. This causes a change in the stiffness matrix (with additional nonlinear terms, i.e., function of the displacements) and the resulting structural analysis needs to be performed by iterative methods, like direct iteration (Picard method) or the Newton-Raphson method. These numerical methods are well known and are available in standard texts on structural analysis (Crisfield, 1991; Reddy, 1993).

In RC structures, among the various types of geometric nonlinearity, the structural instability or moment magnification caused by large compressive forces, stiffening of structures caused by large tensile forces, change in structural parameters due to applied loads (e.g., leading to changed damping or parametric resonance) are significant.

Among the two components of RC, concrete is much stronger in compression than in tension (tensile strength is of the order of one-tenth of compressive strength), while its tensile stress-strain relationship is almost linear, the stress-strain relationship in compression is nonlinear from the beginning.

Several researchers have worked on the nonlinear stress-strain relationship of concrete (Hognestad et al, 1952, 1955, 1961; Rusch, 1960; Kaar, 1978). Hognestad model approximates the stress-strain relationship by a parabola up to the ultimate strength (f_c') and a straight line beyond that up to the crushing of concrete. The maximum crushing strain (ε_u) and the strain at ultimate strength of concrete (ε_0) are about 0.003 and 0.002 respectively.

Steel, on the other hand, is linearly elastic up to a certain stress (called the proportional limit) after which it reaches yield point (f_y) where the stress remains almost constant despite the changes in strain. Beyond the yield point, the stress increases again with strain (strain hardening) up to the maximum stress (ultimate strength, f_{ult}) when it decreases until failure at about a stress (f_{brk}) quite close to the yield strength. The elastic-perfectly-plastic (EPP) model for steel, which is used in this work, assumes the stress to vary linearly with strain up to yield point and remain constant beyond that.

Since concrete and steel are both nonlinear materials, the material non linearity of RC is a complex combination of both. They are approximately considered in the Ultimate Strength Design (USD) method for RC design. In this work, they are used to develop the moment-curvature relationship for an arbitrary RC section, beginning with a stage where the section is uncracked, up to the failure (a stage when the bending moment of the section decreases with increased curvature).

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

1.1 Background

Earthquakes are a major problem for mankind, killing thousands each year. For example, that an average of almost 17,000 people per year were killed in the twentieth century.

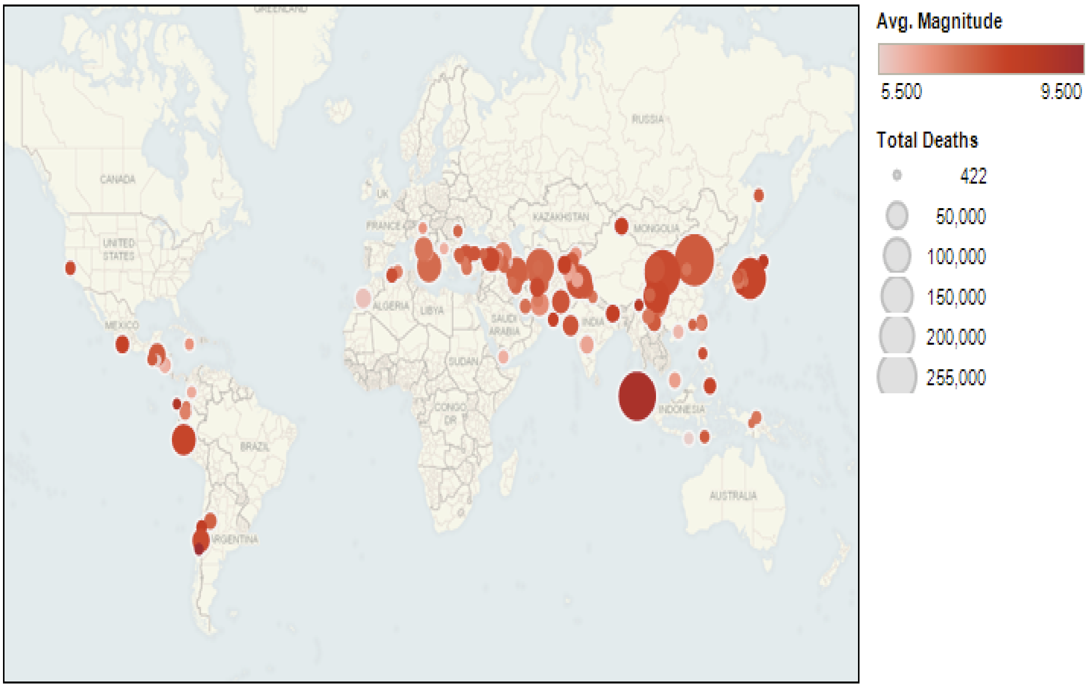


Figure 1.1: A visualization of deadliest earthquake since 1900 (USGS, 2009).

Earthquakes are also multifaceted, sometimes causing life losses and destruction in a wide variety of ways, from building collapse to conflagrations, tsunamis, and landslides.

The essence of earthquake effects on structures is the dynamic nature of earthquake loading. Mechanics as a branch of physics is subdivided into statics and dynamics. Statics studies the systems in static equilibrium, i.e., in a state where the system's internal forces counterbalance external forces acting on the system. Static refers to the fact that the state of the system and the applied forces do not vary in time; they are time-independent. Dynamics however is the study of systems subject to time-varying applied forces. As a consequence of the time variability of the applied forces, the system's internal forces and its state (defined in terms of displacement and deformation) also vary with time. The system's response involves motion. While a static problem has a single time-independent solution, the solution of a dynamic problem involves a description of the system's state at every time point within the period of study. The appearance of inertia effects associated with mass in motion is another key distinction of dynamic problems.

Structural dynamics can be considered as the study of a body or structure in dynamic equilibrium. The mathematical expression of this equilibrium is the equation of motion. While the static equilibrium equation expresses the balance between the structure's internal forces and externally applied forces, the equation of motion expresses the equilibrium of internal and external force terms (which are exactly the same as in the static equilibrium equation) and the mass inertia and damping effects. As the inertia term involves the second derivative and the damping term the first derivative of the displacement with respect to time, the equation of motion is a second-order differential equation with constant coefficients (Dorf, 2004).

Theory for this type of differential equation is well established in mathematics and provides ready tools, both analytic and numerical, for solution of structural dynamics problems.

The purpose of building codes is to promote and protect the public welfare. The public welfare may be broadly construed to include considerations of the health and safety of individual citizens, as well as the economic well-being of the community as a whole. Building codes accomplish this purpose by setting minimum standards for the materials of construction that may be used for structures of different types and occupancies, the minimum permissible strength of these structures, and the amount of deformation that may be tolerated under design loading. Governments have the power to enforce these standards through the code adoption process, i.e., converting the code into a legal standard. If the building code criteria are not specified in a uniform manner, design and construction practice would vary widely, and many structures would be unable to afford their occupants adequate protection against collapse (Dorf, 2004).

Design loading levels are typically set by building codes at levels that have a moderate to low probability of occurrence during the life of the structure. For example, buildings may be designed for earthquake shaking likely to be experienced one time in every 500 years, wind loads anticipated, on the average, one time in every 100 years, or for snow loads that would be anticipated to occur, on average, one time every 20 years.

The significant difference in the recurrence intervals adopted by codes for these various hazards is a function of the hazard itself, and the adequacy of a given return period to capture a maximum, or near maximum, credible event. Building code provisions typically require design for such loading to accomplish two main objectives. The first is to provide a low probability of failure under any likely occurrence of the loading type. This is typically accomplished through prescription of minimum required levels of structural strength. The second is to provide sufficient stiffness such that deflections do not affect the serviceability of the structure, or result in cracking or other damage that would require repair following routine loading. For most structural elements and most loading conditions, these dual design criteria result in structures that are capable of resisting the design loading with either elastic or near-elastic behavior. Consequently, engineered buildings rarely experience structural damage as a result of the effects of dead, live, wind, or snow loads, and rarely completely fail under such loading (Kumar, 2011).

Building code provisions for earthquake-resistant design are unique in that, unlike the provisions for other load conditions, they do not intend that structures be capable of resisting design loading within the elastic, or near-elastic range of response that is, some level of damage is permitted. Building codes intend only that buildings resist large earthquake loading without life-threatening damage and, in particular, without structural collapse or creation of large, heavy falling debris hazards. This unique earthquake design philosophy evolved over time based primarily on two motivating factors. First, even in zones of relatively frequent seismic activity, such as regions around the Pacific Rim, intense earthquakes are rare events, affecting a given region at intervals ranging from a few hundreds to thousands of years. Most buildings will never experience a design earthquake and, therefore, design to resist such events without damage would be economically impractical for most structures. The second reason for this design approach relates to the development history for building code seismic provisions, which is briefly dis-cussed in the next section (Tabeshpour, 2012).

Building code provisions governing design for earthquake resistance may be traced back as far as building regulation enacted in Lisbon, Portugal, following the great earthquake of 1755. Early building code provisions for seismic resistance focused on prohibiting certain types of construction observed to behave poorly in past earthquakes, and to require the use of certain construction details and techniques observed to provide better performance. These features remain an important part of modern codes. However, modern codes supplement these prescriptive requirements with specifications of mini-mum permissible structural strength and stiffness. Although most developed countries develop and enforce their own building codes, the seismic provisions currently used throughout the world generally follow one of four basic models:

a) NEHRP Recommended Provisions, developed by the Building Seismic Safety Council in the United States (BSSC, 1997)

b) Building Standards Law of Japan

c) New Zealand Building Standards Law

d) Eurocode 8

Building code provisions for earthquake resistance may generally be traced to one of three bases. The first of these, herein termed the experience basis, consists of observation of the behavior of real structures in earthquakes, and the development of prescriptive rules intended to prevent construction of buildings with characteristics that are repeatedly observed to result in undesirable behavior (Şafak, 2001).

The second basis is herein termed the theoretical basis. It consists of the body of analytical and laboratory research that has been developed over the years, largely by the academic community, and which provides an understanding of the way structures of different types respond to earthquakes and why.

The final basis is one of designer judgment. The building design community and, in particular, structural engineers — primarily through the SEAOC, the American Society of Civil Engineers (ASCE), the Building Seismic Safety Council (BSSC), Architectural Institute of Japan (AIJ) and other similar groups — have historically taken a leadership role in the development of these building code provisions. These structural engineers have consistently tempered and moderated the information obtained from the experience and theoretical bases, with their independent design judgment, assuring political acceptability of the building code within the design community, if not completely rational or justifiable provisions (Dorf, 2004).

Conventional buildings are mainly designed based on elastic analysis of structures subjected to moderate earthquakes. In this case, the seismic forces are much smaller than the forces introduced by strong ground motions with the considered structural behavior going to nonlinear response during these severe earthquakes. Improving the earthquake resistance of reinforced concrete buildings using a variety of earthquake energy dissipation systems has received considerable attention in recent years by civil engineers.

The nonlinear response and failure mechanism of buildings are taken into consideration by many researchers for the assessment and retrofitting of buildings in recent years. Hence the response spectrum of a building is a good choice for evaluating the seismic response of the structure and nonlinear dynamic analysis provides valuable insight into the full-range of behavior and response of the structure.

The progress of the three dimensional inelastic analysis has been rather slow because of the complicated interactions of bending moment, axial force, and shear in columns in multistory buildings; therefore a lot of effort has been made to accelerate the nonlinear dynamic analysis. In 1974 Bockholt developed the inelastic dynamic analysis of three-dimensional multistory buildings. Emori (1980) adopted the simple assumptions and analytical procedures to reduce the difficulty and complication of inelastic dynamic analysis procedure for reinforced concrete structures and the results were reasonably close to the experimental results. In 1992 Liou introduced a simplified stick model for the nonlinear dynamic analysis of a building based on the strong column-weak beam design philosophy.

Much effort has been devoted in the last twenty five years to the development of models of inelastic response of reinforced concrete elements subjected to large cyclic deformation reversals. Numerous models incorporating information from experimental investigations and on-field observations of the hysteretic behavior of RC structural elements have been proposed. These range from the simple two-component model with bilinear hysteretic law to refined fiber or layer models based on sophisticated descriptions of the cyclic stress-strain behavior of concrete and reinforcing steel.

1.2 Purpose of the Thesis

Building codes require that structures should be designed to withstand a certain intensity of ground motion depending on the seismic hazard. Because of the high forces imparted to the structure by the earthquake, the structures are usually designed to have some yielding. The goal of earthquake engineering is to minimize loss of life due to the collapse of the yielding structure.

Seismically, Turkey is one of the most dangerous countries when the occurrence of destructive earthquakes are examined in the world. Most land area of Turkey settles on the Anatolian tectonic plate and Istanbul, the most populated metropolitan city of the Turkey is also located neighboring this fault. The Centre for International Earth Science Information Network (CIESN) at Columbia University categorises Istanbul as a "very high" seismic hazard location – the highest category. In addition, CIESN places Istanbul in the highest levels of economic loss and mortality, as a result of an earthquake.

Figure 1.2 and Figure 1.3 show seismic hazard map and North Anatolian Fault in Turkey and Figure 1.4 shows major earthquakes, which happened since 1963 till 1999. As it can be seen, Istanbul is a seismically prone city.

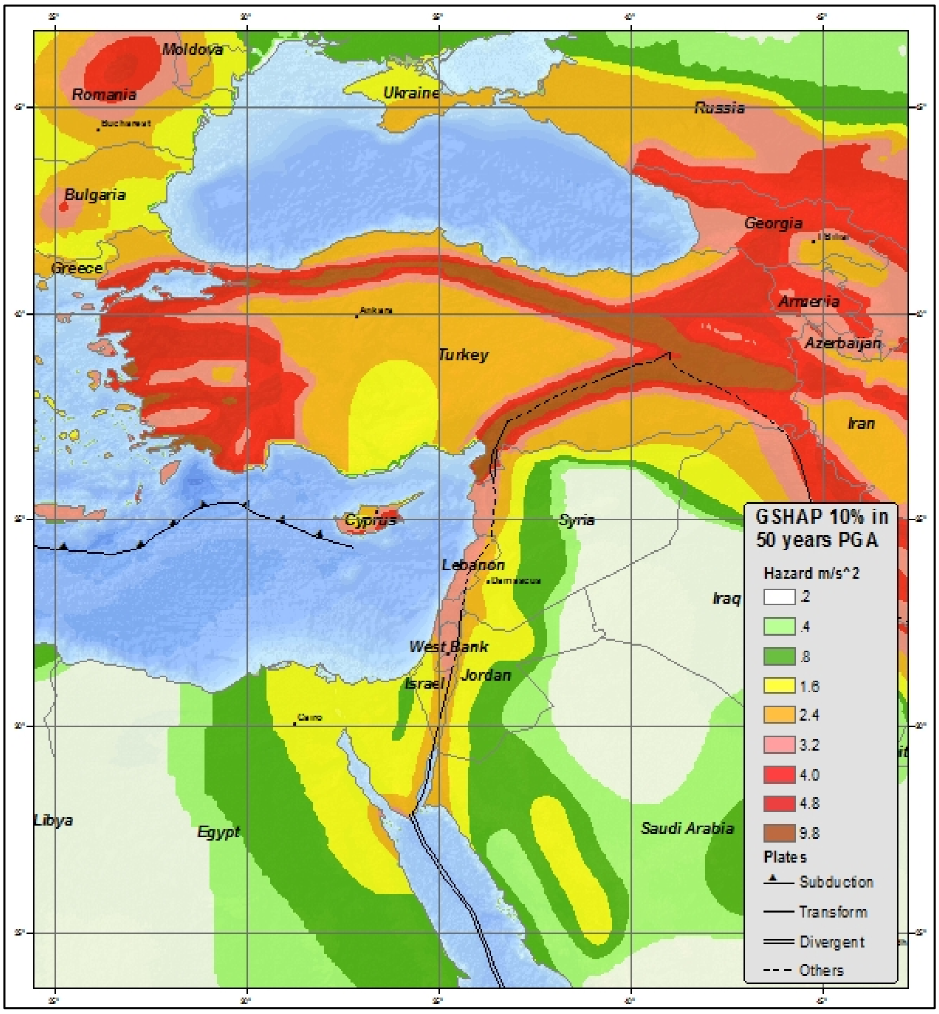


Figure 1.2: Seismic hazard map of Turkey (Cmmiller, 2014).

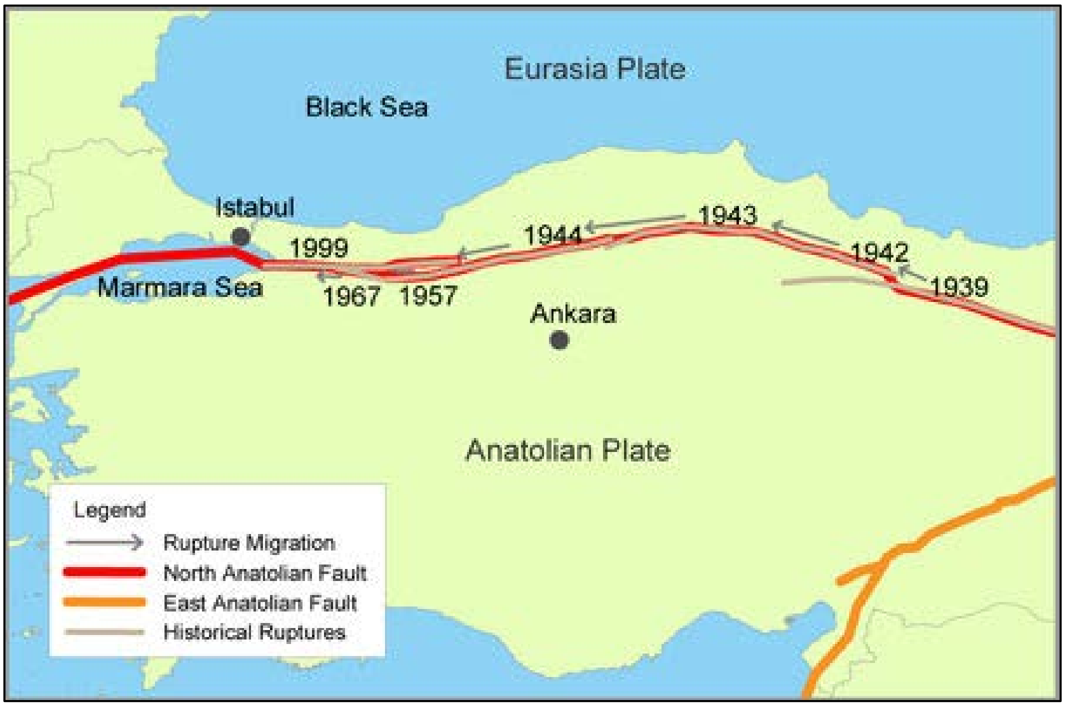


Figure 1.3: North Anatolian faults (Guillermo, 2009).

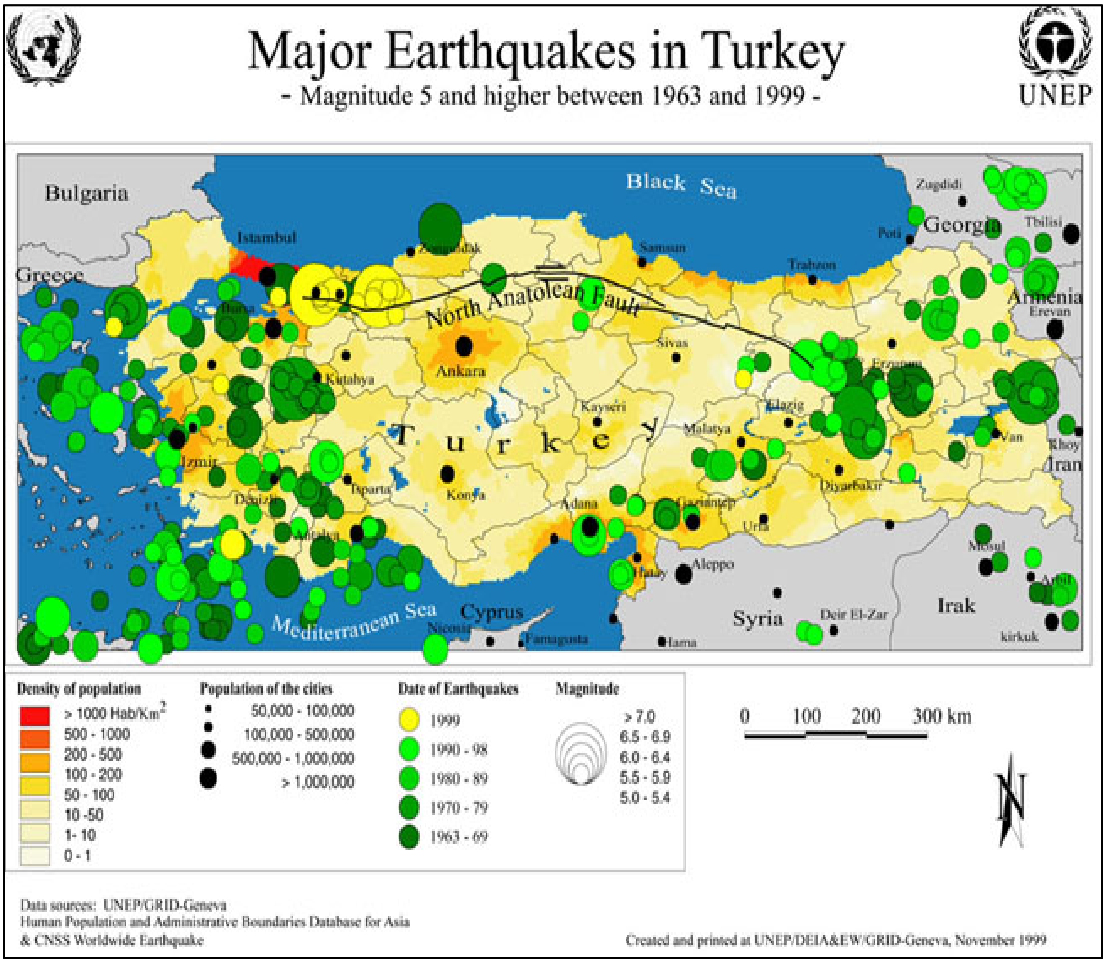


Figure 1.4: Major earthquake in Turkey (thewathers, 2011).

Aim of this thesis is to evaluate seismic performance of an existing reinforced concrete (RC) structures by nonlinear dynamic analysis. Kasimpasa workshop building is located in Istanbul and was built in 1979. In 2010 lots of cracks has been observed on its partition walls and some columns and beams so this structure must be controlled against earthquake before usage. For analyzing this structure, DRAIN-2DX program has been utilized subjected to 7 artificial ground motion records.

In the second chapter different analysis methods are discussed including linear analysis to nonlinear analysis, for which both are classified into dynamic and static analysis. Each method has advantages and disadvantages that is explained in the second chapter.

In the third chapter nonlinear dynamic analysis methodology is defined. Starting from dynamic equation of motion, it is shown how a building is displaced under seismic loads and structural behavior models are described. Furthermore, the non-linear dynamic analysis computer program employed in this thesis, DRAIN-2DX is described.

In the last chapter, structural analysis of an existing four story RC workshop building is carried out and the structural responses are calculated. During the non-linear dynamic analysis 7 artificial earthquake motions are employed, which are compatible with the design spectrum. After analyzing of this building with DRAIN-2DX program, reinforcement and concrete strains are compared to 2007 Turkish Earthquake Code for identifying damage state of each element and then nonlinear dynamic results are compared to linear results which are taken from previous investigations by Istanbul Technical University teams. Finally, contribution of the partition walls in the structural responses are also evaluated by removing them from the analytical model and recomputing the structural responses.

2 SEISMIC PERFORMANCE ANALYSIS

2.1 Introduction

In the last decades, increment in the construction of high rise buildings and skyscrapers, significantly yielded improvements in seismic design procedures in terms of seismic resistance and safety issues. Seismic analysis is a subset of structural analysis and is the calculation of the response of a building structure subjected to earthquakes. It is a part of the process of the structural design, in regions, where earthquakes are prevalent. Earthquake has a wave back and for this reason buildings must be secured in all of directions and must be analyzed separately against seismic loads.

To analyze a building structure under the effect of seismic loads, lots of factors such as building components, structural materials, local site conditions, purpose of usage and occupancy should be taken into account. Seismic analysis of building structures become common in the early of twentieth century when recording earthquake events initiated.

The earliest provisions for the seismic resistance were the requirements to design under a lateral force proportional to the building weight and height (applied at each floor level). This approach was adopted in the appendix of the 1927 Uniform Building Code (UBC-1927), which was used on the west coast of the USA. It later became clear that the dynamic properties of the structure is affected by the loads generated during an earthquake. In Los Angeles County Building Code of 1943 a provision to vary the distribution of seismic loads based on the number of floor levels was introduced. The concept of the response spectra was developed in the 1930s, but it wasn't until 1952 that a joint committee of the San Francisco Section of the ASCE and the Structural Engineering Association of Northern California (SEAONC) proposed using the building period (the inverse of frequency) to determine lateral forces (FEMA 356). Until now since no one can predict maximum earthquake acceleration that will affect the building for analyzing the structures, engineers use the recent earthquake records in the area or in other words use time-acceleration graph. After a few years engineers had started to develop programs for calculating the effects of earthquake in the building and the response of structural systems. The first program was SAP that is published in 1970 (Bing, 2011), an early finite element program analysis (a numerical method for solving differential equations). With improving technology and science more methods have been created to calculate impacts of earthquake on structures correctly. After 1970s seismic performance of older buildings (that were built without taking earthquake impact) has been observed to be relatively poor when compared to the performance of modern RC building and because it was not possible to demolish the entire stock, retrofitting of them for improving their seismic resistance has been initiated.

Earthquake or seismic performance defines a structure's ability to sustain its main function such as safety and serviceability after a particular earthquake exposure.

2.2 Structural Analysis Procedures

For seismic analysis, lateral forces caused by an earthquake have to be calculated which can be carried out by many methods and for different types of buildings. Generally analysis procedures are classified into methods; nonlinear and linear.

During elastic analysis, the relationship between force and the displacement is assumed to be linear, however this type of analysis is not so precise if the seismic forces are very high, causing degradation in the structural stiffness and strength. But on the other hand non-linear analysis methods are more complex than linear analysis and need more information about structure and site, so for regular and nominal importance type of structures, linear analysis is sufficient at most times.

2.2.1 Linear Methods

All displacements, stresses, reactions, etc., are directly proportional to the magnitude of the applied loads. The results of different linear analyses may be superposed. The term "Linear" in linear analysis procedures implies "linearly elastic". In linear analysis procedures, however may include geometric nonlinearity of gravity loads acting through lateral displacement and implicit material non linearity of concrete components using properties of cracked section.

Linear analysis methods are mostly preferred for the design of small and rather simple buildings subjected to low seismicity since the calculated responses can be inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquakes in a nearly elastic manner. This method can be classified into three procedures, namely, equivalent lateral load analysis, response spectrum analysis and time history analysis.

Employing one of the linear procedures, however has some limitations as provided in ASCE 41 (2006).

1- The fundamental period of the building, T, is greater than or equal to 3.5 times T_s (which is constant acceleration region of the design response transitions to the constant velocity region);

2- The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4 (excluding penthouses);

3- The building has a torsional stiffness irregularity in any story. A torsional stiffness irregularity exists in a story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that the drift along any side of the structure is more than 150% of the average story drift;

4- The building has a vertical stiffness irregularity. A vertical stiffness irregularity exists where the average drift in any story (except penthouses) is more than 150% of that of the story above or below; and

5- The building has a non-orthogonal lateral-force-resisting system (ASCE 41).

2.2.1.1 Standard Code Procedures

Standard code procedures include both linear static and dynamic analysis methods. In this methodology prescribes a formula that determines lateral force. These forces are applied to determine the adequately of the structural system. If the systems are not adequate, the design is revised and modified design is reanalyzed. This process is repeated until all the provisions of the building code are satisfied. The procedure relies on principle of statistics and the structural components.

Although this procedure is commonly called a static lateral force procedure, it includes some implicit elements of dynamics. These include the use of the fundamental period of vibration to determine the amplification of ground motion acceleration and the use of vertical distribution of force equations to approximate modal response.

Standard code procedures include all those used by model building codes and those recommended by code development bodies. The advantages of code procedures are that design professionals and building officials are familiar and comfortable with these procedures, and the simplified analyses methods often allow design costs to be minimized.

2.2.1.2 Response Spectrum Analysis

Response-spectrum analysis is a statistical type of analysis for determination of the likely response of a structure to seismic loading (Mostofy, 2006). Response-spectrum analysis seeks the likely maximum response to equation 2.1 rather than the full time history. The earthquake ground acceleration is each direction is given as a digitize response- spectrum curve of pseudo-spectral acceleration response versus period of structure.

$$K.\ u(t) + C.\ \dot{u}\ (t) + M.\ \ddot{u}\ (t) = m_x \ddot{u}_{gx}\ (t) + m_y\ \ddot{u}_{gy}\ (t) + m_z\ \ddot{u}_{gz}\ (t) \qquad \textbf{(2.1)}$$

In the above expression K is the stiffness matrix, C is the proportional damping matrix, M diagonal mass matrix, u(t) is displacement response to ground, $\dot{u}$ (t) is velocity response to ground, $\ddot{u}$ (t) is acceleration response to ground, m_x, m_y, m_z are unit acceleration loads and $\ddot{u}_{gx}$, $\ddot{u}_{gy}$ and $\ddot{u}_{gz}$ are components of uniform ground accelerations. Even though acceleration may be specified in three directions, only a single positive result is produced for each response quantity. The response quantities include displacement, force and stress. Each computed result represents a statistical measure of the likely maximum magnitude for that response quantity. The actual response can be expected to vary within a range from positive value to its negative.

Response spectra are very useful tools of earthquake engineering for analyzing the performance of structures where the equipment is the earthquake.

Thus, if one can find out the natural frequency of the structure, then the peak response of the building can be estimated by reading the value from the ground response spectrum for the appropriate frequency. Response spectrum analysis can also be used in assessing the response of linear systems with multiple modes of multi-degree of freedom systems, although they are only accurate for low levels of damping. In most building codes in seismic regions, this value is from the basis for calculating the forces that a structure must be designed to resist.

The peak acceleration is plotted against the period for the site hazard. The Response Spectra variation is the maximum response of a system degree of freedoms as a function of periods. If the model is assumed to be a single degree of freedom system and its period is known then its response can be read directly off the curve. Then this variation used to investigate the peak dynamic response of a model for different values of its period.

2.2.1.2.1 Horizontal Spectrum

The design of any engineered structure is based on an estimate of ground motion, either implicitly through the use of building codes or explicitly in the site-specific design of large or particularly critical structures. Rarely are there a sufficient number of ground motion recording near a site to allow a direct empirical estimation of the motion to develop relationships that are expressed in form of equation or graphical curves for estimating ground motion in terms of magnitude distance, site condition and other variables from the body of strong motion data in a large region or a particular tectonic setting. This is so for site specific design as well as for regional hazard mapping (Bozorgnia, 2006).

2.2.1.3 Time History Analysis

Linear time-history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. In this method we calculate linear structure dynamic response to arbitrary loading. The dynamic equilibrium equation to solve for the structural response is:

$$K\, r(t) + C\, \dot{r}(t) + M\, \ddot{r}(t) = d(t) \qquad \textbf{(2.2)}$$

Where in above equation K is the stiffness matrix, C is the proportional damping matrix, M diagonal mass matrix, r(t) is displacement response to ground, $\dot{r}(t)$ is velocity response to ground, $\ddot{r}(t)$ is acceleration response to ground and d(t) is forces that applied to structure.

In time history analysis, dynamic response of structures calculate with using short time steps. This should calculate response of structures under stimulate of the ground acceleration based on at least three mapping acceleration. If be selected less than seven mapping acceleration for analysis, we should control structure deformations and internal forces with maximize effectiveness of mapping acceleration or we can use average of mapping acceleration for control of structure if more than seven mapping acceleration be selected.

For this metho, applied loads should be defined in time steps. These time steps are called load steps that is different between modal time-history analysis that has little effect on efficiency and in direct-integration time-history analysis that may cause the stiffness matrix to be re-solved if the load step size keeps changing. This method is divided into two parts. First modal-time-history analysis that modal superposition provides a highly efficient and accurate procedure for performing time-history analysis and direct-integration time-history analysis that is direct integration of the full equations of motion without the use of modal superposition. While modal superposition is usually more accurate and efficient, direct-integration does offer the following advantages for linear problems same as :

1- Full damping that couples of the modes can be considered.

2- Impact and wave propagation problems that might be excited a large number of modes that may be more efficiently solved by direct integration.

Direct integration results are extremely sensitive to time-step size in a way that is not true for modal superposition. Therefore, during the analysis, the time-step should be decreased until its effect disappears on numerical results.

2.2.2 Non-Linear Methods

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Typical instances where nonlinear analysis is applied in structural earthquake engineering practice are to: (1) assess and design seismic retrofit solutions for existing buildings; (2) design new buildings that employ structural materials, systems, or other features that do not conform to current building code requirements; (3) assess the performance of buildings for specific owner/stakeholder requirements

In previous regulations, it was assumed that elements behaviour are elastic and has linear response but nowadays because of geometric nonlinearity effects which are caused by gravity loads acting on the deformed configuration of the structure and leading to an increase of internal forces in members and connections and material nonlinearity, structural elements behave beyond their yield levels subjected to earthquake loads.

Nonlinear analyses require thinking about inelastic behavior and limit states that depend on deformations as well as forces. They also require definition of component models that capture the force-deformation response of components and systems based on expected strength and stiffness properties and large deformations. Depending on the structural configuration, the results of nonlinear analyses can be sensitive to assumed input parameters and the types of models used.

2.2.2.1 Nonlinear Static Analysis

Analyzing a structure with material and geometric nonlinearity can be performed by non- linear static analysis. In this method, lateral load caused by the earthquake, is applied to the structure in the static form, however gradually increased till the displacement of a particular point (the target point), reach to a certain amount under lateral force or the structure come crashing down. This method gives a good approximation of the structural behavior under seismic load, and known as pushover procedure.

A predefined pattern of forces is applied to a structural model that includes non linear properties and the total force is plotted against a reference displacement to define a capacity curve. In this method to find out whether the displacement increments cause changes in the stiffness matrix of the structure during a particular load step, the corresponding rotation increments at the end of each element need to be determined. The procedure at this stage of the nonlinear analysis is the state determination of each element of the structure (Larner and Zhang, 2001).

The non-linear static analysis is a more reliable approach to characterizing the performance of a structure than linear procedures. However, it is not exact and can not be accurate at some cases depending on the stiffness loss during the dynamic response of the structure or can ignore the contribution of higher modes if the procedure does not include multi-modes.

2.2.2.2 Non-Linear Dynamic Analysis

In non-linear dynamic analysis, structural response can be calculated by taking into account the nonlinearities due to structural materials and geometrical properties. A building structure can be analyzed exatly and preciesly with non-linear dynamic analysis, but in this metho, nonlinear characteristics of the structural members should be defined and late the system should be analyzed under the effect of ground motion records.

Nonlinear dynamic analysis can be performed in two ways; First one is to carry out the nonlinear dynamic analysis considering a specific ground motions. Generally recorded or artifically generated accelerograms are taken into account as seismic loads. Then the structural responses and their time variations are calculated.

Second method of nonlinear dynamic analysis is known as Incremental Dynamic Analysis (IDA). In this method, time history is increased exponentially or in other words acceleration in the time record starts from lowest (response of structures is in elastic) to maximum acceleration which inelastic structures can be tolerated. In this method after each analysis, diagram of maximum shear force versus maximum displacement will be drawn which is similar to a push-over curve in non-linear static analysis as Figure2.1 (Baghini and Sorkhi, 2008).

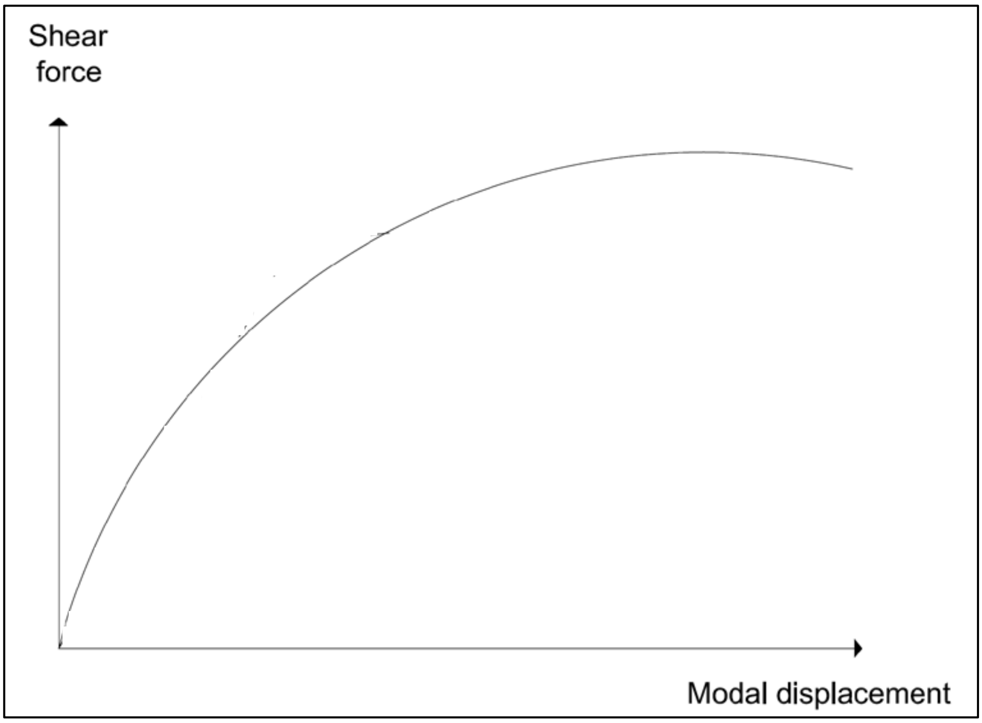


Figure 2.1: Shear force –Displacement curve.

Although nonlinear dynamic analysis is the most precise method for calculating structural response, depending on the extreme amount of outputs and the required computational time, nonlinear dynamic analysis is currently preferred for important structures or for academic studies.

2.3 Earthquake Records for Dynamic Analysis

For earthquake resistant design of critical structures, a dynamic analysis, either response spectrum or time history is frequently required.

The response of structures under earthquake ground motions can be calculated either using a (pseudo-acceleration) response spectrum or an acceleration time history.

Earthquake ground acceleration recordings are important for the following reasons:

1- As the output in modeling earthquake wave motion and explaining earthquake phenomena in the earth, they can help to understand such seismological issues as source mechanism, directivity influence, and soil dynamic non-linearity.

2- As the input to geotechnical and structural engineering systems, they can be used to compute dynamic nonlinear responses and thus to evaluate seismic performance of those systems. This analysis can quantify earthquake impact on various engineering systems and aid in the seismic-resistant design and retrofitting of structures (Larner and Zhang, 2001).

Because nonlinear dynamic analysis is an exact method so the acceleration-time curves must be precise, for this sense, nonlinear dynamic analysis needs more than one ground motion record. According to the Turkish Earthquake Code (2007) the minimum number is pronounced to be three, where the maximum of the structural response should be taken into account during the design. However, if seven or more ground motions are emplyoed, then the averages of the response can be used.

These records must be adopt in magnitude, distance to epicenter, local site conditions and sources mechanism with each other and accelerogram sites must be similar in geology, tectonic, especially in soil layer specification to land surface that building is located.

But with lack of recorded data and the randomness of earthquake ground motion that may be experienced by the structure in the future, usually it is difficult to obtain recorded data which fit the requirements (site type, epicenteral distance, etc.) well. Therefore the artificial seismic waves are widely used in seismic designs, verification of seismic capacity and seismic assessment of structures but the numerical values of the structural response might be in the conservative side when compared to the real earthquake records.

When the artificial ground motion used in this thesis and exhibited in Figure 2.2 is inspected, it can be seen that the peak ground acceleration (PGA) is 0.42g at 7.08s. However, the strong ground motion duration is significantly different that a real accelerogram. Normalized spectra of the 7 artificial ground motions used in this thesis are given with a comparison of the design spectrum provided in the Turkish Earthquake Code in Figure 2.2 for local site class Z2.

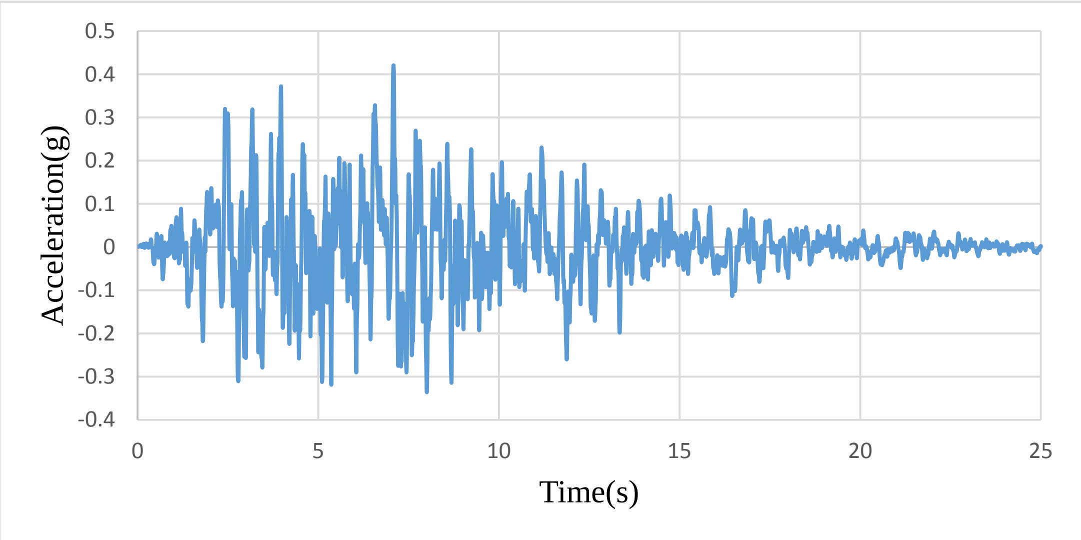


Figure 2.2: Acceleration-time variation of the first artificial ground motion.

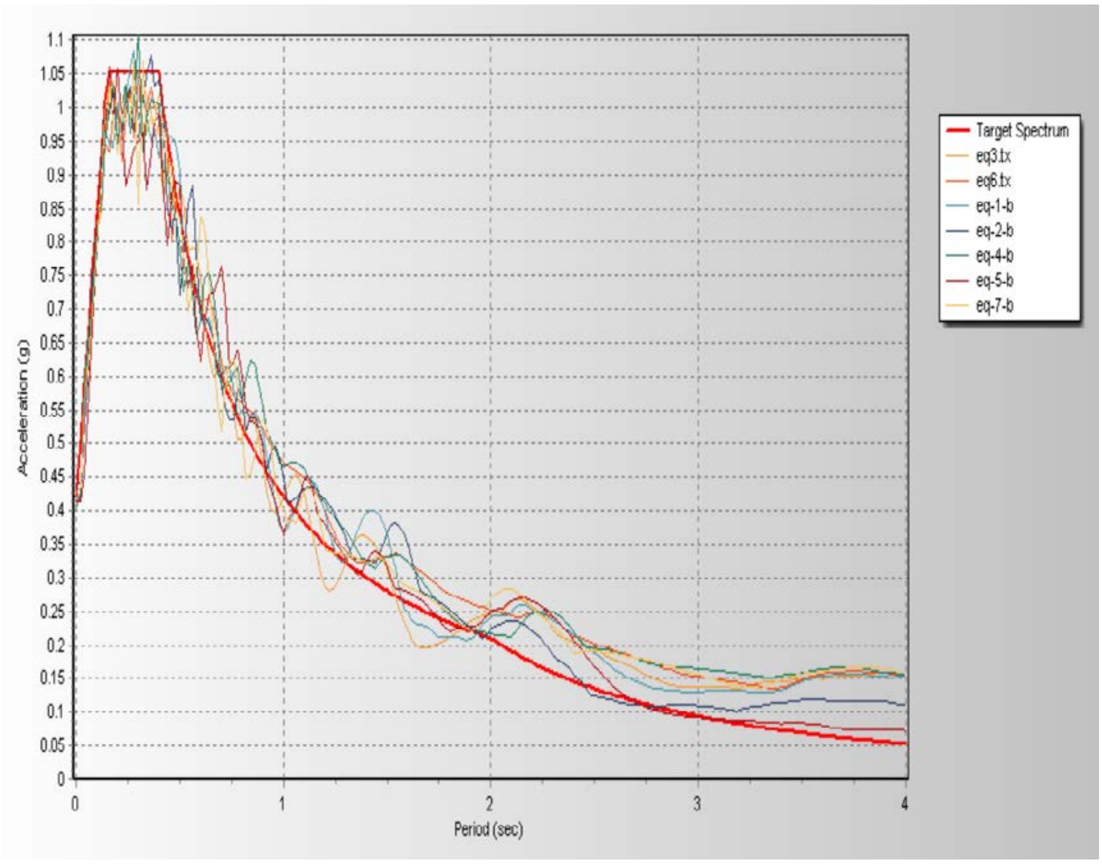


Figure 2.3: Normalized acceleration response spectra of the 7 artificial ground motions used in this thesis and their comparison with design spectrum.

For design purposes on the other hand, seismic codes provide design spectrum which is the envelope of many possible earthquakes. The major drawback of the response spectrum analysis in seismic design of structures lies in its inability to provide temporal information of the structural responses. Such information is sometimes necessary in achieving a satisfactory design.

Dynamic behaviors of inelastic structures during an earthquake are very complex non stationary processes which are expected by random characteristics of earthquake motions not only in the frequency domain but also in the time domain. Since earthquake is a random base phenomenon, it is impossible to detect clearly a possible future earthquake in specified location. The most important point is that the used records should be similar to the previous record of the specified location in shape and some other important parameters. These parameters such as PGA, duration, and frequency content are used to generate an artificial earthquake (Amiri and Asadi, 2011).

3 NONLINEAR DYNAMIC ANALYSIS OF RC BUILDING

Nonlinear dynamic analysis of reinforced concrete structures, which is a sophisticated method due to the dependence stiffness and strength losses to the deformations during a loading interval, will be discussed in this chapter.

Nonlinear dynamic response of a structure can be caused by different loading conditions such as earthquake ground motion, wind pressure, wave action, blast, machine vibration and traffic movement. But among them, earthquake motion mostly causes the unfavorable effects in the response of structures, so consequently more research on nonlinear structural behavior has been carried out in relation to earthquake problems.

Nonlinear dynamic analysis of structures is different from nonlinear static analysis in some points as inertial forces, damping, strain rate effect and oscillation. Defining of dynamic characteristics cannot be clarified solely through a dynamic test on real structures because it is:

1- Difficult to understand behavior due to complex interactions of various parameters.
2- Expensive to build a structure just for destructive testing
3- Capacity of loading devices is insufficient to cause failure (Rajasekaran, 2009).

So for defining of dynamic characteristics and validating mathematical modelling techniques on different structures, specific experiments are generally conducted in well equipped laboratories.

In nonlinear dynamic analysis, engineers must pay attention to model the damping characteristics of structures and selection of ground motions and then interpreting of conclusions.

3.1 Dynamic Equilibrium

The equation of motion for a multi-degree of freedom system is given by:

$$[M].\ddot{r}+[C].\dot{r}+[K].r = P \qquad \textbf{(3.1)}$$

Where $\ddot{r}$ is the relative acceleration, $\dot{r}$ is the relative velocity and r is relative displacement that is a function with variety of time. Matrices of [M], [C] and [K] are mass, damping and stiffness matric respectively, and force P is equal to $P = -[M].\ddot{r}_g$ where $\ddot{r}_g$ is the ground relative acceleration that for structures is equal $\ddot{r}_g=1.\ddot{a}_g$ that $\ddot{a}_g$ is ground acceleration and 1 is the unit vector (Baghini and Sorkhi, 2008).

3.1.1 Mass Matrix

In the nonlinear analysis model because lateral inertia forces and displacements constitute the dominant effect, only horizontal ground accelerations are considered. Mass is only assigned to translational horizontal degrees of freedom without rotational or vertical translational inertia. Translational horizontal degree includes the dead load and a part of live load that is expected to be present in the structure during the ground shaking. In regular buildings it is common we include at least thirty percent of the design live load in the calculation of the mass of structure.

Because axial deformations in the rigid columns are very small so vertical accelerations are ignored. If vertical displacement and accelerations be important, the mass associated with the vertical degrees of freedom, has a significant effect on the response. This be happened for a structure with intractable or rigid frames that vertical inertia can be neglected. But in a structure with flexible frames or joint connection between members, rotational masses at the joints also must be considered. In Figure 3.1 displacements of a two story building is shown.

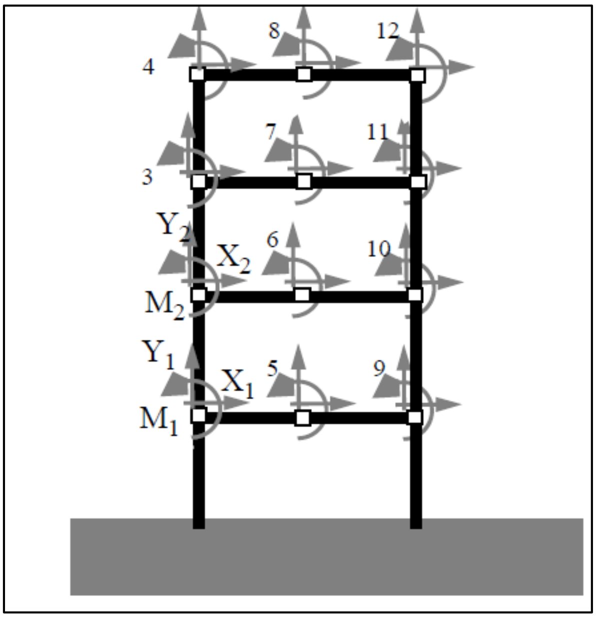


Figure 3.1: Transverse and rotational displacements of a structure.

In other words, if the axial elongations of the frame elements can be ignored in a rigid diagram structure, then it is possible to define the entire system with two translational and one rotational degrees of freedom at floor levels. In such a system mass can be calculated by the following Equations 3.2 and 3.3.

$$m_x = m_y = \frac{\rho_m \times A}{g} \tag{3.2}$$

$$m_\theta = \frac{\rho_m \times J}{g} \tag{3.3}$$

In this equation, m_x, m_y and m_θ are mass for axial displacements and rotational displacement that had been caused with inertia force, g is ground gravitation (is equal to 9.81 $\mathrm{m/sec^2}$), J is the moment of inertia of the floor area and ρ_m is the density in each floors plan of building that can be calculated with eq.3.4.

$$\rho_m = \frac{W_{eff}}{A} \tag{3.4}$$

where A is area that weight is applied on it and W_{eff} is effective weight of each floor under earthquake loads that is equal to dead load plus a percentage of live load (Otani, 1980).

Area moment inertia can be written as inertia of the layout area x-y plane (Eq.3.5).

$$J = I_x + I_y \tag{3.5}$$

3.1.2 Damping Matrix

During motions, all structures lose some amount of energy that this is referred to three main sources in numerical analysis which are nonlinearity of members, energy radiation and inherent damping. Energy dissipated in one cycle of oscillation to the maximum amount of energy accumulated in the structures is defined as damping capacity. The most effective means of deriving a suitable damping matrix is to assume appropriate values of modal damping ratios for all significant modes of vibration of the structure and then compute a damping matrix based on these damping ratios. Damping ratios can be calculated with Rayleigh method. Rayleigh damping known as proportional damping model illustrates damping as a linear combination of mass and stiffness matrix. For linear behavior it can be calculated with equation 3.6 that in this equation α and β are scalars derived by assuming suitable damping ratios for two modes of vibration having the units 1/sec and sec unit respectively.

$$C=\alpha M+\beta K \tag{3.6}$$

But for nonlinear behavior, damping ratios for **n**th mode must be calculated with equation 3.7.

$$\zeta_n = \frac{\alpha}{2} \times \frac{1}{\omega_n} + \frac{\beta}{2} \times \omega_n \tag{3.7}$$

Where in this equation ω_n is the circular frequency of the **n**th mode and α and β can be calculated with specified damping ratios ζ_i and ζ_j for **i**th and **j**th modes, as given in Equation 3.8

$$\frac{1}{2} \times \begin{bmatrix} 1/\omega_i & \omega_i \\ 1/\omega_j & \omega_j \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \begin{Bmatrix} \zeta_i \\ \zeta_j \end{Bmatrix} \tag{3.8}$$

Conversely, stiffness is proportional to the frequency of the structure and loss of contribution of higher modes to the structural response can be result in higher damping.

In nonlinear dynamic analysis the damping matrix can be expressed in proportion to the initial or current tangent stiffness of the structure according to

$$[C]=\alpha.[M]+\beta.[K_0] \tag{3.9}$$

$$[C]=\alpha.[M]+\beta.[K_T] \quad \textbf{(3.10)}$$

That $[K_0]$ is the initial stiffness matrix and $[K_T]$ is the tangent stiffness matrix. In most cases equation 3.9 is used because second option does not have any advantages to first option and just can lead to numerical problems.

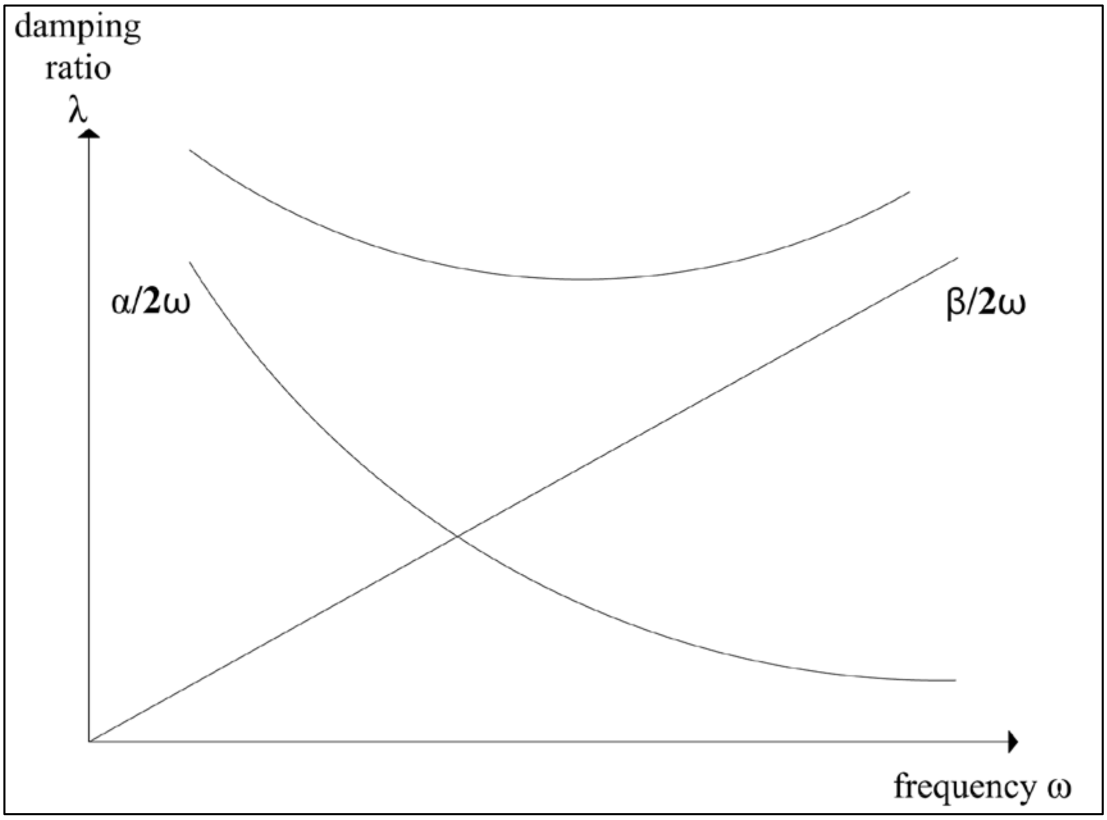


Figure 3.2: Damping ratio related to frequency (Otani, 1980).

3.1.3 Stiffness Properties of RC Members

Building structures must resist forces effecting on them. Deformations in every structural element must be controlled by means of allowable displacement; henceforth stiffness and strength factors must be considered. First is the strength and second is the member stiffness. Stiffness of each element depends on shape of elements, member properties and member materials. In other words strength of each element against displacements is stiffness.

For calculating the effective stiffness of RC columns there are two methods. First, initial stiffness are calculated by using secant of the shear forces versus lateral displacement relationship passing through the point at which the applied force reaches 75% of the flexural strength (point A′ in Figure 3.3). Second method is the column is loaded until either the first yield occurs in the longitudinal reinforcement at the maximum compressive strain of concrete reaches 0.002 at a critical section of the column (point A in Figure 3.3) (Chen and Abers, 2003).

But result of both method are very similar to each other.

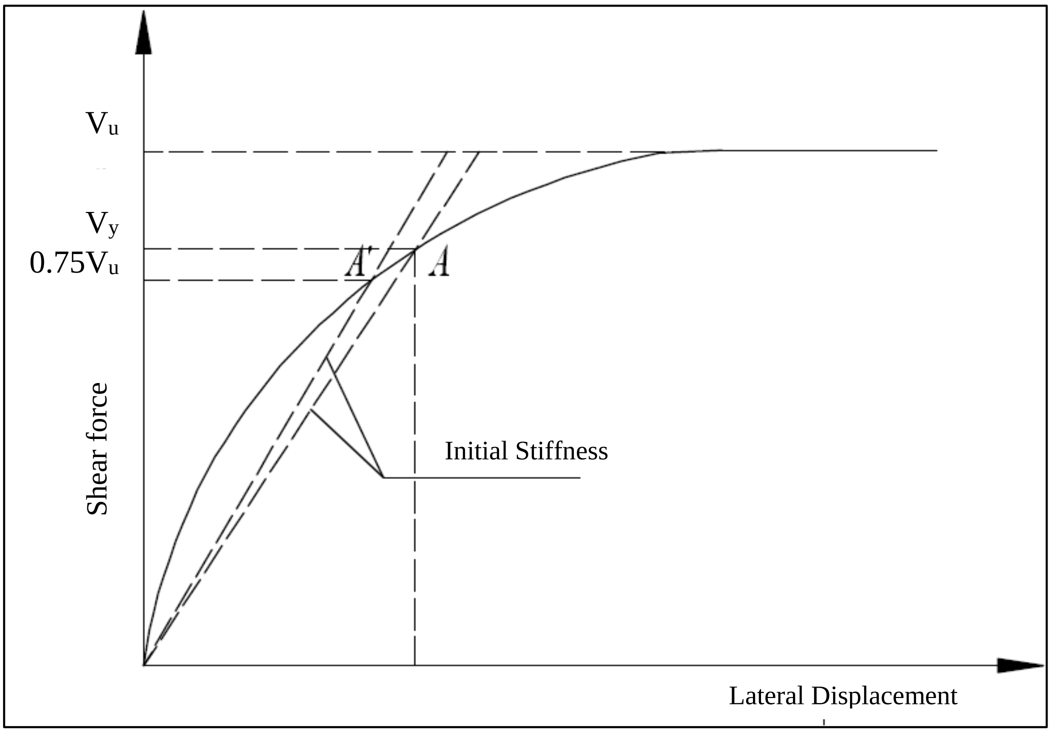


Figure 3.3: Method to determine initial stiffness.

If it is assumed that rotation of columns are fixed in both ends and there is linear behavior throughout the height; then the moment of inertia can be calculated by equation 3.11.

$$I_e = \frac{L^3 \times K_e}{12E_c} \quad \textbf{(3.11)}$$

In this equation, K_e is the initial stiffness, L is the length of element and E_c is the elasticity modulus of concrete. From initial inertia, stiffness ratio can be calculated by equation 3.12.

$$\kappa = \frac{I_e}{I_g} \times 100\% \quad \textbf{(3.12)}$$

Deformations of each element is either one or the combination of the axial deformation, flexural deformation, shear deformation and torsional deformation. After fraction of a concrete beam or column, stiffness value will be decreased and there are two options.

First is to calculate again from first and second is to decrease stiffness value with coefficient. For example in Iranian code after fraction of a concrete column, 20-70% stiffness value must be considered that is different since shape and materials is different.

3.2 RC Member Models

In reinforced concrete frame structures, flexural rotation, shear deformation containing of shear sliding and bond slip are major source of deformation. For the nonlinear dynamic analysis of a building structure, it is necessary to model the stiffness distribution along a RC member and model the members considering an adequate force deformation relationship under stress reversals in time. For this, hysteric models can be employed. Analyzing hysteric behavior of RC members must be based on description of all deformation sources and also interactions of different mechanism.

In RC members these deformations are not created just in critical locations, but rather spread throughout the RC members (Figure 3.4).

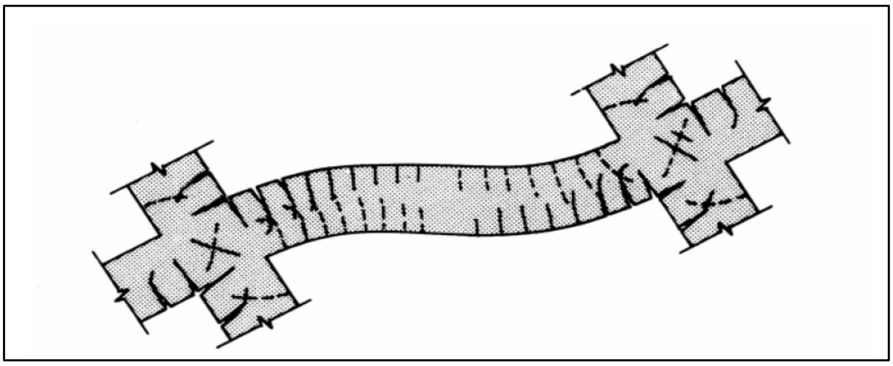


Figure 3.4: Deformation of RC beam under earthquake load (Mostofy, 2006).

3.2.1 Flexural Beam-Columns

Inelastic modeling of moment frame systems primarily involves in component models for flexural members (beams and columns) and their connections. For systems that employ “special” moment frame capacity design principles, the inelastic deformations should primarily occur in flexural hinges in the beams and the column bases. The NEHRP Seismic Design Technical Briefs by Moehle et al, (2008) and Hamburger et al. (2009) provide a summary of design concepts, criteria, and expected behavior for special concrete and steel moment frames. It is important to recognize, however, that the minimum design provisions of ASCE 7 and ACI 318 (2008) standards for special moment frames do not always prevent hinging in the columns or inelastic panel zone deformations in beam-to-column joints. Therefore, the nonlinear model should include these effects, unless the actual demand-capacity ratios are small enough to prevent them. In frames that do not meet special moment frame requirements of ASCE 7, inelastic effects may occur in other locations. The inelastic response of flexural beams and columns is often linked to the response of the connections and joint panels between them. The inelastic behavior in the beams, columns, connections, and panel zone (Figure 3.5) can each be modeled through the idealized springs along with appropriate consideration of finite panel size and how its deformations affect the connected members. On the other hand, in concrete frames, the inelastic deformations in the beams and columns are coupled with the panel zone behavior, due to the bond slip of longitudinal beam and column bars in the joint region. Thus, for concrete frames, the flexural hinge parameters should consider how the deformations due to bond slip are accounted for – either in the beam and column hinges or the joint panel spring.

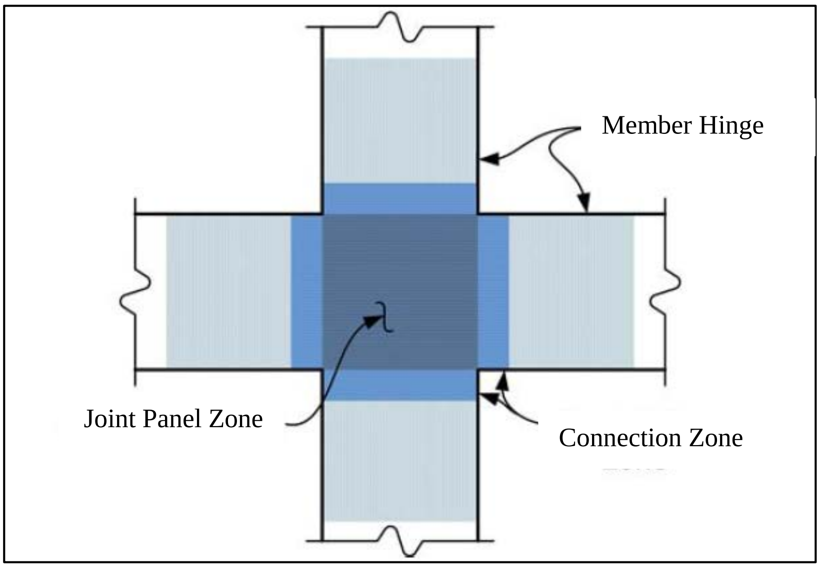


Figure 3.5: Hinging region of beams and columns panel zone (Deierlein, 2010).

3.2.2 Concrete Moment Frames

Concrete frames that meet seismic design and detailing requirements and qualify as special moment frames are somewhat more difficult to model than steel frames. Stiffness of members is sensitive to concrete cracking, the joints are affected by concrete cracking and bond slip, and the post-yield response of columns and joint panels is sensitive to axial load. ASCE 41-06 and PEER/ATC 72-1 provide models and guidance for characterizing member stiffness, inelastic member hinge properties, and strategies for joint modeling. Lowes and Altoontash (2003) and Ghobarah and Biddah (1999) provide further details on modeling concrete beam-column joints. Frames that do not conform to the special seismic detailing but have behavior that is dominated by flexural hinging can also be modeled, provided that the hinge properties and acceptance criteria are adjusted to account for their limited ductility. Frames with members that are susceptible to sudden shear failures or splice failures are more challenging to model. In such cases, nonlinear flexural models can be used to track response only up to the point where imposed shear force and/or splice force equals their respective strengths. Otherwise, to simulate further response, the analysis would need to capture the sudden degradation due to shear and splice failures.

3.2.2 Infill Walls

The infill wall system is conventionally modeled as a frame structure with beams and columns braced with one or two diagonal struts representing the masonry infill (Figure 3.6). In most cases, the infill panel failure will initiate in sliding along the horizontal joints, where the capacity is limited by the shear strength of the mortar. Alternatively, if the panel is strong in shear, the diagonal strut will crush near the frame joint and lose strength. This mode of failure has limited deformation capacity because the crushing will be abrupt. The large panel forces generated in this mode will be distributed along the beam and column members, and they may result in either column or beam shear failures. Depending on their height-to-thickness slenderness, unreinforced infill walls can also fail out-of-plane, sometimes in combination with one of the other modes.

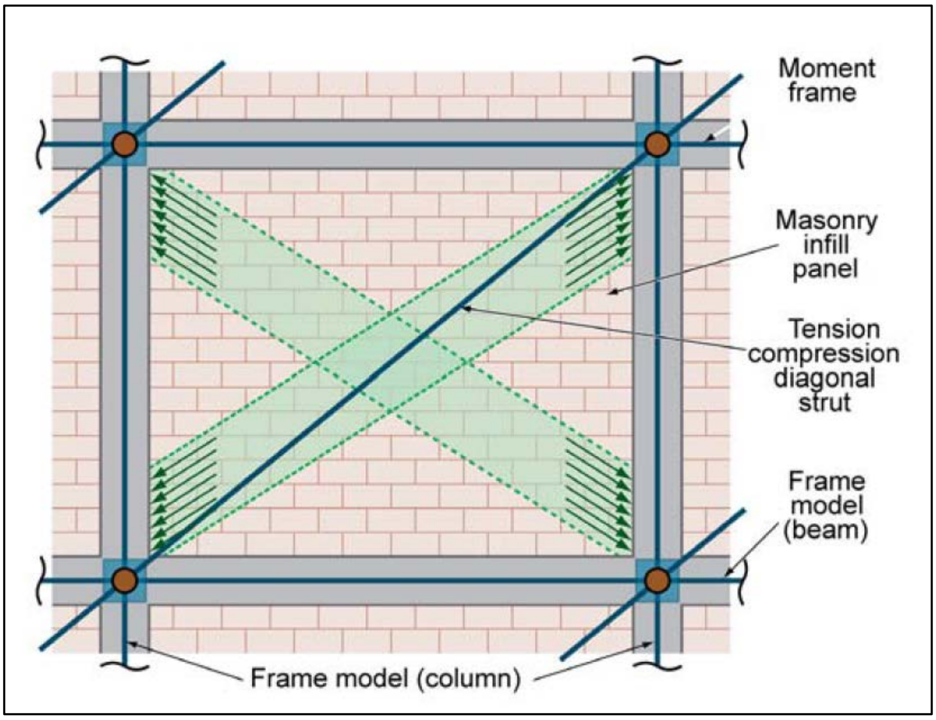


Figure 3.6: infill wall panel and model (Deierlein, 2010).

For either the shear sliding or compression crushing mode, it is reasonable to analyze frames with infill walls using a single equivalent strut or two diagonal compression struts for reversed cyclic loading analysis. Such an approach is adopted by FEMA 306 (FEMA 2009b) and ASCE 41-06. Detailed finite element analysis can also be used if such a refinement is required, but it presents challenges both in terms of computational demands and constitutive modeling of the infill and boundary interface. For software that supports use of a single axial spring, the spring strength should be based on the governing failure mode (sliding or compression failure). Madan et al. (1997) provides an example of a more advanced series spring strut model, which captures the combined behavior of diagonal sliding shear and compression failure, including cyclic deterioration. Kadysiewski and Mosalam (2009) provides a modeling approach that considers both in-plane and out-of-plane failure.

Modeling of the equivalent diagonal strut requires knowledge of the stiffness and cyclic strength behavior. The equivalent strut is represented by the actual infill thickness that is in contact with the frame, the diagonal length, and an equivalent width which may be calculated using the recommendations given in the concrete and masonry chapters of ASCE 41-06. Only the masonry wythes in full contact with the frame elements should be considered when computing in-plane stiffness, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

3.2.3 Shear Walls

Reinforced concrete shear walls are commonly employed in seismic lateral-force-resisting systems for buildings. They may take the form of isolated planar walls, flanged walls (often C-, I- or T-shaped in plan) and larger three dimensional assemblies such as building cores. Nearby walls are often connected by coupling beams for greater structural efficiency where large openings for doorways are required. The seismic behavior of shear walls is often distinguished between slender (ductile flexure governed) or squat (shear governed) according to the governing mode of yielding and failure. In general, it is desirable to achieve ductile flexural behavior, but this is not possible in circumstances such as (1) short walls with high shear-to-flexure ratios that are susceptible to shear failures, (2) bearing walls with high axial stress and/or inadequate confinement that are susceptible to compression failures, and (3) in existing buildings without seismic design and detailing qualifying the wall system as special as defined in ASCE 7.

Cyclic and shake table tests on reinforced concrete shear walls reveal a number of potential failure modes that simple models cannot represent explicitly, however these failure modes are generally reflected in the backbone curves, hysteresis rules, and performance criteria adopted in lumped plasticity models. These failure modes include (1) rebar bond failure and lap splice slip, (2) concrete spalling, rebar buckling, and loss of confinement, (3) rebar fracture on straightening of buckle, and (4) combined shear and compression failure at wall toe. Some of these failure modes can be captured, either explicitly or implicitly, in fiber and finite element type modeling approaches.

3.4 Moment-Curvature

The actual concrete material behavior is nonlinear and can be described by idealized stress-strain models. For taking the response accurately, reinforced concrete members must be calculated considering the nonlinear behavior under deformation.

After modeling of reinforced concrete members, one of the main works in nonlinear analysis is extraction of moment-curvature relation for member sections. This curve is used precisely to determine the load-deformation behavior of a section by nonlinear stress-strain relationships (Figure 3.7).

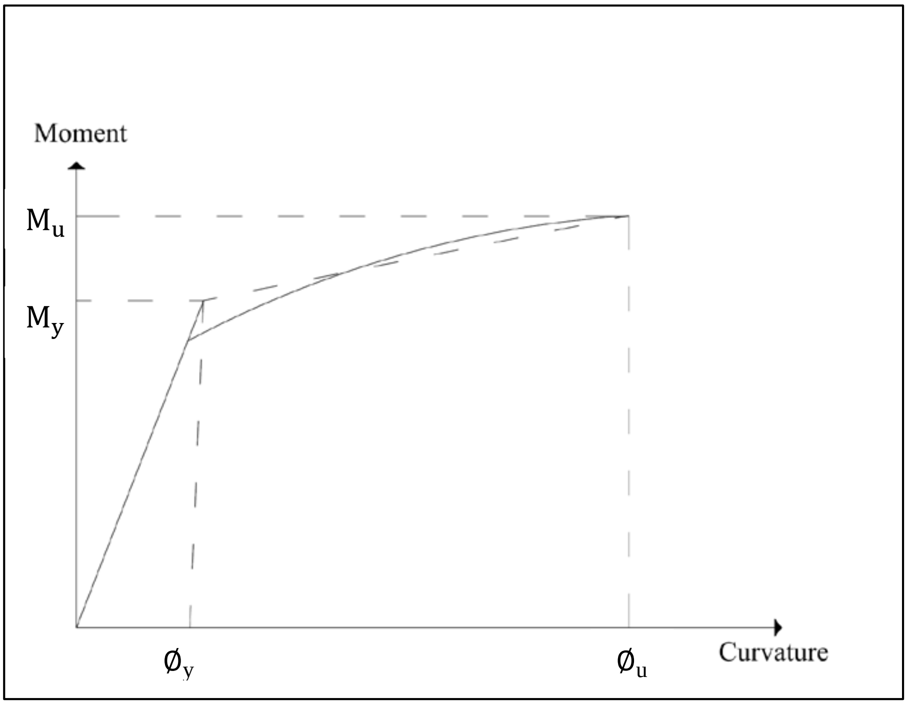


Figure 3.7: Moment Curvature curve (Mostofy, 2006).

As it is explained above moment curvature relation is defined for the specific locations within the structural element, where plastic behavior can occur. These elements act like joints after yielding and have been called plastic joints. Length of this plastic joints are almost the same as the elements depth.

3.5 DRAIN-2DX program

One of the best and computationally fast programs for performing nonlinear dynamic analysis is DRAIN-2DX (Peer, 2011). It is a general purpose computer program for static and dynamic analysis of plane structure. It performs nonlinear static and dynamic analysis and for dynamic analysis it has the power of considering ground accelerations, ground displacements, imposed dynamic loads and specified initial velocities. Static and dynamic loads can be applied in any sequence. For example, a dynamic analysis can be performed to damage the structural and static loads can then be applied to investigate its behavior in the damage state.

In this program the structural state can be saved at the end of any analysis and the analysis can be restarted from any saved state. Mode shapes and periods can be calculated for any state. Static nonlinear analysis is performed by an event to event scheme, where each corresponds to a significant change in stiffness.

The element library contains 1) inelastic truss bar, 2) simple inelastic beam column, 4) simple inelastic connection, which allows for translational as well as rotational force transfer, 6) elastic panel element, which allow vertical, horizontal extensional and flexural stiffness to be input, 7) beam-columns elements for defining beams and columns, 9) inelastic link element, that can act in compression or tension with initial gap or axial force, and 15) fiber beam-column element for steel and reinforced concrete members.

The elements include capabilities for event and internal energy calculations. For analyzing a frame system by this program it is necessary to calculate and input some data, then write this requirements on notepad then run the program for obtaining the results. In nonlinear dynamic analysis, it is necessary to control the building in two directions, therefore two perpendicular directions of the building, namely x-x and y-y directions, should be modelled. For connecting the frames in each segment to each other, a link element which is capable of transferring lateral forces with length of one meter is used and it can be used for connection of two different section or even for connection between beams and columns that are not connect to each other.

3.5.1 Program input

For running Drain-2dX program it is necessary to input data including the structural properties that involves in structural materials, geometric properties, earthquake record and nonlinear characteristics of RC sections.

3.5.1.1 Noodcoords

In this section nodal coordinates are given numerically for the planer system.

3.5.1.2 Restraints

In this section, modal restraints are defined, where 0 indicates that the node is free for the related degree of freedom, and 1 indicates that it is fixed.

3.5.1.3 Slaving

Depending on the rigid diagram behavior, a master node within the frame is selected and the other nodes in the same elevation are slaved to this node.

3.5.1.4 Masses

In this section, nodal masses are numerically defined for each node.

3.5.1.5 Elements group

For input and output, the elements must be divided into groups. All elements in any group must be of the same type and typically all elements of any type will be included in a single group.

Each element type in the element library is identified by a type number. Generally there are seven different type of elements but in most case just two or three of them can be used and each element type request independent criteria to calculate.

In this section element characteristics such as stiffness, area, moment of inertia have been entered for each element.

Each element has a constant viscous damping matrix equal to βK_β, where K_β is the damper stiffness matrix of the element. The value of β is same for all elements in any group, but can be vary from group to group.

In this thesis, element type-7 is used to define the nonlinear behavior of beams, columns and walls; while type-9 element is used for non-structural walls and rigid link elements.

Element type 7

Element 7 uses discrete plastic hinges at the ends of the element to represent the behavior of ductile reinforced concrete beams and walls.

The element geometry is shown in Figure 3.8. The element consists of an elastic beam connected in series to rigid-plastic hinges located at either end of the beam. Rigid end zones may be specified, and elastic shear deformation can be included by specifying an effective shear area. Although the element computations are based on rigid-plastic hinges, the input data are based on the moment-curvature response of the sections at either end of the beam. The flexural stiffnesses at each end of the beam, which may differ for positive and negative moment, are assumed to extend to midspan, excluding rigid end offset, if present. The specification of a plastic hinge length allows the properties of the element components to be determined.

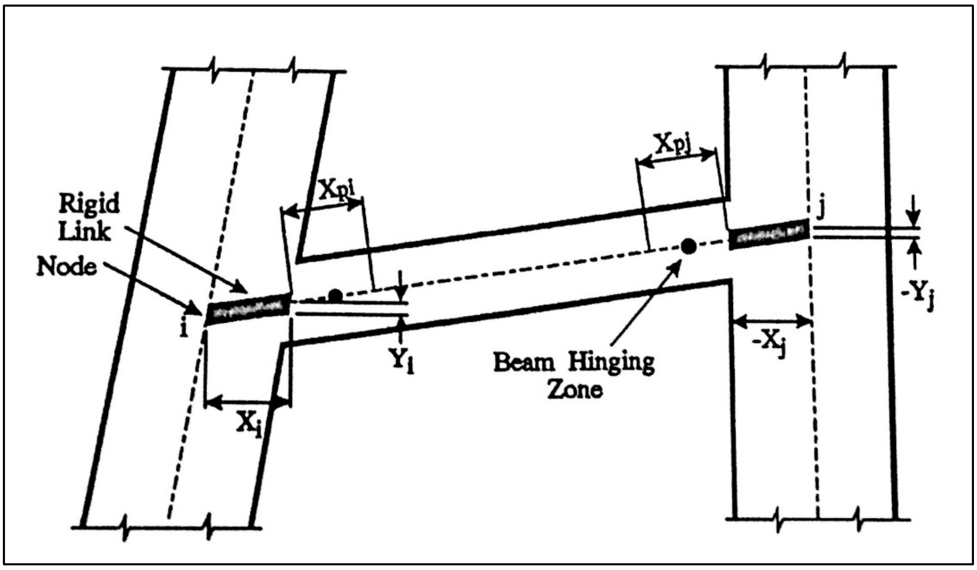


Figure 3.8: Definition of rigid end zones and plastic hinge lengths (PEER, 2011).

Yielding takes place only at the plastic hinges. Each plastic hinge may have a different elastic stiffness and yield strength for positive and negative bending. Uncracked behavior is not modeled rather the member ends are assumed to be pre-cracked.

Element type 9

The element has finite length and an arbitrary orientation. It resists axial force only and can be specified to act in tension (tension force and extension are positive) or in compression (compression force and shortening are positive). A tension element has finite stiffness in tension and goes slack in compression. A compression element has finite stiffness in compression and a gap opens in tension.

3.5.1.6 Section

In this section, if story-wise response are concerned, then the cut locations are defined.

3.5.1.7 Gendisp

Gendisp or general displacement definition is defined by displacement direction for each column. A generalized displacement will typically define a relative displacement between two nodes, or a deformation based on the relative displacement of several nodes.

3.5.1.8 Result

In this section, which response are to be saved are defined.

3.5.1.9 Accnrec

In this section earthquake ground motion and its parameters are defined.

3.5.1.10 Parameters

In this section, parameters regarding the dynamic analysis such as time step are defined.

3.6 Analyzing steps

1- Preparing of the INPUT file;

2- Running the DRAIN-2DX program subjected to artificial different ground motion records;

3- Controlling of top story displacement and base shear force under the effect of each ground motion and comparing them with the 1975 and 2007 Turkish Earthquake Codes;

4- Controlling the permitted story drifts by employing ASCE 41, since these limits are not defined in the Turkish Earthquake Code for evaluating the structural performance of an existing building by nonlinear dynamic analysis;

5- Calculation of concrete and reinforcement strains subjected to selected record (nearest record to average values);

6- Comparing concrete and reinforcement strains with 2007 Turkish Earthquake Code and identifying damage state of each element.

4 KASIMPASA BUILDING

Considering the analysis methods explained in the previous chapters, an RC existing building is analyzed in this chapter by utilizing DRAIN-2DX computer program. During non-linear dynamic analysis, 7 artificial ground motion records are employed.

The main target of this thesis is to evaluate the structural performance of the building considering the regulation of current Turkish Earthquake Code-Chapter 7.

4.1 Introduction

The RC building to be investigated herein is an existing structure constructed in Beyoglu district of Istanbul. It is still serving as a workshop building occupied by approximately 50 people working in daytime.

The building has reinforced concrete shear wall-frame system with trapezoid settlement (base) high edges of which is 20.00^{m} and parallel edges are 28.0^{m} and 27.0^{m} in length in the plan. Base area of the building is approximately ≈550 m^2 and floor areas are increased approximately to ≈603 m^2 thereby establishing cantilevers with 1.50 meter space on rear facade from ground floor. First floor being used as a workshop has the height of 4.7m and the upper floors are 2.6m.

Accordingly investigations conducted in this building have two reasons: Primarily to examine the existing damage on July 27, 2010, and the implementation status of compliance with the architectural and structural design and secondly determine the quality of the existing building materials which was conducted on November 2, 2010.

From the project drawings it is found out that long beams in interior spans have cross-section dimensions of 40/60 cm/cm; short beams to be 40/40 cm/cm; the beams on facade and rear front to be 20/50 cm/cm and flank front to be 25/70 cm/cm in size. A similar plan is applied in all storeys that reinforced with Φ14 as longitudinal reinforcement.

Reinforced concrete columns in the building are also built in sizes of 40×40 (cm×cm) in each storey and longitudinal reinforcements used in the columns and shear walls are

8Φ16 in diameter for all of the columns and transverse reinforcement Φ8 are organized through a schematic drawing and stirrup tightening area for both columns and beams. Shear walls with 20x150 (cm x cm) cross-sectional dimensions are established on facade and both flank fronts of the construction. The cross-section dimension of wall on the facade is 25×80 (cm×cm) and, such partition has the features of a column as per present regulations and they are considered and modelled as columns.

Location and the aerial view of the building can be seen in Figure 4.1. From the geology map provided in the left hand side of this figure, the local site is found out to be Z2 class with spectrum characteristic periods of T_A=0.15s and T_B=0.40s.

Figure 4.1: Location of KASIMPASA building.

Based on the previously conducted test results to figure out the existing structural material strength, the compressive strength of concrete is given in Table 4.1 story wise. On the other hand, general properties of the building are tabulated in Table 4.2

Table 4.1: Compressive strength of concrete in story.

Floor number	Concrete strength (MPa)
Ground floor	11
First floor	5
Second floor	12
Third floor	11

Table 4.2: KASIMPASA building information.

Number stories	Ground floor + three normal floor
Structural system	RC frame-wall
Floor height	4.7m (first floor) and 2.6m (upper floors)
Total height	12.5m
Building location	Beyoglu district - Istanbul
Floor area	603 m^2
Building project registration date	22 February 1979
Purpose of usage	Workshop
Reinforcing steel	S220
Earthquake zone	On the border of 1st and 2nd degrees
Effective ground motion acceleration coefficient (A_0)	Taken as 0.40 to be in the safe side
Building important factor (I)	1
Soil Class	Z2
Characteristic periods	T_A=0.15s - T_B=0.40s
Structural behavior factor for nominal ductility	R=4 for calculating the structural demands R=1 for performance analysis

But it is essential as per either the earthquake regulation, dated 2007 or TS-500, dated 2000, that the lowest concrete quality to be used in the components of structure system be in C20 class, indicating a characteristic compressive strength of f_{ck}=20 MPa.

4.2 Performance Evaluation by Nonlinear Dynamic Analysis

Since nonlinear dynamic analysis will be performed by DRAIN-2DX, planer models considering each perpendicular direction of the building are established for the building. Original layout plan of building is shown in Figure 4-2 and Figures 4-3 and 4.4 show the elevation of the frames along x-x and y-y directions respectively which are required as input data for the program. The planer structural system along the x-x direction consists of 81 beams, 52 columns, 56 shear walls and 138 nodes, while the same structural system is defined by 72 beams, 68 columns, 58 shear walls and 160 nodes along the y-y axis. As one defined, since some of shear-walls on the sides are in their weaker direction, they are modelled as columns. As it can be seen, there are 32 columns in the ground floor. Each frame is connected to each other with horizontal stiff link elements.

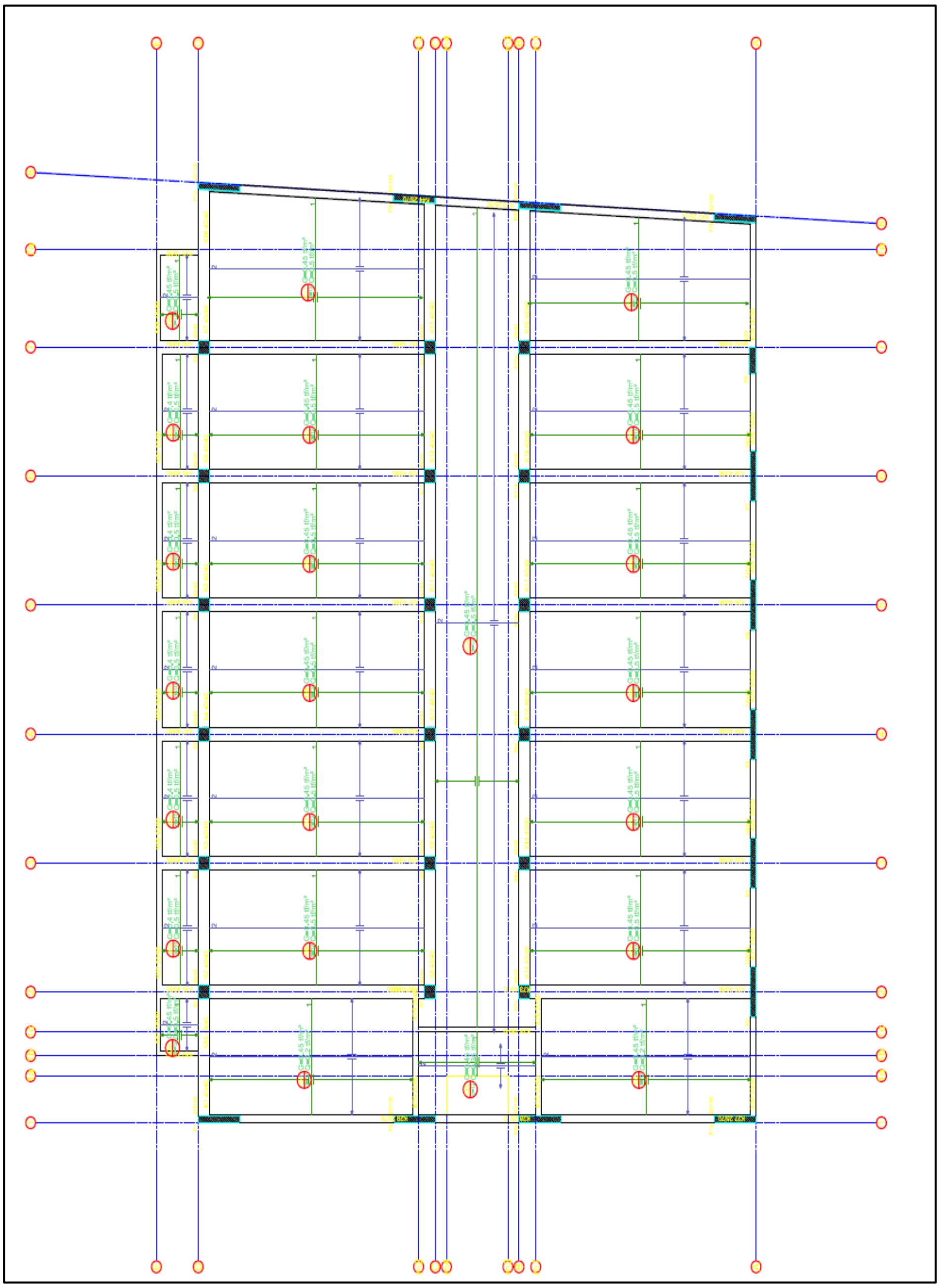


Figure 4.2: Structural layout plane of the building.

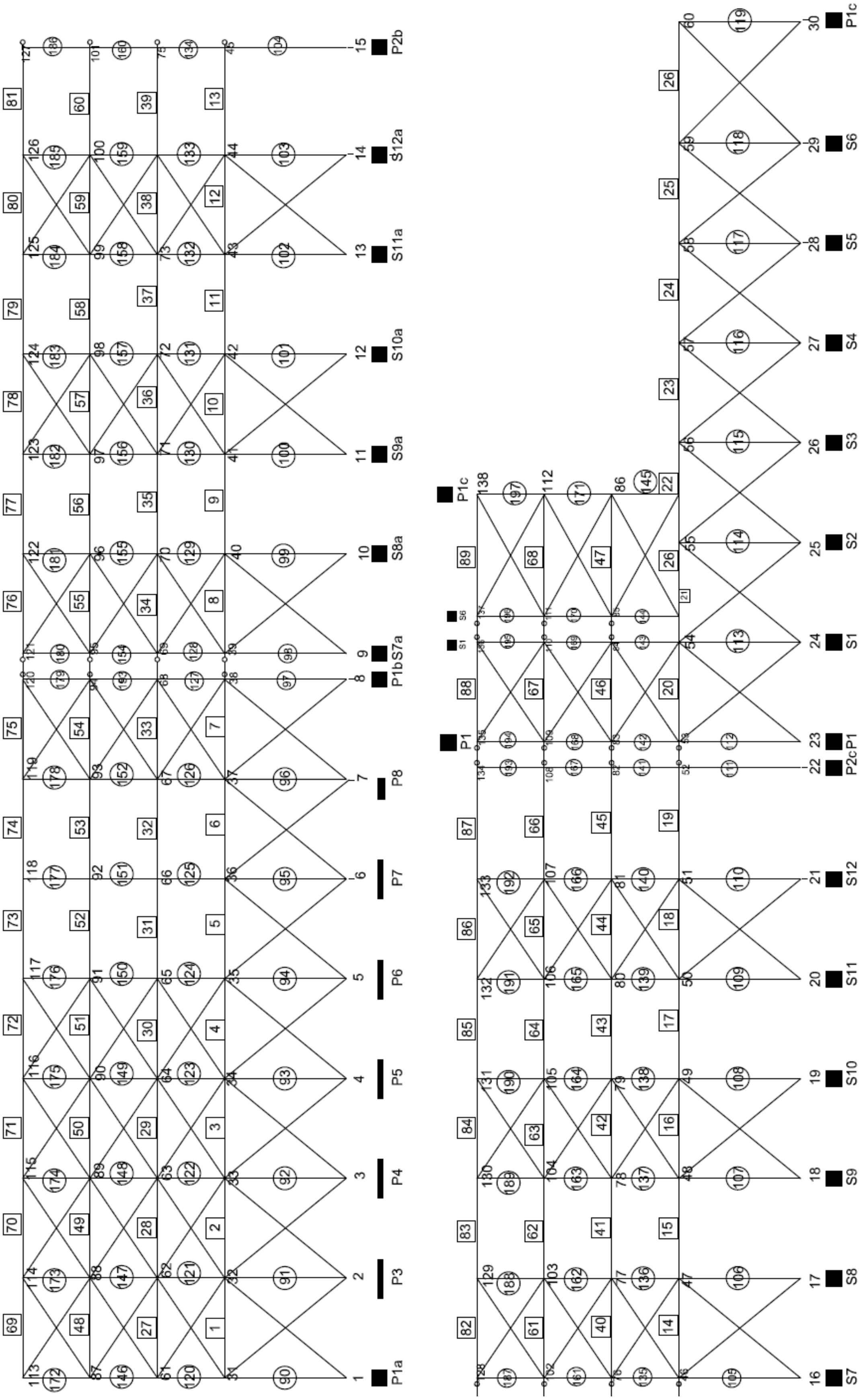


Figure 4.3: Frames in x-x direction.

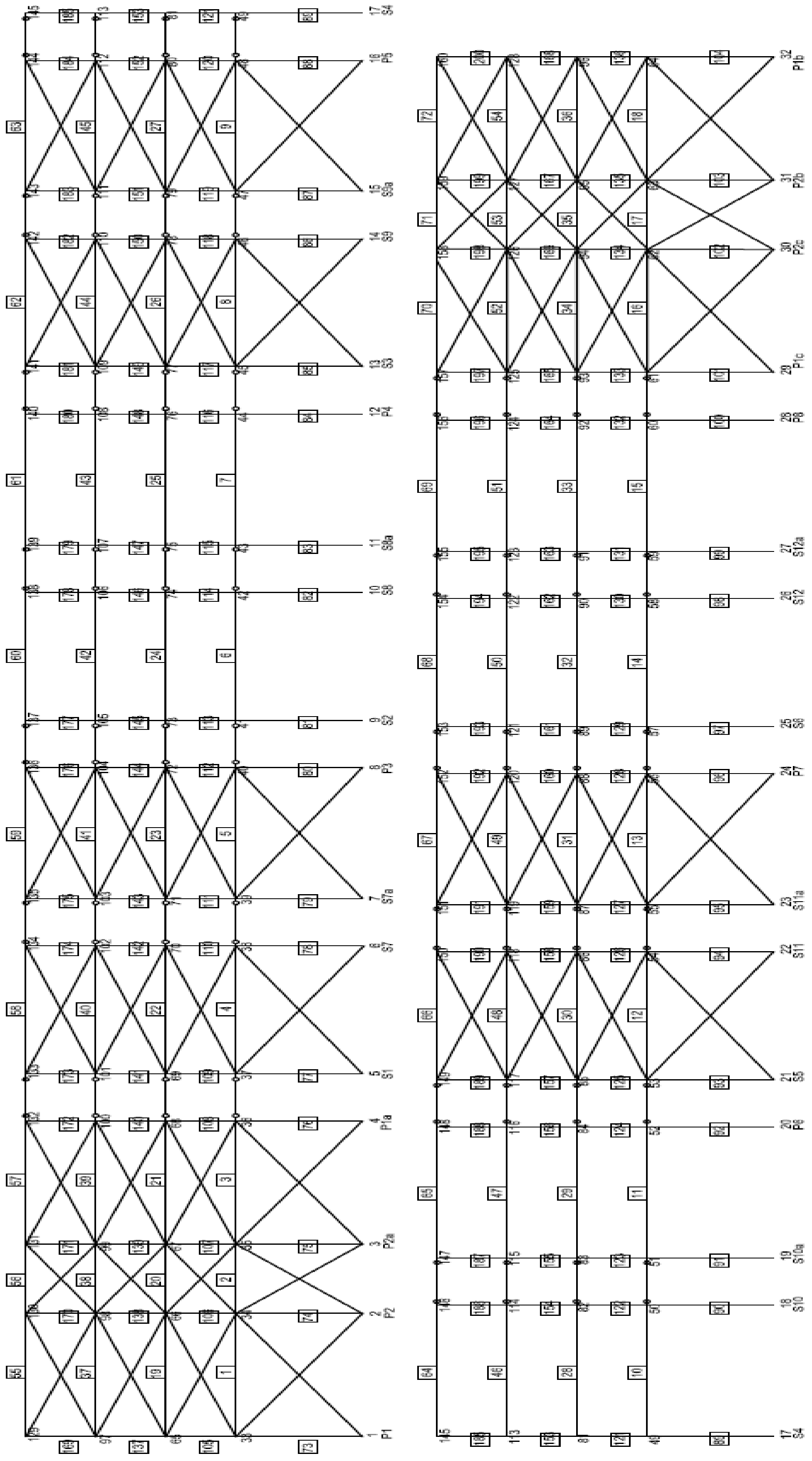


Figure 4.4: Frames in y-y direction.

Calculation of mass in y-y direction for each floor depend on slab type and area of each floor that it has been shown in Table 4.3.

Table 4.3: Nodal Masses for y-y direction.

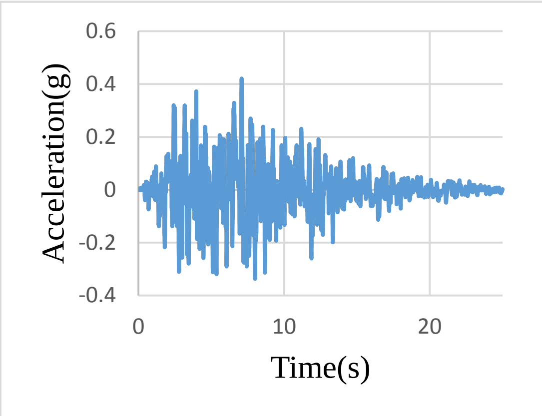
Floors	Mass Coefficient (KN/m^2)	Area of Floors (m^2)	Node numbers	Mass at each Node
1	13	603	32	244.97
2	13	603	32	244.97
3	13	603	32	244.97
4	11.5	603	32	216.70

4.3 Characteristic Properties of the Employed Ground Motions.

During the nonlinear analysis of KASIMPASA building, 7 artificial ground motions are used for which acceleration-time variations which are shown in Figure 4.5 till 4.11. Total, significant, bracketed duration and Housner intensity of motion records are given below in Table 4.4 respectively.

Table 4.4: 7 Artificial ground motion durations and Housner intensities.

Motion records	Uniform duration (s)	Significant duration (s)	Bracketed Duration (s)	Housner intensity (m)
1	14.84	11.265	22.845	1613.8
2	14.625	11.795	23.605	1593.1
3	15.695	12.015	23.335	1561.1
4	15.52	11.905	23.165	1662.8
5	15.325	11.82	22.93	1615.6
6	15.505	11.69	23.58	1689.1
7	14.61	11.95	22.395	1624.5
Average	15.16	11.78	23.12	1622.8



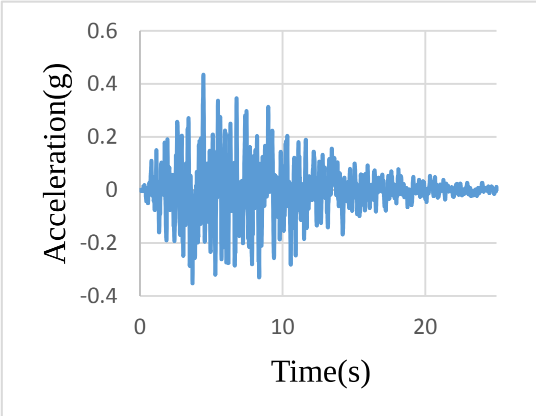
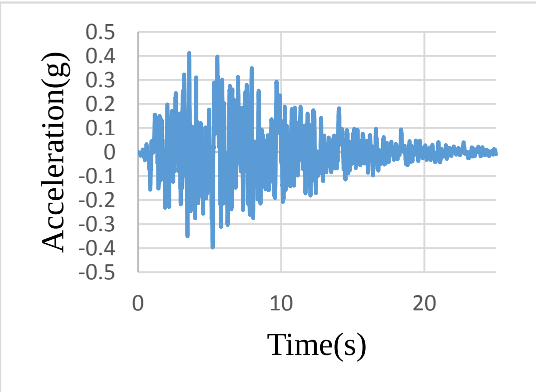


Figure 4.5: 1st earthquake (PGA=0.42g at 7.085s) **Figure 4.6:** 2nd earthquake (PGA=0.435g at 4.4s)



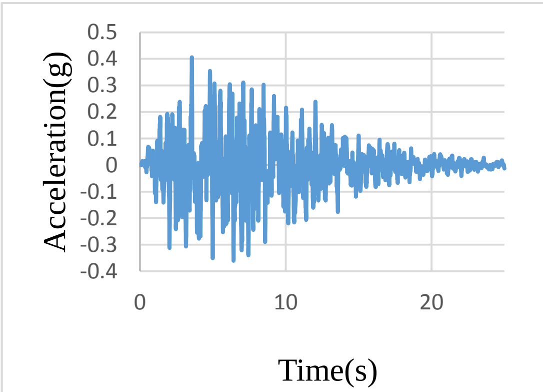
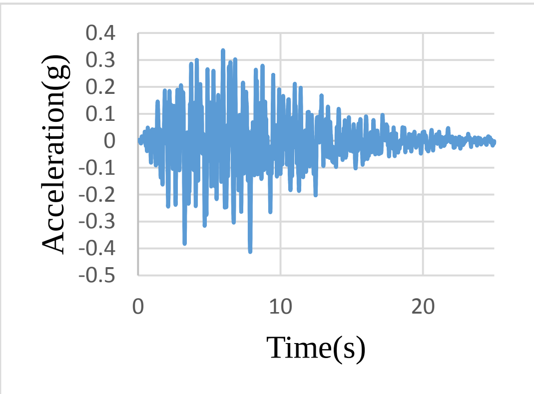


Figure 4.7: 3rd earthquake (PGA=0.413g at 3.57s) **Figure 4.8:** 4th earthquake (PGA=0.406g at 3.56s)



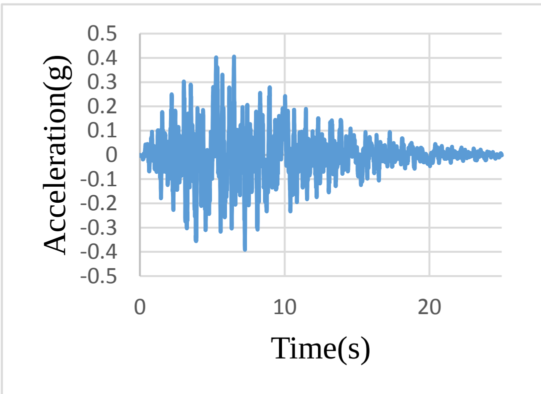


Figure 4.9: 5th earthquake (PGA=0.41g at 7.87s) **Figure 4.10:** 6th earthquake (PGA=0.405g at 6.5s)

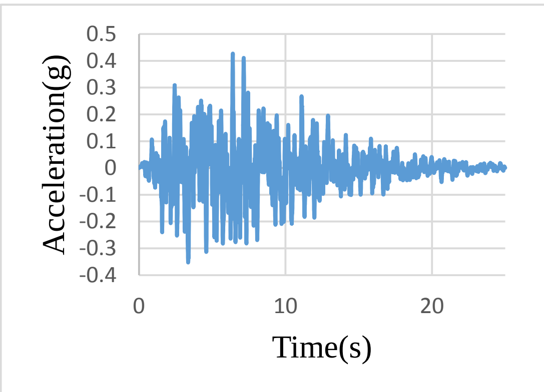


Figure 4.11:7th earthquake (PGA=0.43g at 6.4s)

Spectral response of seven different artificial ground motions compared to target spectrum are illustrated in Figure 4.12.

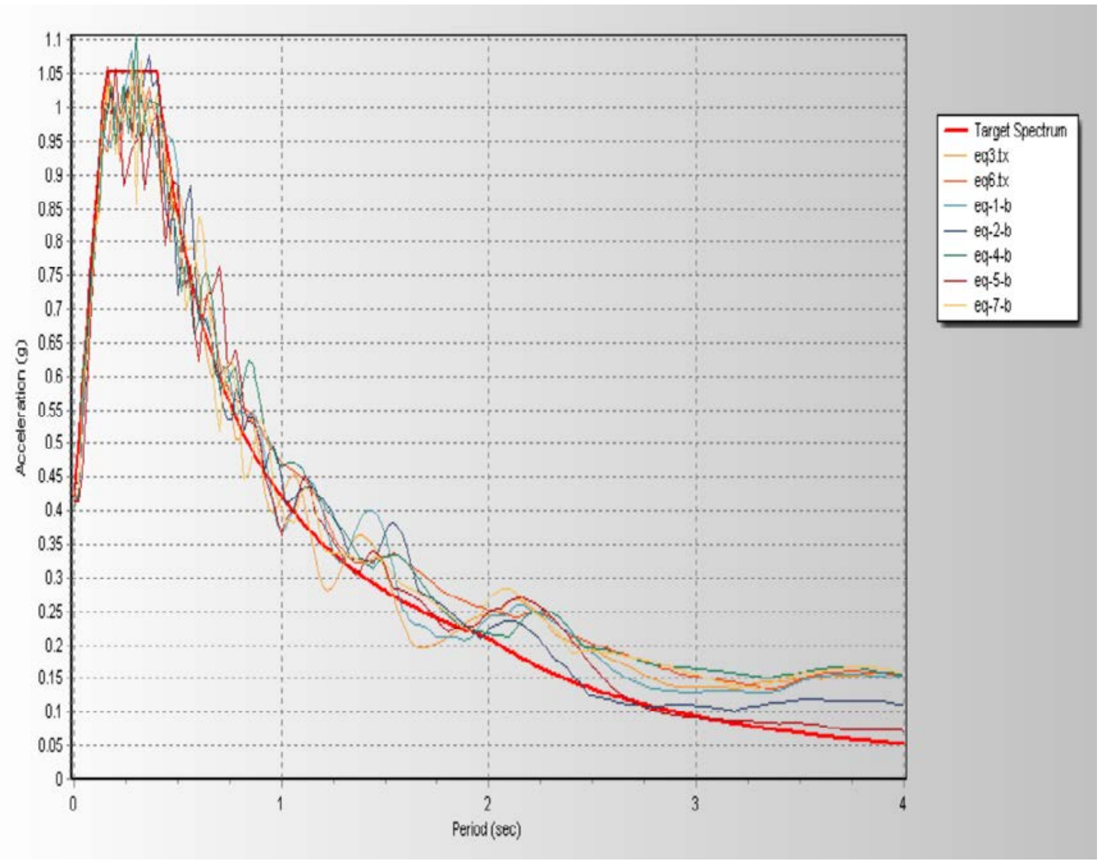


Figure 4.12: Acceleration - Period curve.

4.4 Structural Results

After running of DRAIN-2DX the structural demands in terms of internal forces and deformations will be calculated and compared with the regulations of Turkish earthquake code. In section 7.6 in Turkish Earthquake Code, structural performance evaluations regulations are clearly defined if the structural analysis is performed by nonlinear methods. Table 4.4 shows the limiting values depending on the strain values for concrete and steel. These values are the border values between Minimum Damage Zone-Significant Damage Zone; Significant Damage Zone-Advanced Damage Zone and Advanced Damage Zone- Collapse Damage Zone, respectively.

Table 4.5: Limit to identify Damage Zones.

	ε_c	ε_s
1	0.0035	0.010
2	0.0135	0.040
3	0.018	0.060

Below Figures 4.13 to 4.19 exhibits the base shear force variation of the building in the y-y direction under the effects of 7 artificial ground motions. It can be seen that the base shear demand varies between 2707.9 KN to 3242.3 KN. The average values is calculated to be 2929.37 KN. Similar variation for the x-x direction of the building is given is Appendix-B of the thesis.

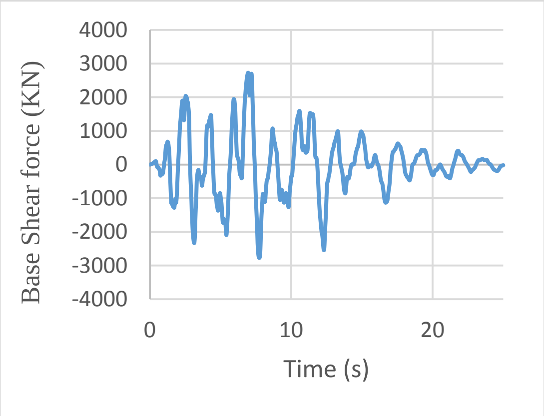


Figure 4.13: 1st earthquake (-2771 KN)

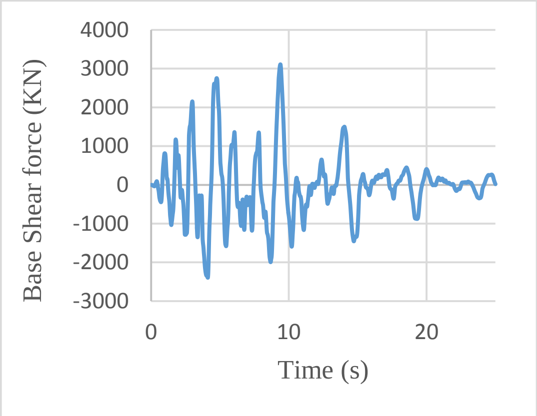


Figure 4.14: 2nd earthquake (3110.7 KN)

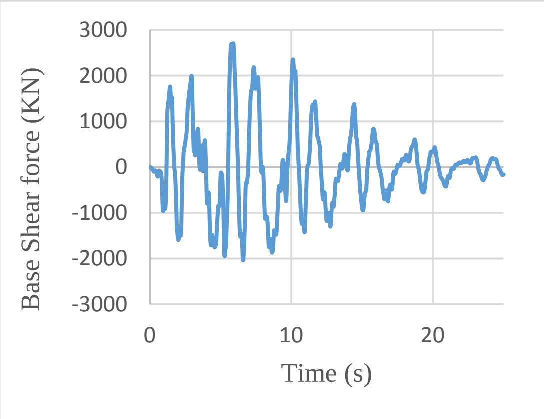


Figure 4.15: 3rd earthquake (2707.9 KN)

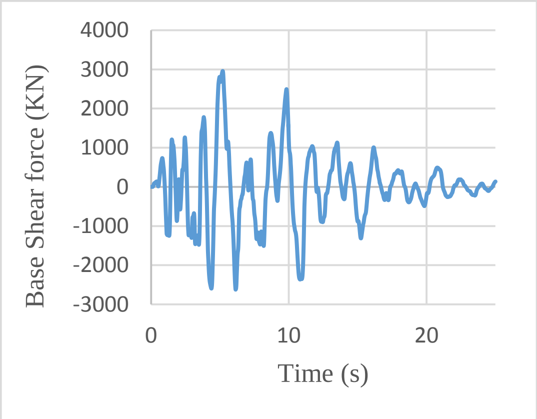


Figure 4.16: 4th earthquake (2957.5 KN)

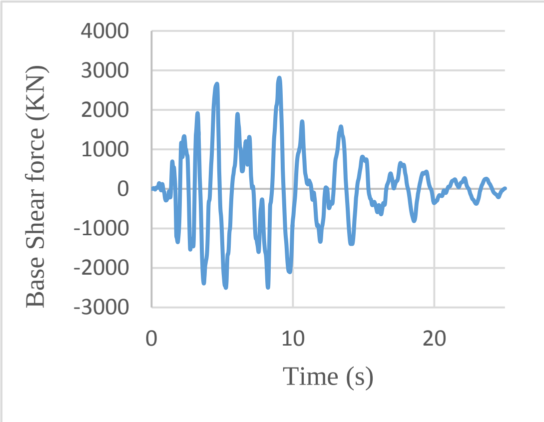


Figure 4.17: 5th earthquake (2815.3 KN)

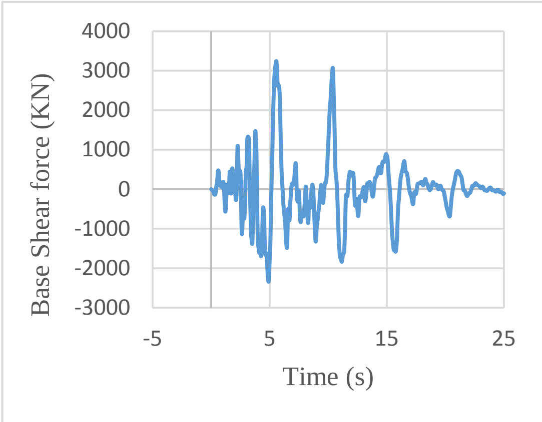


Figure 4.18: 6th earthquake (3242.3 KN)

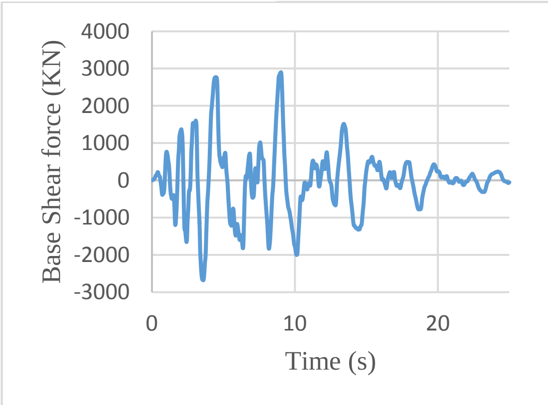


Figure 4.19: 7th earthquake (2900.2 KN)

Since on other important structural response parameter is the top-story displacement, its variation is also investigated for each direction of the building. Figures 4.20 to 4.26 show the roof displacement variation of the y-y direction of the building subjected each ground motion. From the figures, it is understood that the top-story displacement demand varies between 0.102m to 0.148m with an average of 0.120 m. similar variation for the x-x structural direction are provided in Appendix-A.

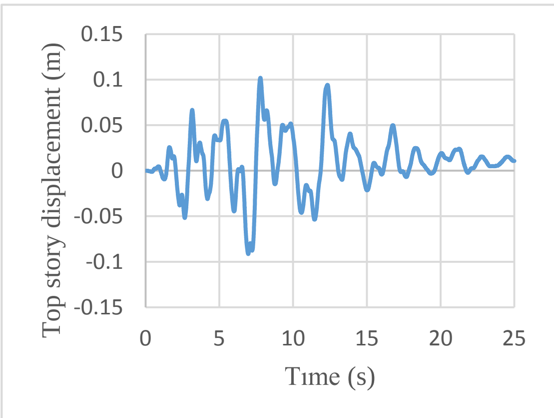


Figure 4.20: 1st earthquake record (0.102m)

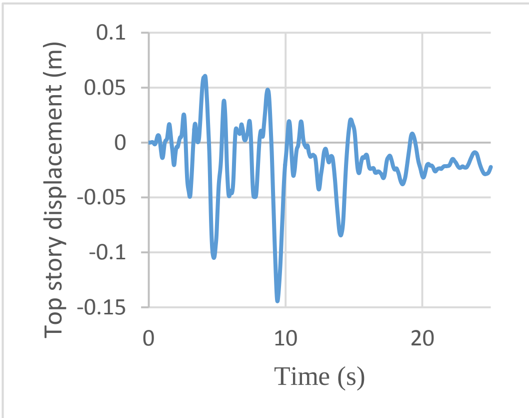


Figure 4.21: 2nd earthquake record (-0.1444m)

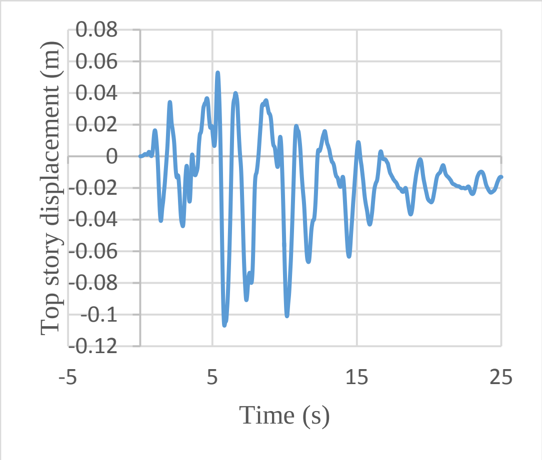


Figure 4.22: 3rd earthquake record (-0.107m)

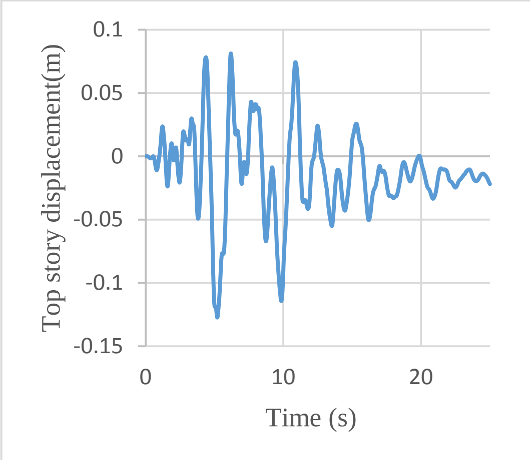


Figure 4.23: 4th earthquake record (-0.127m)

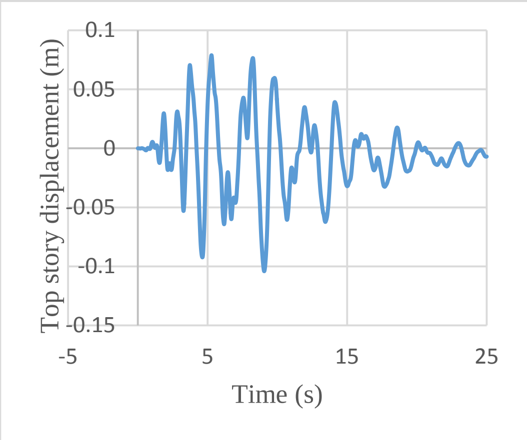


Figure 4.24: 5th earthquake record (-0.104m)

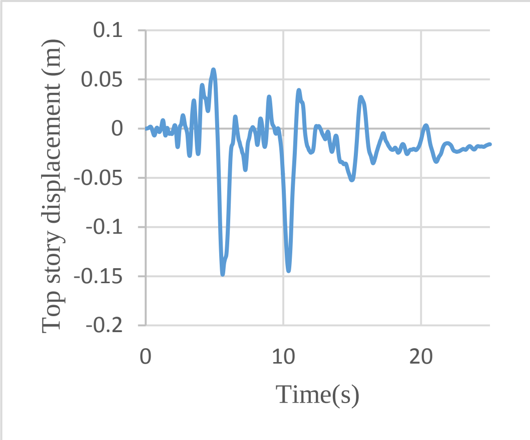


Figure 4.25: 6th earthquake record (-0.148m)

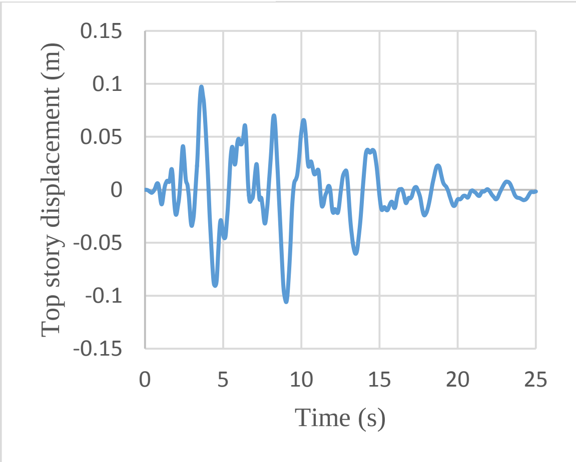


Figure 4.26: 7th earthquake record (-0.106m)

To illustrate the nonlinear hysteretic behavior of the building much better, base shear-roof variations are plotted in Figures 4.27 to 4.33. In this figures, gray and orange vertical lines indicate the code limits for permitted displacement values, while the blue and yellow horizontal lines show the equivalent earthquake force that is considered during the design. Henceforth, it can be seen that either the design base shear forces or the displacement limits are exceeded in this building. However, it should also be considered that during the year of the design, 1975 code was employed and moreover Istanbul was totally defined in the second seismic zone. Hysteresis curves for the x-x direction are given in Appendix-C.

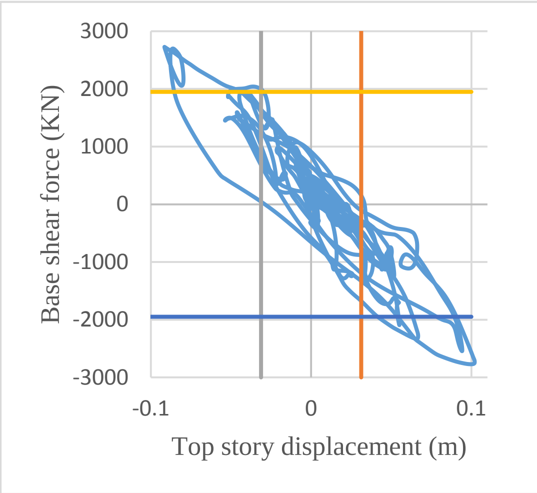


Figure 4.27: 1st earthquake record

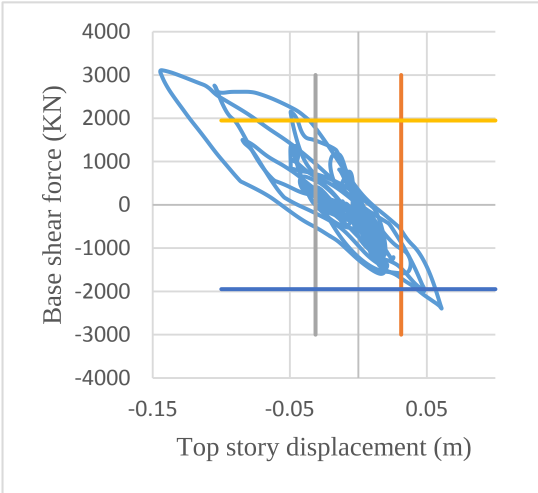


Figure 4.28: 2nd earthquake record

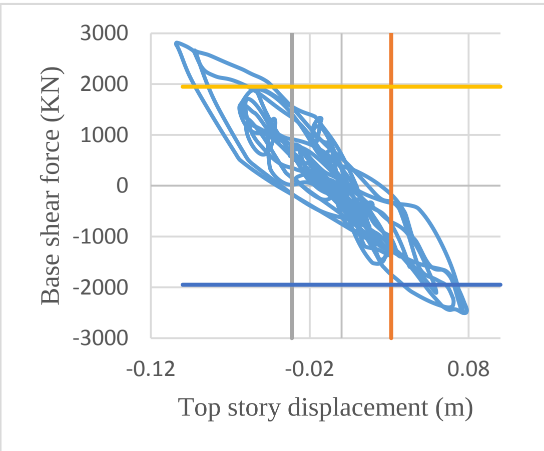


Figure 4.29: 3rd earthquake record

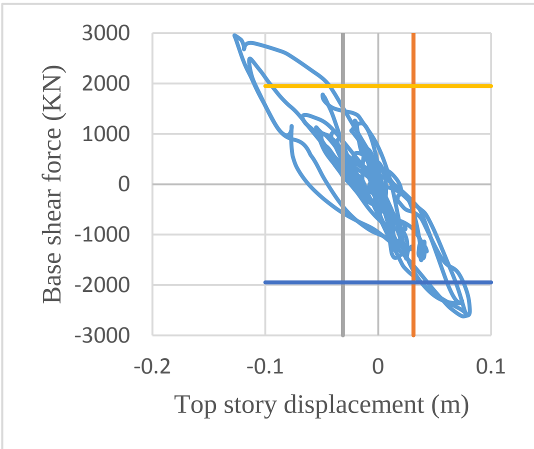


Figure 4.30: 4th earthquake record

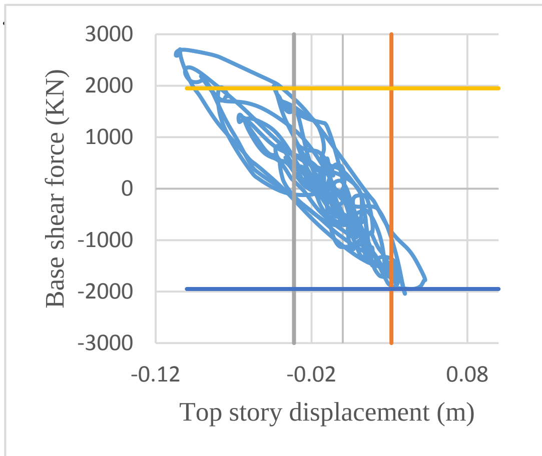


Figure 4.31: 5th earthquake record

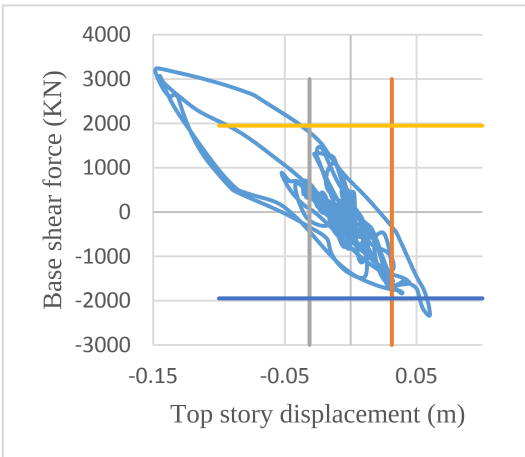


Figure 4.32: 6th earthquake record

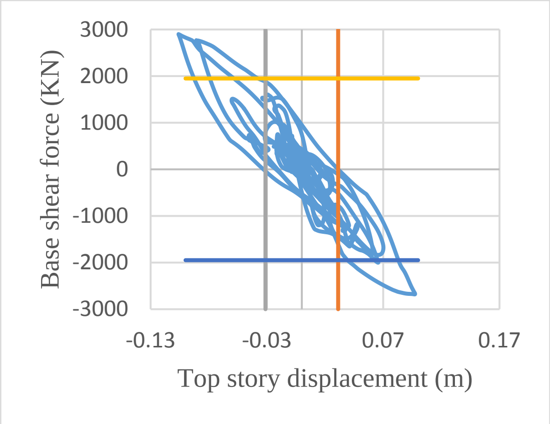


Figure 4.33: 7th earthquake record

To summarize the initial numerical finding, the absolute maximum of the base shear force and top story displacement values are given in Table 4.6.

Table 4.6: Shear force and roof displacement values under seven motions.

Earthquake record	Maximum top story displacement in x-x direction(m)	Maximum top story displacement in y-y direction(m)	Maximum shear force in x-x direction (KN)	Maximum shear force in y-y direction (KN)
1	0.163	0.102	2721.5	2771.7
2	0.134	0.144	2452.1	3110.7
3	0.140	0.107	2339.2	2707.9
4	0.177	0.127	2364.7	2957.5
5	0.158	0.104	2317	2815.3
6	0.207	0.148	3049	3242.3
7	0.139	0.106	2489	2900.2
Average	0.160	0.120	2533.21	2929.37

Before proceeding further for structural performance assessment story drift values should be controlled for extreme amount of displacement. Since there are no defined limits for drifts calculated by non-linear analysis in the Turkish Earthquake Code the limits from ASCE-41 (2006) are employed for life safety performance level.

Figures 4.34 and 4.35 illustrate the maximum values of the story drifts with a comparison of the limiting values taken from ASCE-41 for x-x and y-y directions of the building, respectively.

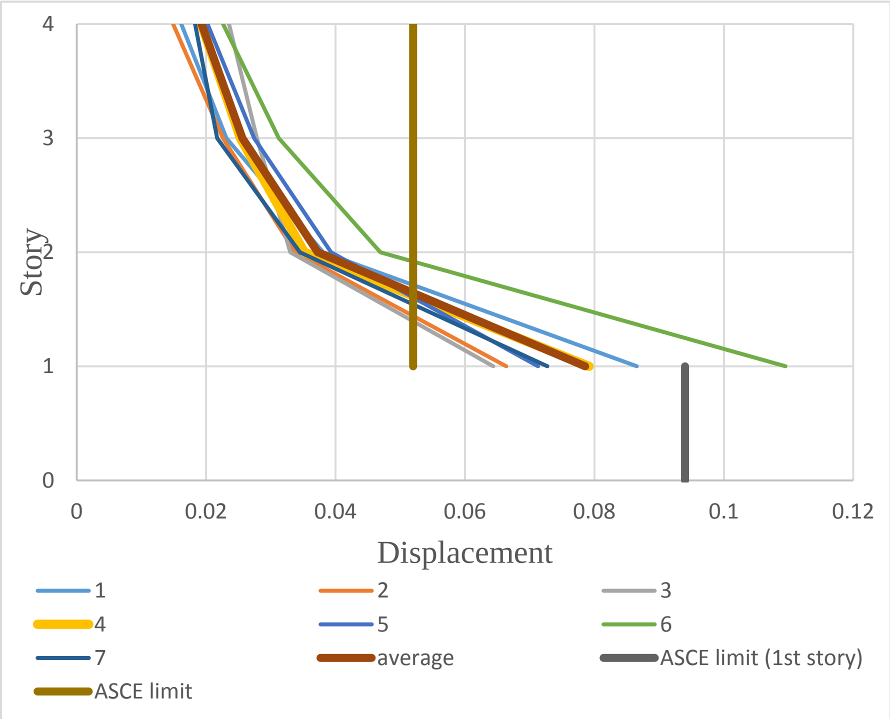


Figure 4.34: Story drift in x-x direction.

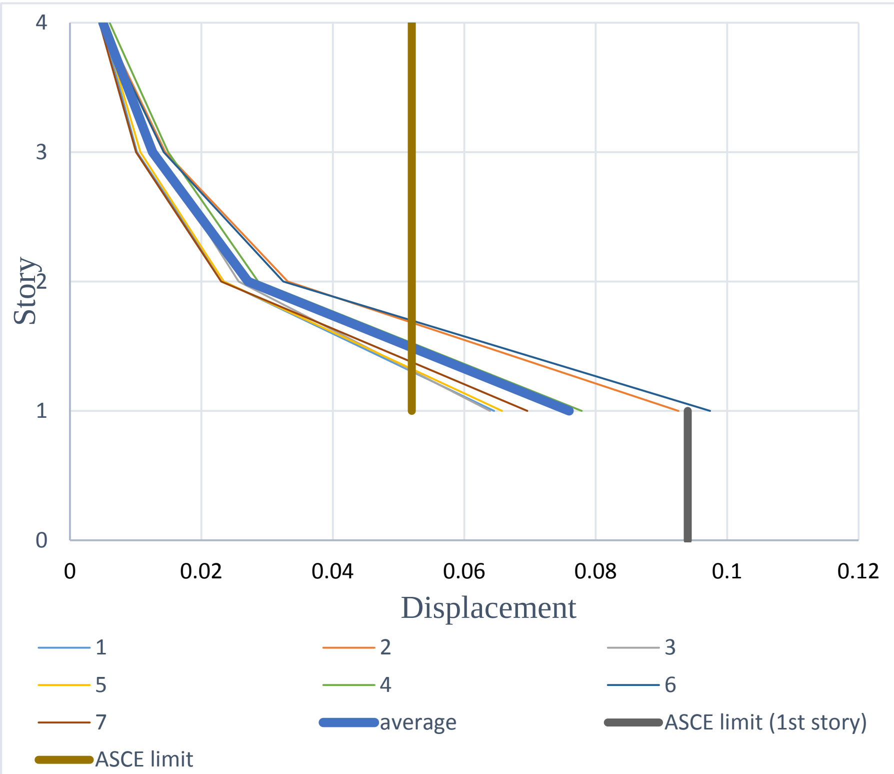


Figure 4.35: Story drift in y-y direction.

The next step is to calculate the strains for concrete and steel then check them with code limits for giving decision about their damage levels. For this propose only ground story's structural elements, which is the critical story under the effect of horizontal loads that is relies on the exceedance of drift limit in this story. Furthermore, the strain values are detail computed under the effect of the fourth ground motion, which is very close to the averages of structural responses as can be seen above in Figures 4.34 and 4.35. Table 4.7 shows the calculated values for concrete and steel strains with the corresponding damage zones for x-x direction. In the Table Φ_t is the total curvature and ε_c and ε_s are the strains for concrete and steel respectively. Similar values and damage zone assignment are tabulated in Table 4.8 for y-y direction of the building.

For example for element number 100 in x-x direction with dimension of 40*40, concrete and steel strains can be calculated same as below:

L_p =h/2=40/2=20cm

Θ_p =0.02227 so $\Phi_p = \Theta_p / L_p$=0.02227/0.2=0.11135

$\Phi_t = \Phi_p + \Phi_y$= 0.12235

$\varepsilon_c = \Phi_t$*x= 0.002967 and $\varepsilon_s = \Phi_t$*(d-x) =0.042303

In above expressions L_p, Θ_p, Φ_p, Φ_t, ε_c, ε_s and x are plastic length, plastic rotation, plastic curvature, total curvature, concrete strain, steel strain and natural axis respectively. As can be seen steel strain of this element is bigger that 0.4 (Significant Damage Zone-Advanced Damage Zone border) so the element number 100 in x-x direction is located in Advanced Damage Zone.

Table 4.7: Damage Zones of elements in x-x direction.

Element No	**Type**	**Φ_t**	**ε_c**	**ε_s**	**Damage zone**
90	wall	0.11845	0.01464	0.15948	Collapse
91	Wall	0.16884	0.00135	0.02735	Significant
92	Wall	0.17009	2.2E-05	0.02889	Significant
93	Wall	0.17014	0.00054	0.02838	Significant
94	Wall	0.16919	0.00071	0.02805	Significant
95	Wall	0.16996	0.00117	0.02772	Significant
96	Wall	0.16261	0.00982	0.02596	Significant
97	Wall	0.0923	0.00294	0.13274	Collapse

Table 4.7 (Continued): Damage Zones of elements in x-x direction.

98	Column	0.11375	0.00483	0.03725	Significant
99	Column	0.1206	0.00464	0.03999	Significant
100	Column	0.12235	0.00297	0.0423	Advanced
101	Column	0.1201	0.00459	0.03985	Significant
102	Column	0.12285	0.00296	0.0425	Advanced
103	Column	0.1199	0.00482	0.03955	Significant
104	Wall	0.0809	0.00265	0.11628	Collapse
105	Column	0.1139	0.00488	0.03726	Significant
106	Column	0.12045	0.00466	0.03991	Significant
107	Column	0.1224	0.00295	0.04234	Advanced
108	Column	0.12015	0.00459	0.03987	Significant
109	Column	0.12285	0.00296	0.04249	Advanced
110	Column	0.1198	0.0049	0.03943	Significant
111	Wall	0.0809	0.00254	0.11639	Collapse
112	Column	0.0906	0.0076	0.12558	Collapse
113	Column	0.12225	0.00621	0.03902	Significant
114	Column	0.1218	0.00148	0.04359	Advanced
115	Column	0.1266	0.00596	0.04088	Advanced
116	Column	0.1293	0.00182	0.04602	Advanced
117	Column	0.12955	0.00183	0.04611	Advanced
118	Column	0.12775	0.00194	0.04533	Advanced
119	Wall	0.07315	0.00056	0.10697	Collapse
1	Beam	0.16836	0.00259	0.0597	Advanced
2	Beam	0.14952	0.00213	0.0532	Advanced
3	Beam	0.15076	0.00209	0.05369	Advanced
4	Beam	0.15056	0.00209	0.05362	Advanced
5	Beam	0.15064	0.00208	0.05365	Advanced
6	Beam	0.15684	0.00225	0.05578	Advanced
7	Beam	0.16264	0.00072	0.05945	Advanced
8	Beam	0.2088	0.00258	0.07468	Collapse
9	Beam	0.20696	0.00266	0.07392	Collapse
10	Beam	0.20664	0.00269	0.07377	Collapse
11	Beam	0.208	0.00262	0.07434	Collapse

Table 4.7 (continued): Damage Zones of elements in x-x direction.

12	Beam	0.20424	0.00266	0.07291	Collapse
13	Beam	0.2092	0.00287	0.07453	Collapse
14	Beam	0.2088	0.00264	0.07462	Collapse
15	Beam	0.20708	0.0027	0.07392	Collapse
16	Beam	0.20736	0.00277	0.07395	Collapse
17	Beam	1.7388	0.02273	0.62063	Collapse
18	Beam	0.20848	0.00275	0.07439	Collapse
19	Beam	0.2108	0.00279	0.0752	Collapse
20	Beam	0.21556	0.00374	0.07602	Collapse
21	Beam	0.20032	0.00262	0.0715	Collapse
22	Beam	0.18184	0.00338	0.0639	Collapse
23	Beam	0.18892	0.00247	0.06743	Collapse
24	Beam	0.16096	0.00232	0.05723	Advanced
25	Beam	0.1682	0.00313	0.05911	Advanced
26	Beam	0.19872	0.00367	0.06985	Collapse
27	Beam	0.15844	0.00178	0.05684	Advanced
28	Beam	0.1606	0.0022	0.05722	Advanced

Table 4.8: Damage Zones of elements in y-y direction.

Element No	**Type**	**Φ_t**	**ε_c**	**ε_s**	**Damage zone**
73	Wall	0.1843	0.003019297	0.0283117	Significant
74	Wall	0.17404	0.001880369	0.02770643	Significant
75	Wall	0.17424	0.003064958	0.02655584	Significant
76	Wall	0.17274	0.004107941	0.02525786	Significant
77	Wall	0.13484	0.007817242	0.04207356	Advanced
78	Column	0.12856	0.009980688	0.03758651	Significant
79	Column	0.13476	0.007672715	0.04218849	Advanced
80	Wall	0.09385	0.013036854	0.12492265	Collapse
81	Column	0.1442	0.004109178	0.04924482	Advanced
82	Column	0.13805	0.003933926	0.04714457	Advanced
83	Column	0.14195	0.003817334	0.04870417	Advanced
84	Wall	0.0915	0.004885565	0.12961944	Collapse
85	Column	0.1442	0.0083372	0.0450168	Advanced

Table 4.8 (Continued): Damage Zones of elements in y-y direction.

86	Column	0.138	0.010696194	0.04036381	Advanced
87	Column	0.14195	0.00807762	0.04444388	Advanced
88	Wall	0.09165	0.012725483	0.12200002	Collapse
89	Column	0.1442	0.004109178	0.04924482	Advanced
90	Column	0.13805	0.003933926	0.04714457	Advanced
91	Column	0.14195	0.003817334	0.04870417	Advanced
92	Wall	0.0915	0.004885565	0.12961944	Collapse
93	Column	0.1442	0.0083372	0.0450168	Advanced
94	Column	0.138	0.010696194	0.04036381	Advanced
95	Column	0.14195	0.00807762	0.04444388	Advanced
96	Wall	0.09165	0.012725483	0.12200002	Collapse
97	Column	0.1467	0.004196109	0.05008289	Advanced
98	Column	0.13875	0.003968713	0.04736879	Advanced
99	Column	0.14445	0.003843215	0.04960328	Advanced
100	Wall	0.1214	0.00516792	0.08831008	Collapse
101	Wall	0.18255	0.002990627	0.02804287	Significant
102	Wall	0.17965	0.001408417	0.02913208	Significant
103	Wall	1.1819	0.014088119	0.18683488	Collapse
104	Wall	2.18225	0.057808917	0.31317358	Collapse
1	Beam	0.08292	0.008648929	0.02203147	Significant
2	Beam	0.13089	0.013652416	0.03477688	Advanced
3	Beam	0.08121	0.008470568	0.02157713	Significant
4	Beam	0.07368	0.003202148	0.02405945	Significant
5	Beam	0.09714	0.003869914	0.03207189	Significant
6	Beam	0.08806	0.003827106	0.02875509	Significant
7	Beam	0.09738	0.004232155	0.03179844	Significant
8	Beam	0.08782	0.003816675	0.02867672	Significant
9	Beam	0.09684	0.004208687	0.03162211	Significant
10	Beam	0.08796	0.00382276	0.02872244	Significant
11	Beam	0.08329	0.003619801	0.0271975	Significant
12	Beam	0.07382	0.007699758	0.01961364	Significant
13	Beam	0.08318	0.008676048	0.02210055	Significant
14	Beam	0.07352	0.007668467	0.01953393	Significant

After calculation of concrete and reinforcement strains of each elements in both direction and compared them with 2007 Turkish earthquake code, in x-x direction 15 columns of 30 columns (50%) are located in Significant Damage region while 9 columns are in Advanced Damage Region (30%) and 6 columns are in Collapse Damage Zone (20%) and also 11 beams of 28 are in Advance Damage Zone (39%) and 17 are in Collapse Damage Zone (61%).

In y-y direction, 7 columns are in Significant Damage zone (22%), 17 columns are in Advance Damage zone (53%) and 8 columns are in Collapse Damage zone (25%) and for beams also 13 out of 14 beams are in Significant Damage zone (93%) and one beam is in Advance Damage zone (7%).

5 CONCLUSION

Before than analyzing this building with non-linear dynamic analysis, KASIMPASA building was analyzed by linear analysis method.

In the Table 5.1, element damage zone for the ground story results of x-x and y-y direction from the nonlinear dynamic analysis and linear analysis are which had been done before, compared to each other, in which linear performance analysis values are taken from the previous investigations conducted by Istanbul Technical University teams.

Table 5.1: Element damage zones according to Linear and Nonlinear analysis.

Linear Analysis (x-x)				
	Minimum	Significant	Advanced	Collapse
Columns				100%
Beams	6%	3%	18%	74%
Walls			20%	80%
Non-Linear dynamic Analysis (x-x)				
	Minimum	Significant	Advanced	Collapse
Columns		47%	47%	6%
Beams			39%	61%
Walls		55%		45%
Linear Analysis (y-y)				
	Minimum	Significant	Advanced	Collapse
Columns		5%	26%	68%
Beams	31%		7%	62%
Walls			88%	13%

Table 5.1 (Continued): Element damage zones in Linear and Nonlinear analysis.

Non-Linear dynamic Analysis (y-y)				
	Minimum	Significant	Advanced	Collapse
Columns		6%	94%	
Beams		93%	7%	
Walls		40%	7%	53%

When linear and nonlinear analysis results are compared, it insight be pronounced that non-linear method is conservative. In the nonlinear analysis each structural element loos its stiffness and strength depending on its hysteretic behavior which yields to the fact that structural forces are limited after degradation. However, in the linear method only cracked sections stiffnesses are defined initially but these do not change during the analysis. When structural periods before and after the loadings are inspected, this phenomena can be clearly seen.

As can be seen in Figure 4.3 in some frames there are infill walls and had been run program with them and for second times structure has been analyzed without any infill walls which top story displacement and base shear force are shown below in Figures 5.1 till 5.4 subjected to fourth artificial earthquake record.

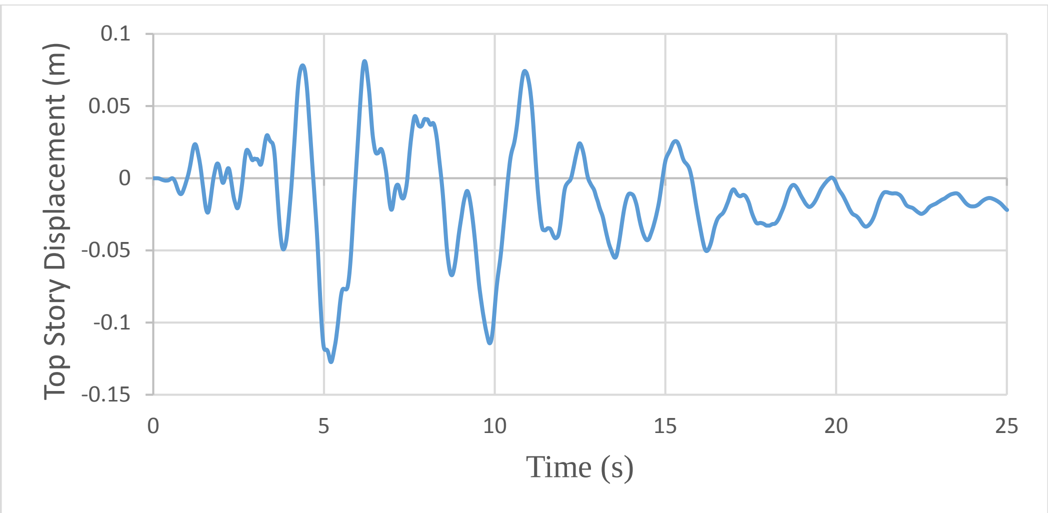


Figure 5.1: Displacement subjected to 4th record with infill walls (Max: 0.127m).

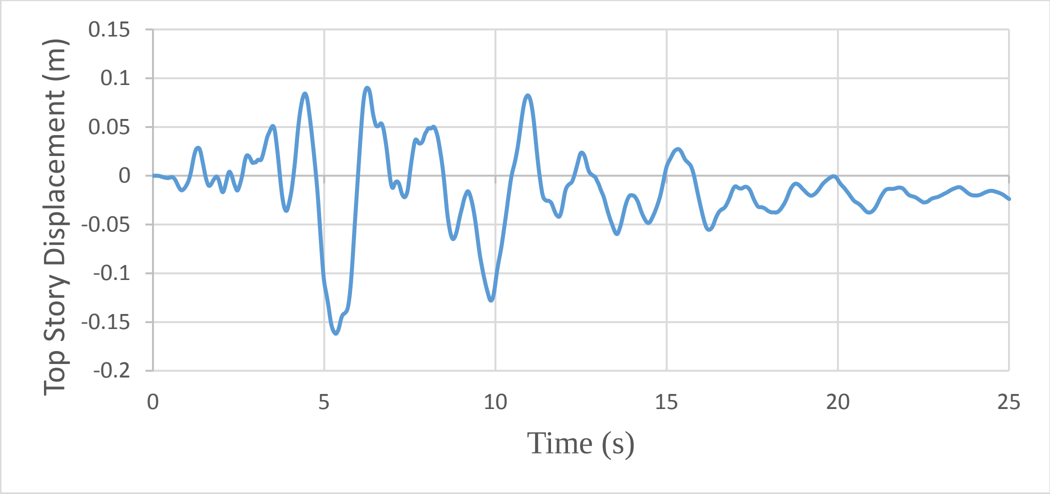


Figure 5.2: Displacement subjected to 4th record without infill walls (Max: 0.162m).

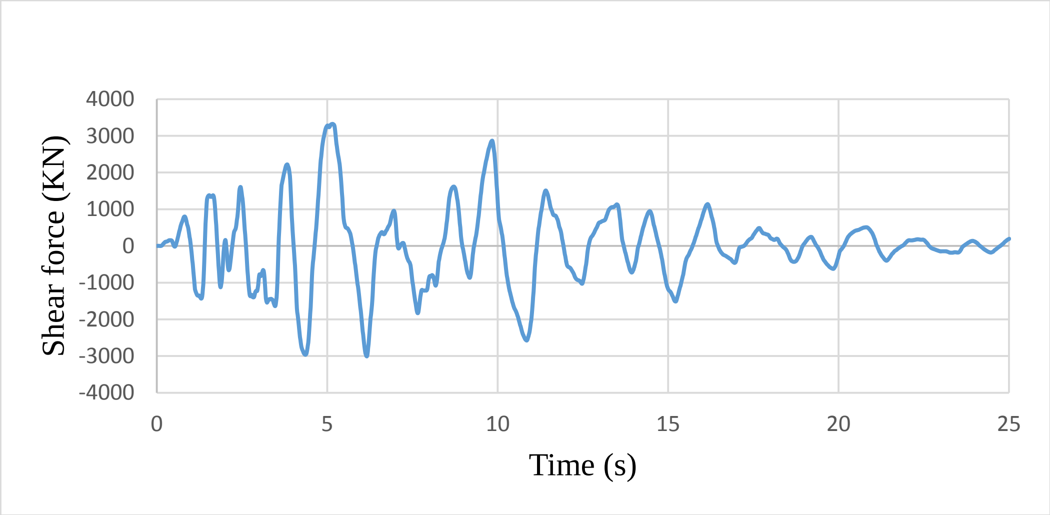


Figure 5.3: Shear force subjected to 4th record with infill wall (Max: 3324KN).

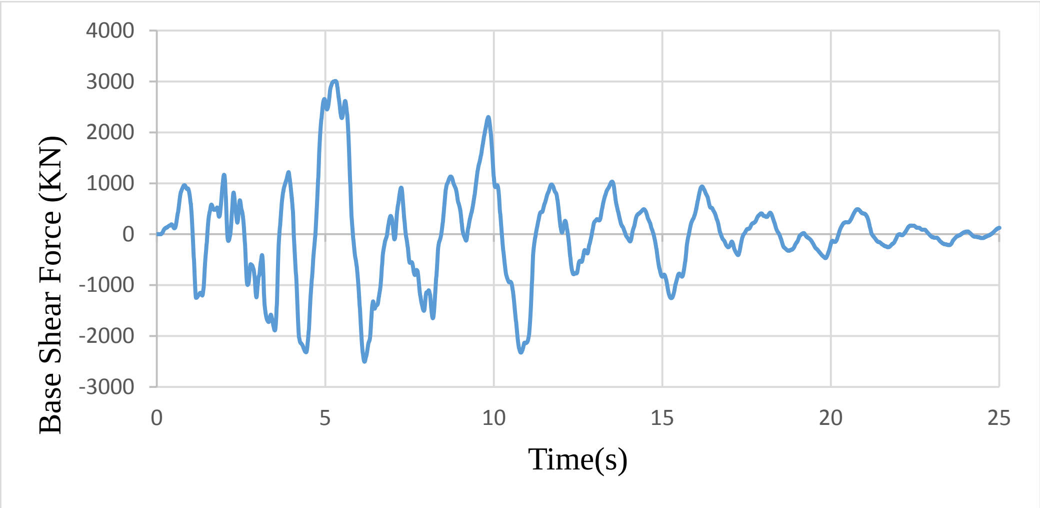


Figure 5.4: Shear force subjected to 4th record without infill walls (Max: 3007KN).

As can be seen top story displacement with infill walls (0.127m) is smaller that without infill walls (0.162m) and in the other hand base shear force with infill walls (3324KN) is bigger that without infill walls (3007KN).

After comparing top story displacement and base shear force the vibration period results are compared in Table 4.9 for both analysis.

Table 5.2: Vibration periods with infill walls and without infill walls.

Vibration Periods (s)	3D Linear Model		2D Non Linear Model with Infills		2D Non Linear Model without Infills	
	Initial	End	Initial	End	Initial	End
T_1 (x-x)	0,645	0,949	0,596	2,433	0,781	2.459
T_2(y-y)	0,515	0,820	0.160	0.366	0.193	0.413

As can be seen above in Table 4.9 after analyzing the structure without infill walls, vibration periods be greater in campare with infill walls analysis.

In this thesis seven artificial earthquake records have been used but this records are very tough and acceeleration are bigger that real earthquake records so in the reality earthquake, KASIMPASA building acts better. But in totally in x direction 20% and in y direction 25% of columns in ground floor are in Collapse Damage zone and it illustrates that the structure is not strong enough against earthquake with this magnitudes.

So KASIMPASA building must be retrofitted to improve its seismic resistance. The primary purpose of earthquake retrofitting is to strengthen the property and keep this structure from being partially or totally collapse and preserve life safety.

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APPENDICES

APPENDIX A : Roof displacement variation of the building in the x-x direction under the effects of 7 artificial ground motions.

APPENDIX B : Base shear force variation of the x-x direction of the building subjected each ground motion.

APPENDIX C : Hysterestic curves for the x-x direction subjected to 7 artificial ground motions.

APPENDIX A

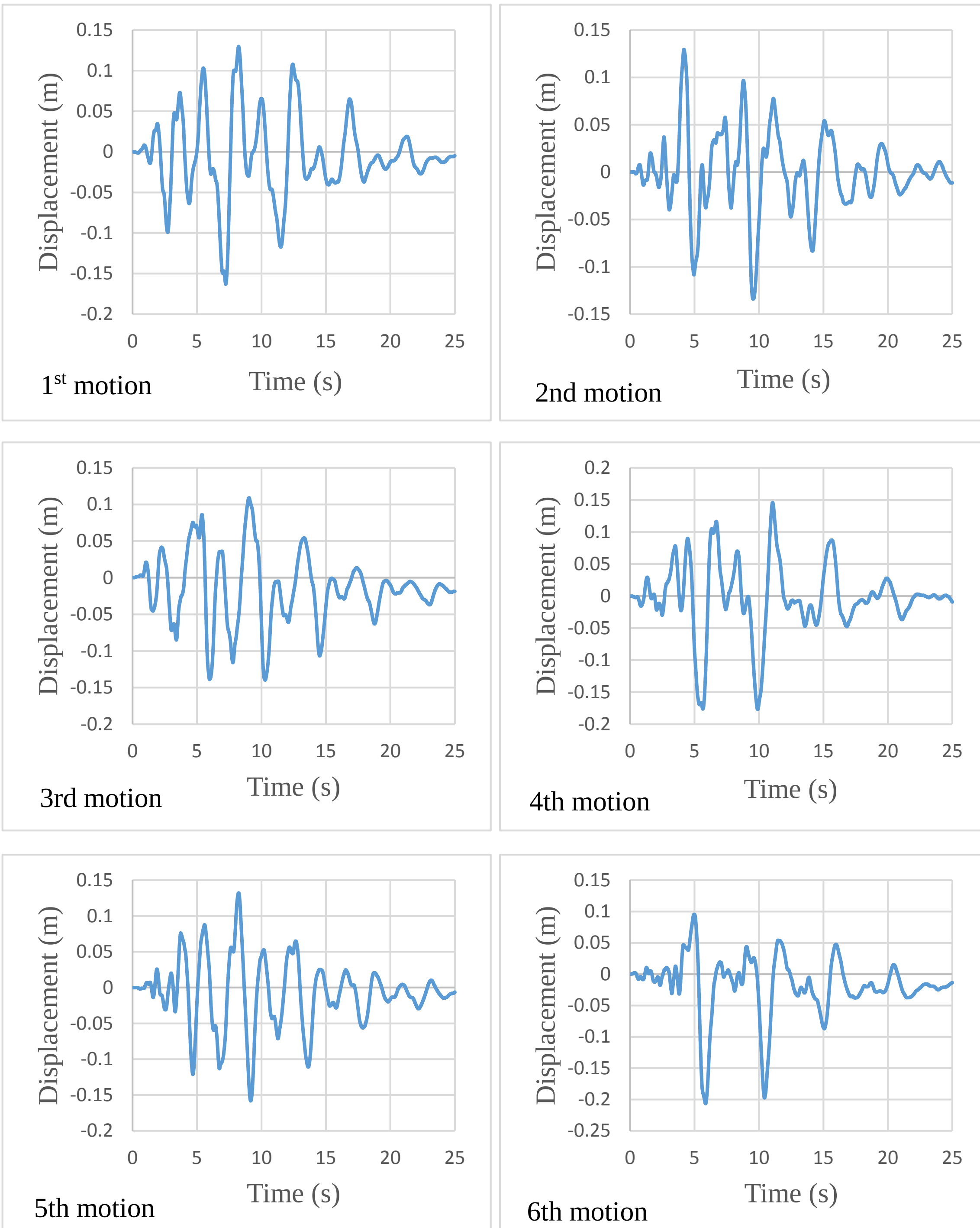

Figure A.1: Roof displacement variation of the building in the x-x direction under the effects of 7 artificial ground motions.

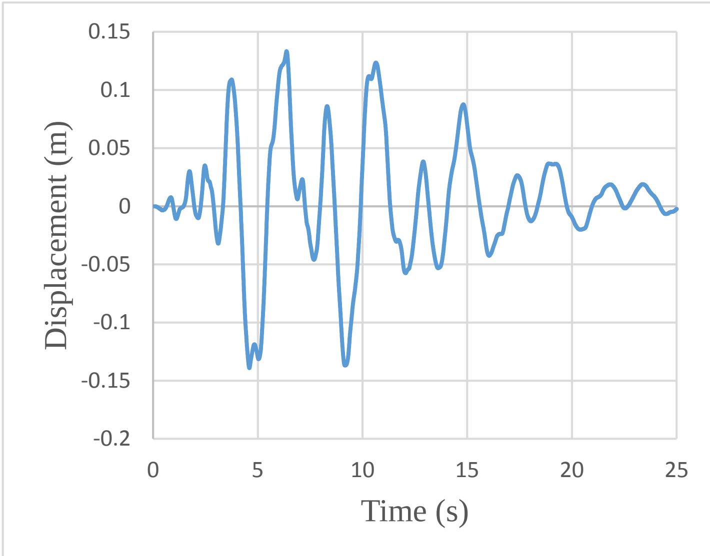

Figure A.1 (Continued): Roof displacement variation of building in the x-x direction under the effects of 7 artificial ground motions.

APPENDIX B

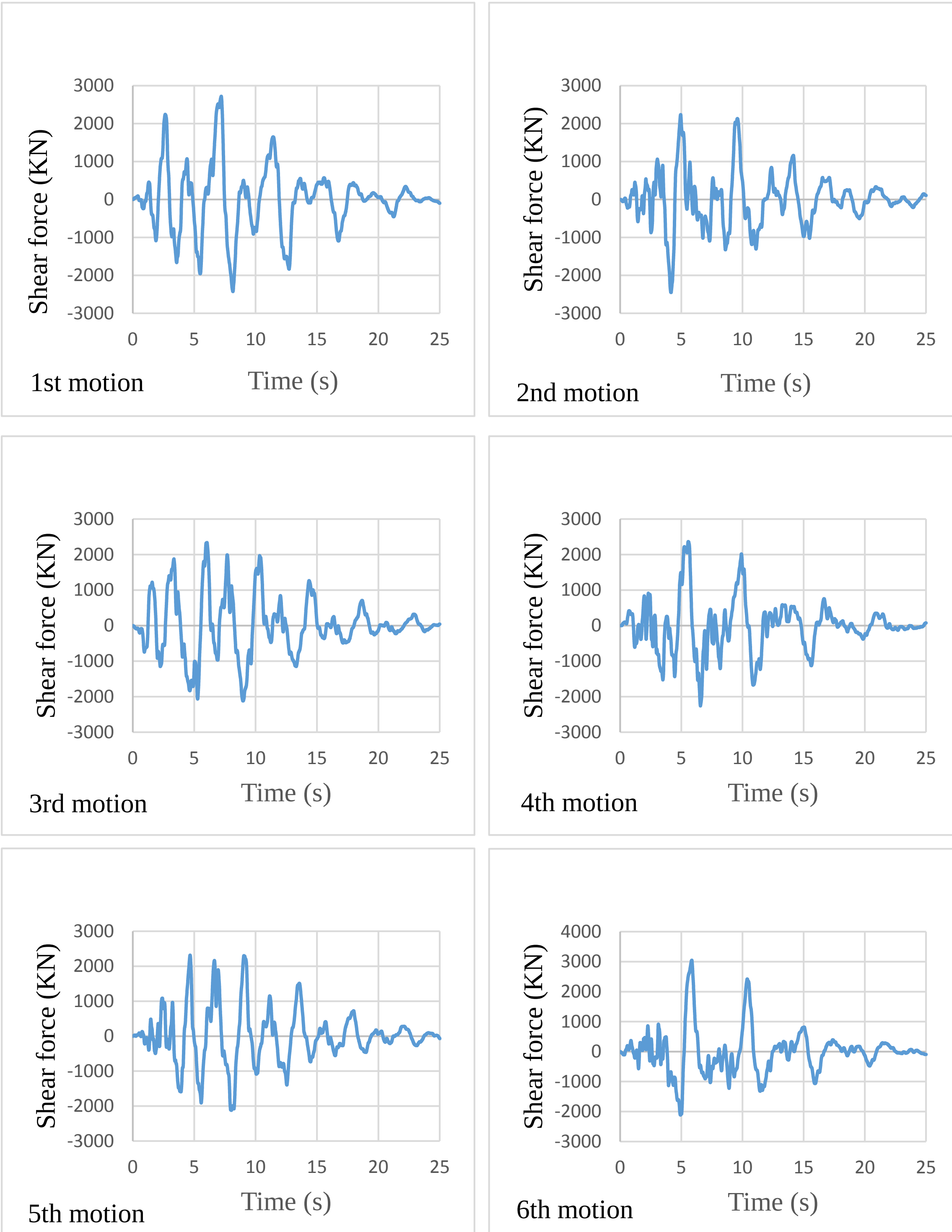

Figure B.1: Base shear force variation of x-x direction subjected to 7 motions**.**

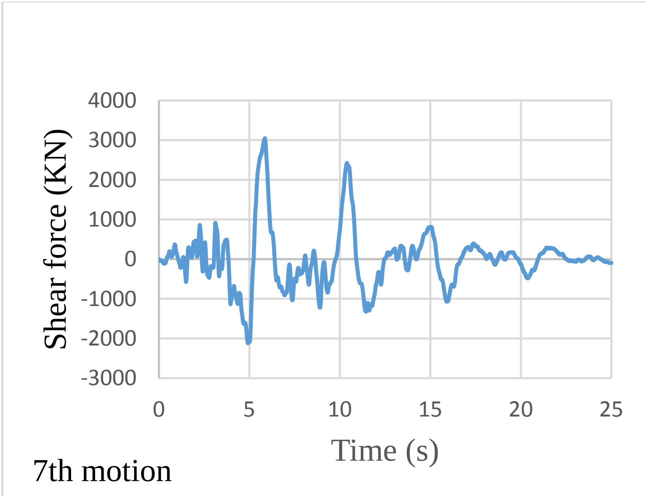

Figure B.1 (Continued): shear force variation of x-x direction subjected to 7 motions**.**

APPENDIX C

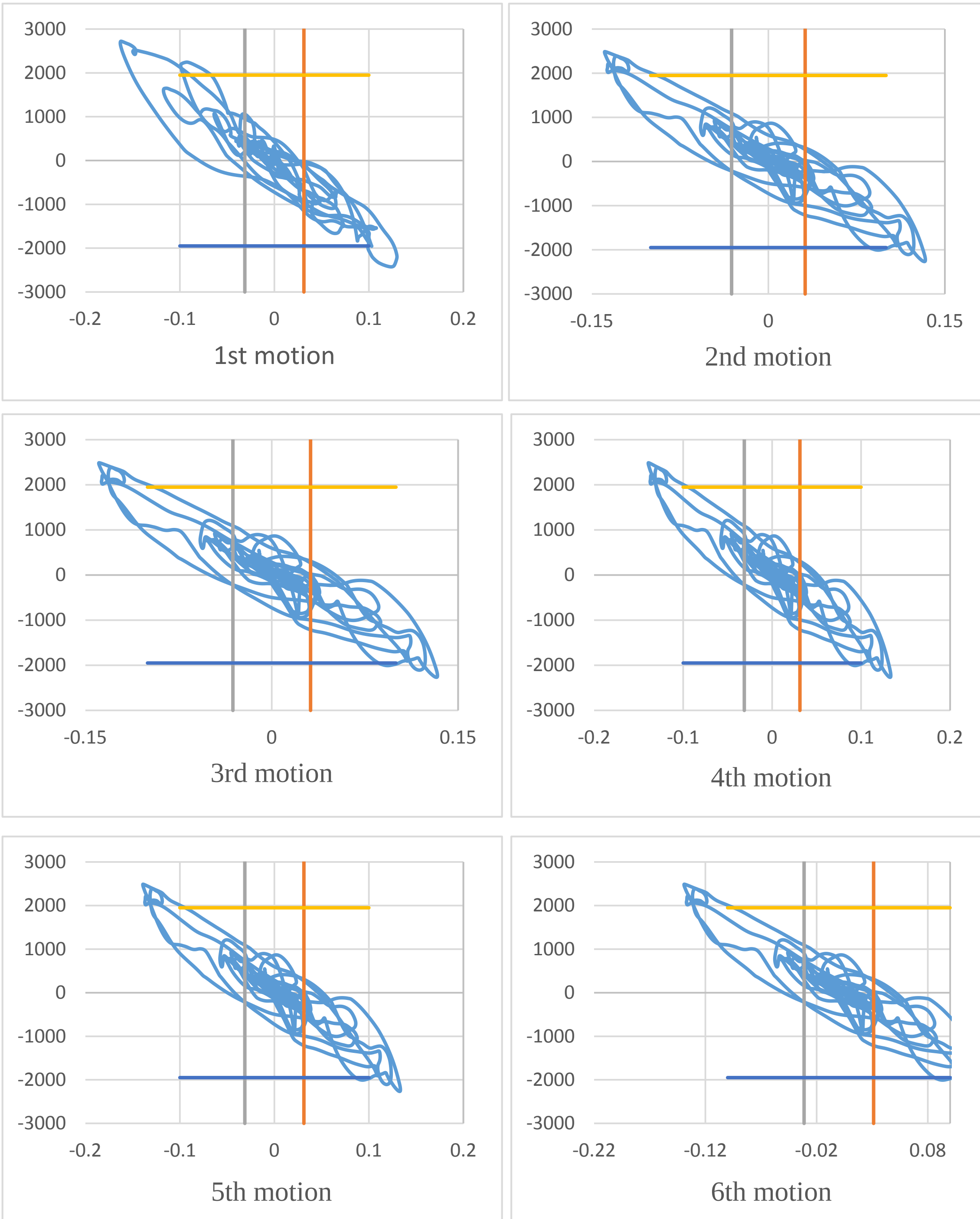

Figure C.1: Hysterestic curves for x-x direction subjected to 7 artificial motions.

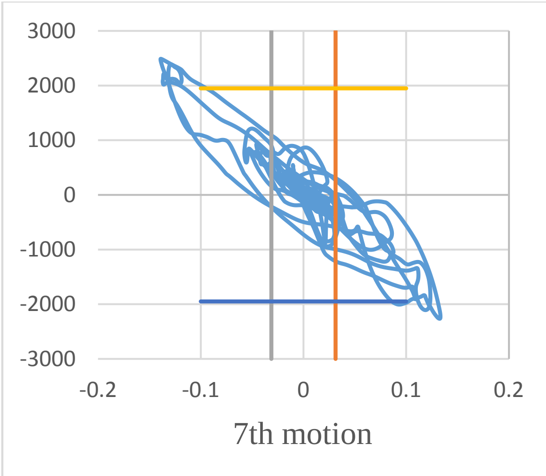

Figure C.1 (Continued): Hysterestic curves for x-x direction subjected to 7 motions.

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