ISTANBUL TECHNICL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE, ENGINEERING AND TECHNOLOGY

DESIGN AND ANALYSIS OF A HIGH-RISE BUILDING LOCATED IN TWO DIFFERENT SEISMIC ZONES UTILIZING ISTANBUL SEISMIC DESIGN CODE FOR TALL BUILDINGS (ISDCTB)

M.Sc. THESIS

Mustafa Sahib AL-LUHAIBI

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

İSTANBUL YÜKSEK BİNALAR DEPREM YÖNETMELİĞİNE GÖRE İKİ FARKLI SİSMİK BÖLGEDE KONUMLANAN YÜKSEK BİR BİNANIN ANALİZ VE TASARIMI

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Haziran 2017

Mustafa Sahib Al-Luhaibi, a **M.Sc.** student of ITU **Graduate School of Science Engineering and Technology,** student ID **501151032**, successfully defended the thesis entitled "**DESIGN AND ANALYSIS OF A HIGH-RISE BUILDING LOCATED IN TWO DIFFERENT SEISMIC ZONES UTILIZING ISTANBUL SEISMIC DESIGN CODE FOR TALL BUILDINGS (ISDCTB)**" which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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Date of Submission: 05 May 2017
Date of Defense: 12 June 2017

To my parents,

FOREWORD

This thesis has been written to fulfill the M.Sc. degree requirements of the Civil Engineering Department/ Structural Engineering Programme at Istanbul Technical University (ITU).

I would like to express my special appreciation and thanks to my supervisor Assoc.Prof. Dr. Beyza Taşkın for her excellent guidance and support during this process. Without her active support I would be completely lost.

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I would like to take this opportunity to thank my parents, my sisters and my brother for their love, support and encouragement throughout my life. Words cannot express how grateful I am.

A heartfelt thanks to my beloved Marwa. Despite the distance, your love always makes me feel like you are beside me. I love you and I missed you.

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Last but not least to all my friends everywhere and my colleagues at ITU, I would like to thank you for your wonderful cooperation and for being in my life.

June 2017
Mustafa Sahib Al-Luhaibi
(Civil Engineer)

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ABBREVIATIONS

TEC : Turkish Earthquake Code

ISDCTB : Istanbul Seismic Design Code for Tall Buildings

PEER : Pacific Earthquake Engineering Research Centre

RC : Reinforcement Concrete

NAF : North Anatolian Fault

EAF : East Anatolian Fault

GSHAP : Global Seismic Hazard Assessment Program

LATBSDC : Los Angeles Tall Buildings Structural Design Council

TBI : Tall Building Initiative

CTBUH : Council on Tall Buildings and Urban Habitat

SEAONC : Structural Engineers Association of Northern California

TTBDG : Toronto Tall Building Design Guidelines

UBC : Uniform Building Code

NDTHA : Nonlinear Dynamic Time History Analysis

ASCE : American Society of Civil Engineers

ATC : Applied Technology Council

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DESIGN AND ANALYSIS OF A HIGH-RISE BUILDING LOCATED IN TWO DIFFERENT SEISMIC ZONES UTILIZING ISTANBUL SEISMIC DESIGN CODE FOR TALL BUILDINGS (ISDCTB)

SUMMARY

The buildings are exposed to dynamic loads which are caused mainly and commonly by both wind and earthquakes and this study will be focused on the seismic design due to the huge earthquake effects which have been known as one of the major problems that threaten the life of people in vast areas around the world. The aim of the thesis is covering the topic of design and analysis a high-rise RC building depending on basic motivations and challenges summarized as the increase of the tendency towards the construction of high-rise buildings around the world, to improve the author's abilities in the analysis and design of this kind of structures due to the lack of experience in this field specially in Iraq as well as checking the capability of the draft version of Istanbul Seismic Design Code for Tall Buildings (ISDCTB-2008) in evaluating the seismic performance by employing its requirements not only for Istanbul but also for site with less effects of seismicity which has been already taken into account in preparing the new draft version of Turkish Earthquake Code-2016 by adding a new chapter for ‘‘Tall Structures’’. The thesis is composed of five chapters, each of them dealing with different aspects of the study. Chapter one is an introduction to general issues related to the earthquake effects and explains the purpose of the thesis in addition to the review of the previous literature. Chapter two presents some general issues regarding the tall buildings and defines the performance-based design procedure, which is utilized in design and analysis the case study as well as explains the main guidelines for tall buildings in Iraq and Istanbul. Chapter three examines the methodology of the analysis including the analytical techniques, the nonlinear modeling of RC components, the ground motion selection and the summary of the seismic design stages and their acceptance criteria. Chapter four describes the case study and the results and chapter five summarized the conclusion of the study.

The performance-based design procedure is developed by Pacific Earthquake Engineering Research Center (PEER 2010/5) and explained in ISDCTB-2008 as four design stages including two basic analysis types for designing and verification; linear response spectrum and nonlinear time history analysis. These stages have been followed in analysing the case study building and the results showed that the case study has an adequate structural system to be built in two different seismic zones.

İSTANBUL YÜKSEK BİNALAR DEPREM YÖNETMEIİĞİNE GÖRE İKİ FARKLI SİSMİK BÖLGEDE KONUMLANAN YÜKSEK BİR BİNANI ANALİZ VE TASARIMI

ÖZET

Binalar, hem rüzgar hem de depremler tarafından çoğunlukla ve yaygın olarak oluşan dinamik yüklere maruz kalmaktadır ve bu çalışma, insan hayatını tehdit eden başlıca sorunlardan biri olarak bilinen büyük deprem etkilerinden dolayı sismik tasarım üzerine odaklanacaktır Dünyanın dört bir yanındaki geniş alanlarda. Tezin amacı, dünyadaki yüksek katlı binaların inşasına yönelik eğilimin artması olarak özetlenen temel motivasyon ve zorluklara bağlı olarak yüksek katlı bir RC binası tasarlamak ve analiz etmek, yazarın yeteneklerini geliştirmek Irak'ta bu alandaki tecrübesizlik nedeniyle bu tür yapıların analizi ve tasarımı ve İstanbul Sismik Tasarım Yüksek Lisansı Tasarımı Kodunun (ISDCTB-2008) sismik performans değerlendirilmesinde kapasitesinin kontrol edilmesi Sadece İstanbul için değil, aynı zamanda, " Yüksek Yapılar " için yeni bir bölüm ekleyerek, 2016 Türk Deprem Yönetmeliği'nin yeni taslak halinin hazırlanmasında daha önce dikkate alınan sismisiteden daha az etkilenen bir mevkide de kullanılmaktadır. Tez beş fasıldan oluşmakta ve her biri çalışmanın farklı yönlerini ele almaktadır. Birinci bölüm, deprem etkileri ile ilgili genel konulara giriş niteliğindedir ve önceki literatürün gözden geçirilmesine ek olarak tezin amacını açıklamaktadır. İkinci bölüm, yüksek binalarla ilgili bazı genel konuları ve performansı temel alan tasarım prosedürünü tanımlarken, tasarım ve analizde örnek olay incelemesinin yanı sıra Irak ve İstanbul'daki yüksek binalar için ana hatları açıklamaktadır. Üçüncü bölüm analitik teknikler, RC bileşenlerinin doğrusal olmayan modellemesi, yer hareketinin seçimi ve sismik tasarım aşamalarının özetini ve kabul kriterlerini içeren analiz metodolojisini inceler. Dördüncü bölüm vaka incelemesini ve sonuçlarını ve beşinci bölümde çalışmanın sonucunu özetledi.

Performansa dayalı tasarım prosedürü Pasifik Deprem Mühendisliği Araştırma Merkezi (PEER 2010/5) tarafından geliştirildi ve ISDCTB-2008'de tasarım ve doğrulama için iki temel analiz tipi de dahil olmak üzere dört tasarım aşaması olarak açıklandı; Doğrusal tepki spektrumu ve doğrusal olmayan zaman geçmişi analizi. Vaka incelemesi bina analizinde bu aşamalar takip edilmekte ve sonuçlar, vaka incelemesinin iki farklı sismik bölgede inşa edilecek yeterli bir yapısal sisteme sahip olduğunu göstermektedir.

1. INTRODUCTION

1.1 General

A lot of people around the world expressed their feelings during earthquake events when they woke up one day to feel the whole home violently shaking and they couldn't see what was going on, but they could hear things falling off of shelves and breaking and the noise was immense. The shaking continued for some seconds, but for people, they feel like a really long time, and the most important thing came to their mind is “Is our building stiff enough to resist this shaking? And how long will it stay still resisting? ’’. In the engineering concept, this is called the objective of seismic design.

The buildings, in most regions of the world, are exposed to dynamic actions which are caused mainly and commonly by both wind and earthquakes. However, the concept of designing these buildings to resist wind and earthquake effects are distinctly different. Some of the remarkable differences in the structural design are due to the following ideas:

(a) The design forces which affect the building during these natural actions. In wind design, the building is subjected to a pressure on its exposed surface area which will be the influential factor for the building; so it is force-type loading. Whilst in earthquake design, the building is subjected to ground motion at its base, and this earthquake shaking, in turn, induces inertia forces in the building that depend on the mass of the building and the distribution of this mass and cause stresses; so this type of loading is caused by displacement.

(b) Under wind forces, generally, there is no reversal of loads only when the wind’s direction reverses which needs a large duration of time, while under the earthquake, the ground motion is cyclic which causes a reversal of stresses just during a short time.

(c) Under wind forces, if the exposed area is constant, the wind force is proportional to the height of the building. On the other hand, the base shear value, which is created by the earthquake shakings, is generated at the bottom and it decreases with increasing the height due to a decrease of the cumulative weight.

(d) According to the equation of motion, the mass, the stiffness in addition to the damping of the building are the main factors in the seismic design, specially the mass of the building which determines the inertia forces that are induced by the earthquake, but they have less influence in causing the wind force.

(e) When a structure is subjected to a seismic load, if the center of mass and the center of stiffness are at different points, torsion will develop while wind load doesn't cause torsion because it acts at the surface of the building.

This study will be focused on the seismic design due to the earthquake effects which have long been feared as one of nature's most damaging hazards. The earthquakes have been known as one of the major problems that threaten the life of people in vast areas around the world for thousands of years. In the last 100 years, over one million people lost their lives due to earthquakes. Recent major earthquakes have caused severe damage to the people and properties in the urban area where the structures are concentrated. For example, in 1999 two notable earthquakes happened in Turkey, Kocaeli earthquake (also known as Marmara earthquake) with a magnitude of 7.4 and a maximum Mercalli intensity of IX (Violent), this event lasted in an effective duration of 45 seconds, killing around 17,000 people and left approximately half a million people homeless, and Düzce earthquake with a magnitude of 7.2 and a maximum Mercalli intensity of IX (Violent) causing damage with more than 894 fatalities. These consequences make us surprised that a lot of people lost their lives in Turkey in an area where building codes and standards were supposed to have been as good as elsewhere. From these past earthquakes, the experience has proven that many buildings, which are designed according to typical methods of construction, lack basic resistance to earthquake effects. For that reason and to improve the performance of the buildings depending on the main concept of the design methodology for structures to be built in earthquake zones which must provide means of assessing the level of protection as well as ensuring that the desired performance is achieved, the seismic design of the buildings has developed over the years and that's obvious through the changes that have occurred in the conditions of the building seismic codes in various regions around the world in last years. In Turkey, the official institutions for earthquake published the currently enforced version of the seismic code in 2007 (TEC-2007) which also includes major issues of evaluation the seismic performance of existing buildings. But this code doesn’t cover specifically the seismic design of high-rise buildings, which have increased significantly in the past decades especially in Istanbul, therefore a draft version of new code with name of “Istanbul Seismic Design Code for Tall Buildings (ISDCTB)” has been prepared in 2008 that basically follows “Guidelines for Performance-Based Seismic Design of Tall Buildings’’ which is developed by the Pacific Earthquake Engineering Research Centre (PEER 2010/05).

Generally, there are two analysis methods to be employed design the buildings: elastic and inelastic. The elastic design of the buildings requires that the behavior of the building during earthquakes should be elastic without damage and that may make the cost of the project overstated. The seismic design should balance between the cost and the acceptable damage, to make the project reasonable. This provident balance is existed based on extensive researches and damage assessment studies for post-earthquake. Consequently, the inelastic behavior of the building, which allows controlled and limited damage and makes the structure dissipate the energy input to it during the earthquake, may be necessary and realistic for the structure design. Therefore, the main philosophy of the earthquake resistant design of structures aims, by employing the code conditions, to help these structures to resist:

i. Minor earthquakes without damage to structural and non-structural elements;
ii. Moderate earthquakes with minor damage to structural elements, and some non-structural damage; and
iii. Major earthquakes with some structural as well as non-structural damage, but without collapse (for saving people’s life and property inside the building).

Regarding the buildings, there are some characteristics that both architects and design engineers should follow to create the earthquake-resistant design of a building, and they are seismic structural configuration, lateral stiffness, lateral strength, and ductility, in addition to other aspects. First, seismic structural configuration includes three main aspects, namely geometry, in other words, shape and size of the building, location and size of structural elements, and location and size of non-structural elements, and to ensure to get a good configuration, engineers should take into account the influence of each aspect and follow cohesive architectural features that result at the end in good structural behaviour. Secondly, the lateral stiffness of the building refers to the initial stiffness of the building, knowing that stiffness decreases with increasing of damage. At least the minimum lateral stiffness should be uniformly distributed in both directions of the building plan, which ensure that there is no expected harm to people inside the building and no damage to contents of the building. Thirdly, lateral strength of the building refers to the maximum resistance that the building offers during its entire history of resistance to relative deformation. The minimum lateral strength should be also achieved in each direction of the building plan so that it can resist a low-intensity earthquake with no damage and a strong intensity ground shakings with no collapse. Finally, ductility refers to the ratio of the maximum deformation and the yield deformation. A satisfactory level of ductility should be sustained to accommodate the lateral deformation. These four characteristics control the behavior of buildings during an earthquake, and the performance of the buildings is expected to be critical if any one of those characteristics is not achieved.

Due to the importance of seismic action in building design, the designer should have an idea about the definition of seismic hazard. A seismic hazard is a probability that an earthquake will occur in a certain geographic location within a certain period of time, and it is generally represented by hazard curves that describe the relationship between ground motion values, (for example; peak ground acceleration, PGA, peak ground velocity, PGV, or peak ground displacement, PGD) and their annual probabilities of exceedance or recurrence interval. The recurrence interval, in other words, the return period of earthquake events with a certain degree of intensity is defined as the average period of years between the occurrences of earthquakes which have the same or greater degree of intensity. The probability of exceedance is an estimation that earthquakes with a certain degree of intensity or more will occur at a location within a specific number of years. For example, in Turkish Earthquake Code (TEC-2007), the design earthquake considered corresponds to an earthquake with the return period of 475 years, the probability of exceedance of the design earthquake within a period of 50 years is 10 %, for ordinary buildings with importance factor of 1.0, and 2,475 years for the most important buildings with factor of 1.5, where the probability of exceedance equal to 2 % in 50 years. The main purpose of seismic hazard analysis is to provide parameters for estimating seismic risk and with a hazard thus estimated, risk can be assessed and included in the performance-based engineering of buildings to control the levels of damage in a building over the full spectrum of earthquake events which may occur, by using and calculating suitable design parameters, such as effective ground acceleration, shape and ordinates of the design spectra, etc., which are required for each design event to achieve the performance objective.

1.2 Purpose of Thesis

The obvious increase in the tendency to construct high-rise buildings around the world, especially in urban areas, is the focus of attention. In Iraq, this trend represents a step in the change of conventional design of low and mid-rise buildings, which are common there, and a beginning of new era in buildings construction. However, the lack of experience in this field made the investors and the owners of this kind of projects to rely on foreign companies for the purpose of designing tall buildings. This makes the basic motivation and the main challenge for this thesis to be prepared to cover this topic, which is believed to improve the author’s abilities in the analysis and design of high-rise RC structures. Iraq is classified as a low seismicity region. The importance and the challenge of the seismic design for each designer encourage the idea of making a comparison of designing and evaluating the seismic performance of RC high-rise building under two different level of seismic hazard and in this case study, in addition to a location represents the seismic conditions of Iraq, Istanbul is selected as a location for high earthquake effects. The scope of the work and the purposes of this study are summarized below:

1- Applying the Istanbul Seismic Design Code for Tall Buildings (ISDCTB-2008) requirements in analysis and design a model of RC high-rise building with height more than 60m in two locations; Iraq (northern part) and Turkey (Istanbul).

2- Check the seismic performance, considering the same soil conditions, and evaluate the damage state for the two cases depending on the required damage indexes.

3- Redesign the case study (if required) depending on the results with comparing the economic side as an index between the first and second design.
4- Etabs 2015, the structural software, will be used as the tool of building analysis and design.

Turkey is fundamentally vulnerable to earthquakes because of its geographical location which sits between two huge tectonic plates, Eurasia and Africa/Arabia, which are inevitably grinding into one another, north to south. The Anatolian plate, on which most of the Turkish landmass lies, is being squeezed westwards towards the Aegean Sea. Periodic movements happen along two main faults, the North Anatolian Fault (NAF) and the East Anatolian Fault (EAF) and Figure 1.1 shows a map of plate boundaries affecting Turkey. For that reason, an accurate and detailed study is required to achieve the people's aspirations for safe life as much as possible and to avoid that vast amount of loss and damage that happened in the past years.

For each country, the seismic hazard is described by a map of seismic zones defined by the National Authorities depending on the local hazard as shown in Chapter 4 and summarized in Figure 1.2, which shows a map of seismic hazard for Turkey and Middle East from the Global Seismic Hazard Assessment Program (GSHAP) in terms of peak ground acceleration with a 10% probability of exceedance (or a 90% chance of not being exceeded) within the next 50 years.

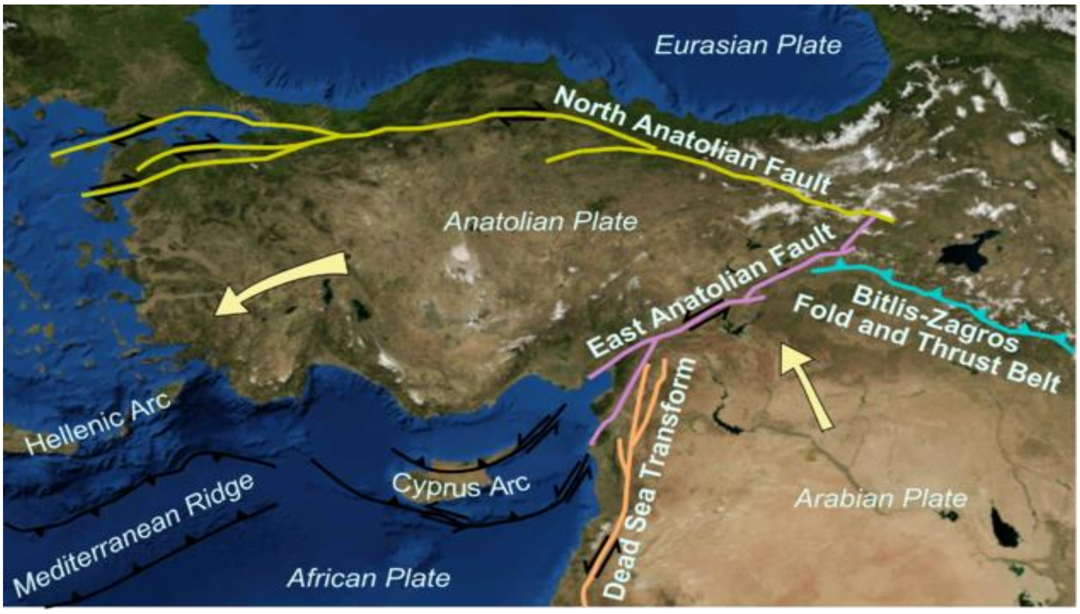


Figure 1.1: Map of plate boundaries affecting Turkey (www.wikipedia.org).

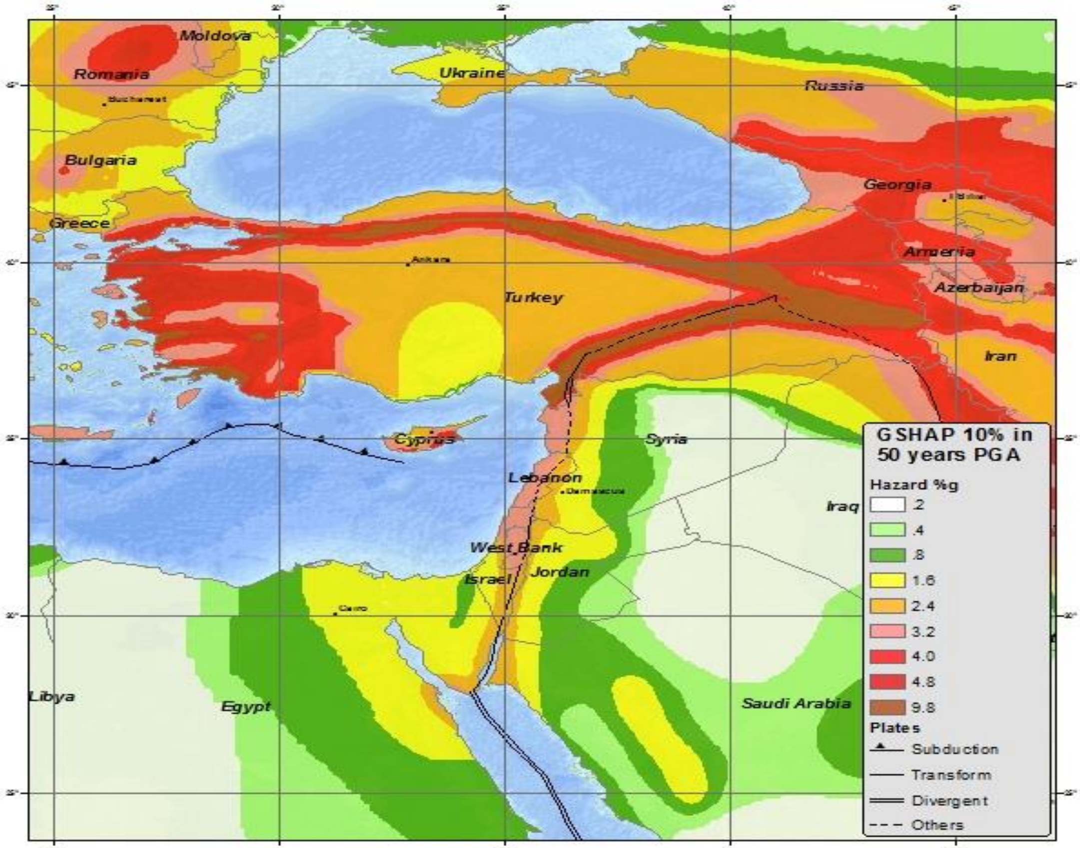


Figure 1.2: Map of seismic hazard for Turkey and the Middle East from the Global Seismic Hazard Assessment Program (GSHAP) in terms of peak ground acceleration with a 10% probability of exceedance (or a 90% chance of not being exceeded) within the next 50 years (www.wikipedia.org).

1.3 Overview of The Previous Literature:

After the publishing of Turkish Seismic Code (TEC-2007), a lot of studies on the seismic performance of RC buildings, particularly for low and mid-rise buildings, were conducted based on it in last years. However, a few studies concerned with the seismic performance of high-rise RC buildings designed based on the draft version of Istanbul Seismic Design Code for Tall Buildings (ISDCTB-2008), such as studies by (Başot, 2010), Binzet et al. (2014) etc. On the other hand, regarding the high-rise RC buildings, there are a lot of cases studied for evaluating the seismic performance especially in the active seismic zones around the world. For instance; in China (Jun et al. (2008), Meng et al. (2014) and Wu and Ou (2015)), in Japan (Nakai et al. (2012)), in Chile (Lagos et al. (2012)), in Macedonia (Apostolska et al. (2016)) and in India

(Azam et al. (2013)). Some of the mentioned studies above are briefly discussed in the following paragraphs.

A very interesting study, which is conducted by (Başot, 2010), discussed the comparison of nonlinear dynamic analysis results for an RC high-rise building considering the regulations and the conditions of TEC-2007 and ISDCTB-2008. A study is applied on Barbaros Plaza Building (A 25-story RC high-rise building) which is located in Istanbul and designed according to 1975 seismic code. This study mentioned that the major difference between the two codes is indicated to be the design spectrum which is defined in the codes. TEC-2007 predicts a design earthquake with a probability of exceeding 10% in 50 years, while ISDCTB-2008 design earthquake is defined for 3 different earthquake levels, D1, D2 and D3 and earthquake with 10% chance of exceeding in 50 years is classified as D2 earthquake level. The spectrum defined according to TEC-2007 principles is between D3 earthquake level and D2 earthquake level spectra that defined in ISDCTB-2008 principles. The nonlinear dynamic analysis is applied to the building by using SAP2000 and the results showed that there is a difference in the seismic performance of the building due to the difference of determination of the damage limits in these two codes. In TEC-2007 principles, building performance level is at collapsing region. The most important reason for this is the plastic joints formed at some elements. According to (Başot, 2010), ISDCTB-2008 defines the structural damage limits, but the damage state is not clearly indicated, therefore it is unclear to what extent the draft version can be applied in Istanbul for high-rise buildings.

Binzet et al. (2014) described the seismic performance of a high rise building based on the criteria defined in Istanbul Seismic Design Code for Tall Buildings (ISDCTB-2008). In his study, the Renaissance Tower, a 43 storeys office type building located in the Asian side of Istanbul with 195 m total height is analysed and designed based on the performance-based design approach. The structural system of this building is divided into gravity system, for gravitational action, and lateral load resisting system, for seismic action, which is composed of reinforced concrete core and buckling restrained braces (BRB). The BRB system connected to the core in one direction and it’s employed to control the lateral displacement demands due to seismic actions.

A seismic hazard analysis has been performed by involving involves the site specific response spectrum for two levels as MCE level with 2% exceedance in 50 year and ground motion time history data sets are prepared in both directions to be used in nonlinear time history analysis in order to determine the deformation demands in terms of concrete and steel strains which are compared with the limits defined in ISDCTB-2008. A series of nonlinear time history analysis has been performed using the seven earthquake ground motion pairs provided in the seismic and Perform 3D software is used to apply the performance-based design and analysis of the Renaissance Tower. Binzet et al. (2014) indicated to some guidelines of ISDCTB-2008 including the proposed analysis procedure which is divided into 4 stages. Firstly the preliminary design which is depending on a linear response spectrum performed in order to determine the structural system in terms of stiffness and strength. Secondly, nonlinear time history analysis is performed based on nonlinear properties of the members determined in the first stage. The following stages are performed as a verification of the preliminary design based on the deformation based acceptance criteria. The results of the analysis showed that the building will suffer minimum damage under design earthquake and after a minimum time of recovery, it will be functional.

In a similar way, (Özuygur, 2015) discussed the performance-based design of an irregular tall building according to the draft version of ISDCTB-2008. A residential building, which is located in Istanbul, has 50 stories with a total height of 198m. The lateral load resisting system consists of shear walls with beams distributed in the floor and the building has an irregular structural plan which is not suitable for high rise building. The building has preliminary design by elastic response spectrum analysis and nonlinear time history analyses performed by using Perform 3D software to check the performance of the building in accordance with the regulations of ISDCTB-2008. After design and analysis the building, the conclusion indicated that more shear reinforcement is needed and the axial forces at the outer boundary of shear walls were larger than that obtained from the preliminary design due to the irregularity of the floor of the building.

In Skopje, a case study by Apostolska et al. (2016) was performed to design and evaluate the seismic performance of one of the highest buildings in Macedonia, in Cevahir Sky City complex, with 44 stories and total height of 134.5m and this case is considered a unique experience and beginning of new era in construction of high rise buildings in this region. This building is located in a very active seismic zone and the structural system is designed as a system of RC shear walls. The analysis, design and evaluating the seismic resistant of the structure were carried out basically according to EN1992 and EN1998 as well as the recommendations and procedures for design of high rise buildings of Los Angeles Tall Buildings Structural Design Council (LATBSDC), 2008 edition and Tall Building Initiative: Guidelines for Performance-Based Seismic Design of Tall Buildings, (PEER Report 2010/05). The analysis procedure was performed within three phases: the first phase included the linear (elastic) analysis of the structure by using ETABS software, in the second phase, a nonlinear pushover static analysis was carried out by using SAP2000 and finally nonlinear dynamic analyses were performed to evaluate the seismic performance of the structure. The analytical results concluded that the structure exhibits a convenient dynamic behavior, a favorable bearing and deformability capacity for the expected seismic effects in both linear and nonlinear domain. In conclusion, the role of RC structural coupled shear wall systems, in resisting the shear forces and reducing the deformations, is also mentioned and this is due to the big stiffness and strength that they have which make them a preferable and economical solution for stabilization of high-rise buildings up to 50 stories.

Nakai et al. (2012) discussed the performance-based seismic design of RC high-rise building in Japan depending on the conditions of Japanese code that requires evaluation the buildings higher than 60m, based on time history response to calculate forces and deformations in each part of the building to ensure that the adequate performance is achieved, comparing with certain criteria, before approving the project to start construction. This study was applied on RC residential building with a height of 100m and the structural system contained symmetrical core walls, with super beams for connection in the center which make the seismic forces from any direction concentrated at the center of the plan. An oil damper is provided, based on top of the super beams and four connecting columns, for absorbing energy and controlling the flexural deformations. The input earthquakes in the seismic response analysis, based on past recorded ground motions, were of two levels as defined in the Japanese building code: Rare Earthquakes and Extremely Rare Earthquakes. The analysis results showed that: (1) the response of the building, which represented by some damage indexes, was within the design criteria and (2) sufficient energy absorption due to the technique in using the oil dampers.

A study by Azam et al. (2013) compared the seismic performance of multi-storeyed RC buildings with structural system of moment resisting frame and different arrangement of shear walls and indicated to the importance and the role of shear walls in improving the seismic performance of the buildings. This study is applied to a different number of stories with same plan and different numerical models due to the different locations of shear walls and at the end, a comparison of the structural behavior was discussed in terms of stiffness, strength and damping characteristics. Two ways of analysis are carried out: elastic response spectrum and a nonlinear static pushover. The results showed that the lateral strength in taller buildings was influenced by the provision of shear walls, however, the influence was less on the lateral stiffness. The shear walls also have a significant influence on the damping characteristics and the natural period of the system. The results of the study indicted also to the most suitable arrangement of shear walls in the system for leading to better seismic performance which symmetrically placed in the plan forming a core and, to a lesser extent, symmetrically placed on all sides of the plan, considering the same total length of shear walls.

2. GENERAL ISSUES ON TALL BUILDINGS

2.1 Definition of High-Rise Building:

A high-rise building or a tower block, as called in some European countries including the UK, is generally known as a tall building or a structure that serves as a residential or an office building. A tall building term doesn't have internationally a limited definition, in other words, there are no common criteria to determine the building if it is tall or not. Council on Tall Buildings and Urban Habitat (CTBUH) organization defines the tall building depending on the following categories, which are not just about the height of the building or the number of floors, which is considered a poor indicator due to the changing of floor height according to the buildings and their functions:

a) Height Relative to Context

   The height of the building is compared to the context in which the building is constructed. For example, a building with 14-story may not be considered a tall building in a city like Chicago, Dubai or Hong Kong, however, in some European cities, this may be taller than most of the buildings exist there.

b) Proportion

   There are many buildings with slender shapes that give the appearance of a tall building, despite that they are not particularly high. On the contrary, there are some large buildings that are really tall but their floor area excludes them from being classified as a tall building.

c) Tall Building Technologies

   Some buildings contain technologies, such as specific vertical transport technologies, high structural wind bracing, can be classed as a tall building.

Generally in CTBUH, a building with 14 or more stories – or more than 50 meters (165 feet) in height – could be defined as a "tall building". In additional to this definition, CTBUH categorizes two groups of buildings: The first one called ‘‘supertall building’’ for those whose height is taller than 300m and a second one called ‘‘mega tall building’’ if the height of the building exceeds 600m.

According to Los Angeles Tall Buildings Structural Design Council (LATBSDC 2014) and Structural Engineers Association of Northern California (SEAONC2016), the tall building is defined as this with height, h_n, greater than 160 feet. The height, h_n, is the height of level n, which may be taken as the roof of the structure, above the base that is considered at the average level of the ground surface adjacent to the structure. However, the adopted definition of tall buildings in PEER/TBI 2010 is depending on “the unique seismic response characteristics of these buildings, including (1) a fundamental translational period of vibration significantly in excess of 1 s, (2) significant mass participation and lateral response in higher modes of vibration, (3) a seismic force resisting system with a slender aspect ratio such that significant portions of the lateral drift result from axial deformation of walls and/or columns as compared to shearing deformation of the frames and walls.”

A different height criterion is adopted for defining the tall building in Istanbul Seismic Design Code for Tall Buildings (ISDCTB-2008), which determines the tall building as the ones whose heights are taller than 60 meters above the ground surface, but in case of a building taller than 75m, an extra analysis should be applied to assess the performance of the building.

In contrast to the mentioned guidelines, Toronto Tall Building Design Guidelines (TTBDG) defines the tall buildings by referring to the importance of site context where the building is located. The building can be considered tall, if its height taller than ‘‘the width of the adjacent street right-of-way or the wider of two streets if it is located at an intersection.’’

2.2 Performance-Based Design:

In recent years, a significant number of tall buildings have been designed, intensively in urban areas, depending on structural seismic force resisting systems that aren’t compatible with the criteria of the building code which is applicable in that time of their design. In other cases, these buildings complied with the requirements of the valid building code, except in some aspects, either the height limit specified by the building codes for the selected structural system is exceeded or the applied seismic-force-resisting system is not covered by these codes.

Consequently, a performance-based design procedure is used and developed for understanding the seismic behavior of these buildings. In this procedure, the building will be designed for purposed nonlinear response and then the nonlinear structural analysis will be applied to verify the seismic performance and check the capability of the structure to achieve that response when subjected to various levels of earthquakes. This procedure is considered an orientation towards a new method based on a nonlinear deformation-based design which helps the design engineers to overcome the obstacles of the traditional seismic design method which is depending on the linear strength-based design.

PEER guidelines indicated some advantages of using this procedure including ‘‘(a) More reliable attainment of intended seismic performance (b) Reduced construction cost (c) Accommodation of architectural features that may not otherwise be attainable (d) Use of innovative structural systems and materials.’’ Nevertheless, PEER mentioned in section 1.3 some points that designer using this procedure should take into account and the most important one is the background knowledge that engineer should have, in the context of ‘‘ground shaking hazards, structural materials behavior, and nonlinear dynamic structural response and analysis in addition to capacity design principles and detailing of elements to resist cyclic inelastic demands, and assessment of element strength, deformation, and deterioration characteristics under cyclic inelastic loading’’, which are considered qualifications to use this procedure.

The performance-based design procedure is applied by implementing three main stages after the conceptual design, which is done by design engineers and includes a selection of ‘‘the structural systems and materials, their approximate proportions, and the intended primary mechanisms of inelastic behavior.’’ These sequential stages consist of preliminary design and followed by two stages of evaluation; service level evaluation and maximum considered earthquake evaluation. At preliminary design, the designer aims, depending on the capacity design concepts, to ‘‘select a target yielding mechanism that is practical within the ductility limits of available structural components and to target yielding to occur in components that are reliably capable of ductile response. ’’ Starting with capacity design principles at first stage is considered a good point particularly with regard to establishing the required system and element actions to the anticipated building response, while applying these principles may not be desirable in the final stage of design (TBI-PEER, 2010).

Next step is the service level evaluation, by applying an earthquake shaking with a minimum return period of 43 years which means 50% probability of exceedance in 30 years that is represented by linear response spectrum with 2.5% damping. The objective of this step is to anticipate some acceptable damages for structural elements which do not affect the function of the building even if they are not repaired. Nevertheless, achieving the requirements of this level may not result in a sufficient strength for the building when subjected to an earthquake more severe than the service level earthquake shaking, therefore the third step is required (TBI-PEER, 2010).

The maximum considered earthquake evaluation is applied as a last stage in this procedure by implementing nonlinear response history analysis to evaluate the response of the structure against a limited shaking which is represented by ‘‘using appropriate contemporary models for the description of regional seismic sources and ground motion prediction equations ’’ in order to get an adequate strength to avoid collapse (TBI-PEER, 2010).

Aydınoğlu (2014) pointed out some problems and challenges that design engineers face when using this procedure in practice specially ‘‘in the transformation stage from the prescriptive code design to a more non-prescriptive performance-based design.’’ These problems are in view of the fact that there is no consensus on some critical areas by referring to issues that are not united in opinion in the most important and major tall building guidelines in the world. These issues are summarized in the following:

(1) The Minimum Base Shear Requirement:

After the development of the guidelines for tall buildings in the last years, the prescriptive code design requirements are almost completely eliminated except the minimum base shear strength requirement which is kept only in LATBSDC-2008 as a capacity design requirement. However, in TBI-PEER 2010, there is no requirement for the minimum base shear.

(2) Preliminary Design Issues:

In TBI-PEER 2010, the preliminary design stage is treated in details including some recommendations related to ‘‘system configuration, wind effects, limiting building deformations, setbacks and offsets, diaphragm demands, outrigger elements, etc.’’. On the other hand, this stage is applied as capacity design rules in LATBSDC-2008.

(3) Some Structural Design Issues:

There are also some issues that are not agreed upon the common guidelines related to design some cases such as core wall systems with coupling beams or outriggers, podiums and some geotechnical and foundation applications.

2.3 Design Guidelines and Objectives of Tall Buildings in Iraq and Istanbul:

Regarding the purpose of this study which refers to design and analysis of a case study of tall building to be constructed in two different locations; in Iraq (northern part) and in Turkey (Istanbul), in other words, under two different seismic hazards, the design procedure used in these two different regions should be clarified. In fact, due to the classification of Iraq as a low seismicity area, there is no own seismic code and there are general recommendations on building construction, which is in fact based on Uniform Building Code (UBC). Therefore major projects including tall buildings are designed and constructed depending on code that the design company or office follows and in most cases these companies are foreign. Nowadays a lot of Turkish companies are working there; for that reason applying the Turkish design codes are also feasible. Furthermore, it will give the opportunity to make a comparison with the same procedure and guidelines that will be followed in these two areas.

As mentioned before, a draft version of “Istanbul Seismic Design Code for Tall Buildings-ISDCTB-2008” has been introduced to design engineering profession in Istanbul and this code is based on performance based design approach which is mentioned in PEER. Although there is a ‘‘ Tall Structures’’ chapter in the draft version of the Turkish Earthquake Code-2016, due to the language barriers and time limitations, ISDCTB-2008 will be considered here in this research, which is very similar.

The basic idea of this approach is to study the nonlinear behaviour of the buildings when subjected to different level of earthquake through a set of acceptance criteria which is defined in ISDCTB-2008 in terms of deformation and force requests on each structural element individually and the analysis and design criteria is presented by two terms; deformations for ductile response and strength for brittle response modes. According to recommendations of ISDCTB-2008, evaluating the performance of the building under various levels of earthquake ground motion is conducted by estimating the magnitude of the damage that will occur in each structural element and represented by the nonlinear deformations (the stage beyond the elastic strain limits) then comparing this magnitude with the acceptable damage limits whether it exceeds or not. Notwithstanding that the performance-based design approach is based on nonlinear analysis methods, linear analysis methods are allowed in ISDCTB-2008 particularly in determining the performance objectives by applying strength-based design approach. Other common damage indices such as story drifts are also used for evaluating the overall performance of the structure as well for the damage state of non-structural elements. The performance objective of the structure is defined in ISDCTB-2008 under the maximum considered earthquake (the last design step of performance-based design procedure) as a collapse prevention performance level. The seismic design which is mentioned in ISDCTB-2008, is defined for three different earthquake levels: D1 for earthquake with 50% probability of exceedance in 50 years and return period of 72 years, D2 for earthquake with 10% probability of exceedance in 50 years and return period of 475 years and D3 for earthquake with 2% probability of exceedance in 50 years and return period of 2475 years. The minimum performance objectives of tall buildings are identified in Table 2.1 below according to the estimated damage that occurs under these three different earthquake levels (D1, D2 and D3). Figure 2.1 shows the performance levels and their regions which are classified as; Minimum Damage region (immediate occupancy performance region) which places below MH/KK line and defines the absence of any damage or a very limited damage to tall buildings and its structural and non-structural elements under earthquake shakings effect, Controlled Damage (life safety performance region) which places between MH/KK and KH/CG lines and describes the performance of the tall building when experiences a limited and repairable damage in its structural and non-structural elements due to earthquake effect, Extensive Damage region (collapse prevention performance region) within the range between KH/CG and İH/GG lines which represents a further widespread damage in the tall buildings and in the elements forming them under the effect of earthquake which occurred prior to the collapse stage and finally Collapse region which locates after İH/GG limit.

Table 2.1: The minimum performance objectives of tall buildings according to ISDCTB-2008.

Building Occupancy Class	(D1) Earthquake Level	(D2) Earthquake Level	(D3) Earthquake Level
Special class buildings: Health, education, public administration buildings, etc.	---	MH/KK	KH/CG
Normal class buildings: Housing, hotel, office buildings, etc.	MH/KK	KH/CG	İH/GG

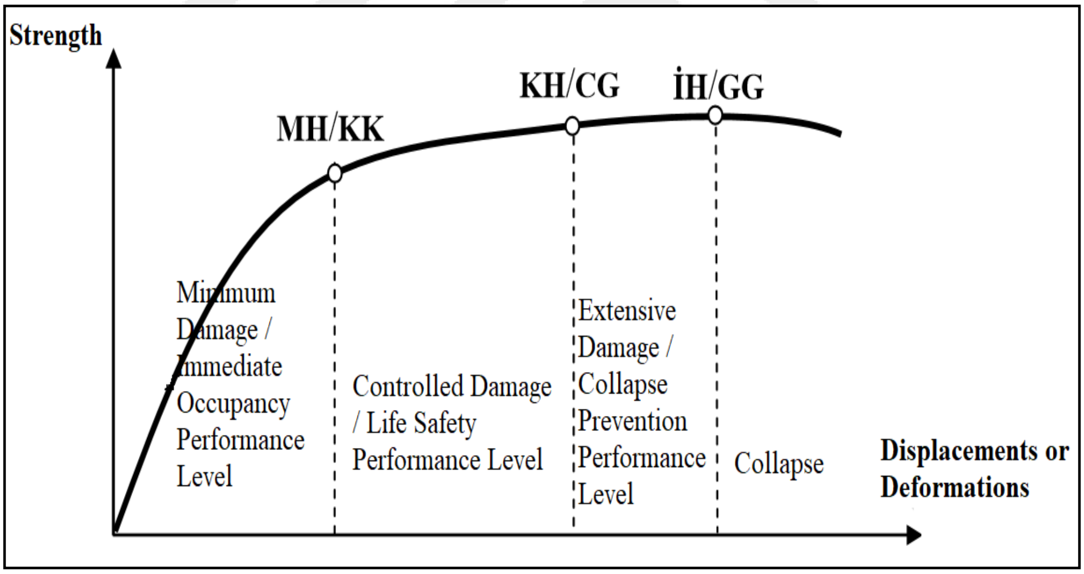


Figure 2.1: Performance levels and regions according to ISDCTB-2008.

In ISDCTB-2008 document, the design spectrum of the earthquake is portioned into five regions, as shown in Figure 2.2, in contrast to that in TEC-2007, which is divided into three regions:

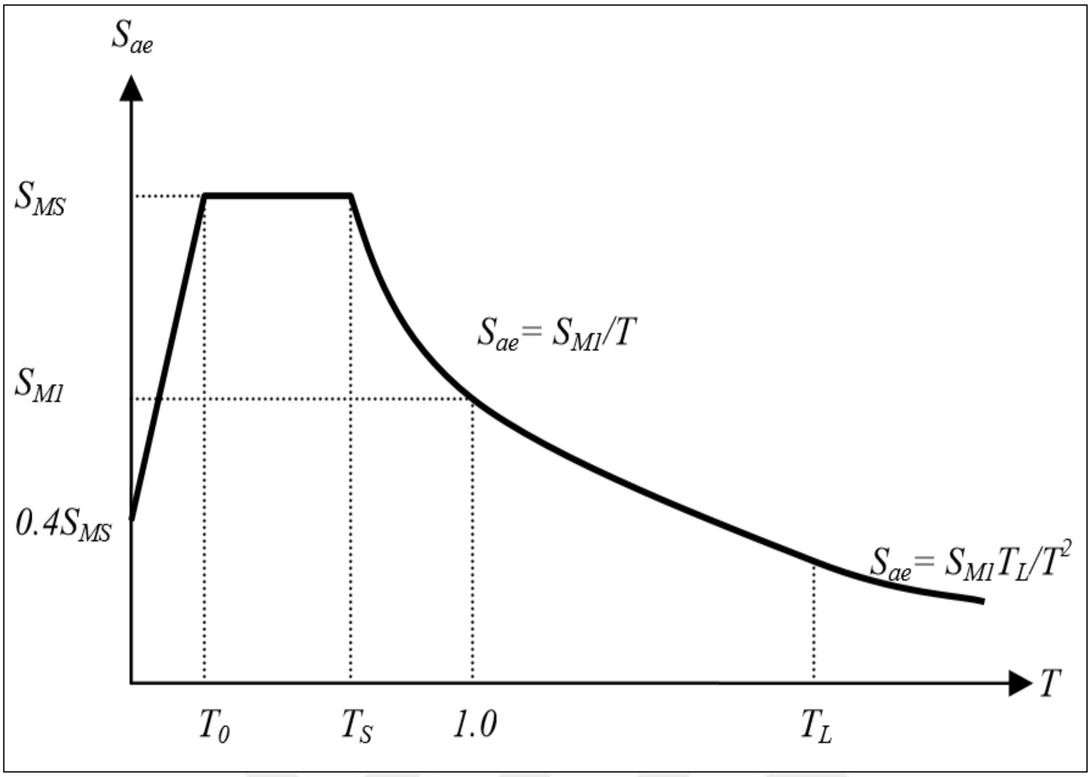


Figure (2.2): Earthquake design spectrum as defined in ISDCTB-2008.

The parameters shown in Figure 2.2 are:

S_{M1}: Spectral acceleration at T=1 sec for the considered ground motion class

S_{MS}: Short period spectral acceleration for the considered ground motion class

T: fundamental period of vibration of the structure

T_0: Spectrum corner period, 0.2 S_{M1}/ S_{MS}

T_S: Spectrum corner period, S_{M1}/ S_{MS}

T_L: Long-period transition period and this value is taken as 12s for Istanbul.

There are four stages defined in ISDCTB-2008 as the design procedure. The table below shows these stages in terms of the requirements of the design type, earthquake level, performance objective, analysis type, structural system behavior coefficient, story drift ratio limit, section stiffness in RC frame members, material strength and acceptance criteria.

Table 2.2: The stages of the design procedure as defined in ISDCTB-2008 for tall buildings.

Design Stage	Design Stage I-A	Design Stage I-B	Design Stage II	Design Stage III
Design Type	Preliminary Design	Design	Verification	Verification
Earthquake Level	D2 For Normal Class Building D3 For Special Class Building	D2 For Normal Class Building D3 For Special Class Building	D1 For Normal Class Building D2 For Special Class Building	D3 For Special Class Building
Performance Objective	Life Safety	Life Safety	Immediate Occupancy	Collapse Prevention
Analysis Type	Linear Response Spectrum	Nonlinear Time History	Linear Response Spectrum	Nonlinear Time History
Structural System Behaviour Coef.	R≤7	---	R=1.5	---
Story Drift Ratio Limit	2 %	2.5 %	1 %	3.5 %
Section Stiffness In RC Frame Members	Effective Stiffness (from TEC-2007)	Effective Stiffness (moment curvature analysis)	Effective Stiffness (moment curvature analysis)	Effective Stiffness (moment curvature analysis)
Material Strength	Characteristic Strength	Expected Strength	Expected Strength	Expected Strength
Acceptance Criteria	Strength & Story Drift Ratio	Strain & Story Drift Ratio	Strength & Story Drift Ratio	Strain & Story Drift Ratio

The requirements and conditions of these design stages will be discussed in details in next chapter.

3. METHODOLOGY FOR PERFORMANCE EVALUATION

3.1 Analytical Techniques for Performance Evaluation:

Analytical techniques that are used in the design and the analysis of high-rise buildings are defined and explained herein with the focus on the example case study structure. The analysis methods will be conducted according to which design stage the engineers are working on. Two main and general methods that are mentioned in PEER/TBI 2010 and ISDCTB-2008 are considered. First of all, some of ISDCTB-2008's basic points that design engineers should take into consideration are mentioned below:

- For high rise buildings, the Spectral Mode Combination Method shall be used for the linear elastic analysis to be carried out at the design stages defined previously as Design Stage (I-A) and Design Stage (II). A Complete Quadratic Combination (CQC) Rule will be applied to combine contributions of each vibration mode.
- In Mode-Superposition Method, the sufficient number of modes to be taken into account will be determined according to the modal shear forces calculated in each floor for each direction as follows:

  $$V_{xin} = M_{xin}\, S_{aen} \qquad ; \qquad M_{xin} = \Gamma_{xn} \sum_{j=i}^{N} m_j \Phi_{xjn} \tag{3.1}$$

  Here, S_{aen} is the acceleration spectrum of the nth mode; M_{xin} is the effective participating masses calculated for nth mode in the given x-x lateral earthquake direction which is also the same direction of the shear force; m_j is the mass of storey j; Φ_{xjn} is the nth mode shape of jth storey along the x-x direction; N is the total number of storeys and Γ_{xn} is the modal contribution factor for the nth mode shape for the x-x direction lateral earthquake:

  $$\Gamma_{xn} = \frac{\sum_{j=i}^{N} m_j \Phi_{xjn}}{\sum_{j=i}^{N} m_j \Phi_{xjn}^2 + m_j \Phi_{yjn}^2 + m_{\theta j} \Phi_{\theta jn}^2} \tag{3.2}$$

  The above-given assumptions are based on the concept that the building floors will behave as rigid diaphragms in their horizontal planes.

- For the nonlinear analysis of high-rise buildings to be conducted at the design stages defined in Table 2.2 as Design Stage (I-B) and Design Stage (III), Direct Time-Domain Integration Method shall be used.
- For the analysis in the time domain to be carried out, at least seven earthquake ground motion sets, consisting of acceleration records for two horizontal components perpendicular to each other in the horizontal direction, shall be effected simultaneously (in the same direction) in the direction of the principal axes of the structural system. Then the axes of the acceleration records will be rotated by 90º and the analyses are repeated. The design basis earthquake requests will be calculated as the average of the results obtained from these analyses (at least 2 x 7 = 14 analyses). In the selection of these acceleration records for the province of Istanbul, the lateral displacement earthquake source mechanism, $7.0 < M_w < 7.5$ moment magnitude and soil type B or C shall be taken as a basis.
- In case of linear or nonlinear analysis of high-rise buildings, the damping ratio shall be $\xi = 0.05$ at most. In the analysis, it is necessary to consider the second order (P - Δ) effects.
- For the calculation of floor masses in linear or nonlinear dynamic analysis of high-rise buildings, the Live Load Participation Factor, n, is used and will be defined as follows for the incoming cases, however, it will not be greater than 0.30:

  $$\begin{array}{ll} n = 0.01\,(50 - N) & N \leq 40 \\ n = 0.1 & N > 40 \end{array} \qquad (3.3)$$

  Here, as mentioned before, N is the total number of storeys.
- Depending on the characteristics of the structural system, the effect of the earthquake in the vertical direction can also be taken into account when is deemed necessary by the Independent Control Board.

In view of the above, the following analysis methods as defined below should be employed depending on the design stage:

3.1.1 Linear response spectrum analysis:

Three-dimensional mathematical models should be used in conducting the linear response spectrum analysis under two horizontal components of ground motions, which have been previously defined according to the design stage and building class (D2 or D3 level) represented by the linear design spectra. These models should satisfy some requirements representing ‘‘the spatial distribution of mass to an extent adequate for calculation of the significant features of the building’s linear dynamic lateral response’’ in addition to the stiffness of the entire structural system including the primary lateral force resisting system and any additional significant lateral stiffness caused by the vertical load bearing elements and the non-structural components (PEER/TBI, 2010). These intended requirements of the linear and nonlinear modeling will be discussed later in this chapter. At the end of this analysis, acceptable performance of the structure will be evaluated by using the linear response parameters in terms of forces and displacements. In case the linear response parameters exceed the acceptance criteria, the nonlinear time history analysis may be used to illustrate acceptable performance for the structure.

3.1.2 Nonlinear dynamic time history analysis (NDTHA):

This method is considered to be the most realistic way to evaluate the performance of a building subjected to dynamic loads and its calculated parameters inclines to be the ‘‘exact’’ solution. A three-dimensional model of the structure will also be used for performing this methodology and this model should represent all forces and deformations caused by structural and non-structural components and remarkably affect the seismic behavior of the structure. These elements form the lateral load resisting system as well as the elements of the vertical load resisting members, which participate significantly in the lateral stiffness and strength of the entire structural system. In this method, the selection of the ground motions will be in accordance with some recommendations that will be discussed in ground motions paragraph and will be applied as demonstrated above in the basic points.

3.2 Rules and Recommendations of Analysis Models:

ISDCTB-2008 document indicated to some regulations that the required model of the structure should achieve to represent the performance of the structure ideally as follows:

- In linear analysis, the modeling of the frame elements will be done with linear finite elements method. However in nonlinear analysis, within the framework of the lumped plasticity approach, the plastic hinges defined by the finite elements method or within the framework of the spread plasticity approach, fiber elements can be used. Plastic hinge's length can be selected from relevant literature and from an appropriate empirical relationship adopted by the Independent Control Board. Alternative modeling approaches, approved by the Independent Control Board, can also be used in the nonlinear analysis. In the linear and nonlinear models of steel frames, the shear deformations in the column-beam joints must be properly considered.
- Modeling of reinforced concrete shear walls and their elements will be carried out by utilizing shell elements in the linear analysis. In order to be compatible with the effective bending stiffness of the cracked sections of the finite elements, the empirical relations given in TEC-2007 (7.4.13) may also be used to reduce the modulus of elasticity (E) of shell elements and define an effective bending rigidities as follow:

  (a) For beams: $(EI)e = 0.40\ (EI)o$

  (b) For columns and frames: $(EI)_e = 0.40\ (EI)_o$ if $N_D / (A_c\ f_{cm}) \leq 0.10$

  $(EI)_e = 0.80\ (EI)_o$ if $N_D / (A_c\ f_{cm}) \geq 0.40$
- In nonlinear analysis within the framework of the spread plasticity approach, fiber elements or other alternative modeling approaches approved by the Independent Control Board can be used to the model of reinforced concrete walls and their elements. The cross-sectional requirements defined in TEC-2007 (3.6.1) may not apply to reinforced concrete walls of buildings over 75 m in height. The shear stiffness of reinforced concrete shear walls must be appropriately taken into account.

- The effective bending stiffness of the cracked sections will be used in structural elements, which are idealized as frame elements. In the preliminary design stage, the empirical relations mentioned above can be used. In the other subsequent design stages, the effective bending stiffness will be obtained from the moment-curvature relationship of the section by the following equation and as illustrated in Figure 3.1:

  $$(EI)_e = \frac{M_Y}{\Phi_y'} = \frac{M_N}{\Phi_y} \qquad (3.4)$$

  Here M_Y refers to the first yielding in the cross section; the corresponding curvature Φ_y' is either the concrete strain reaching the value of 0.002, or the reinforcement strain (whichever occurs earlier). The effective curvature Φ_y, which corresponds to the effective plastic moment M_N, is calculated by taking the concrete compression strain at 0.004 or the steel strain at 0.015 (whichever is earlier). In the calculation of the moment capacities of the columns, only the axial forces due to the vertical loads case can be used.

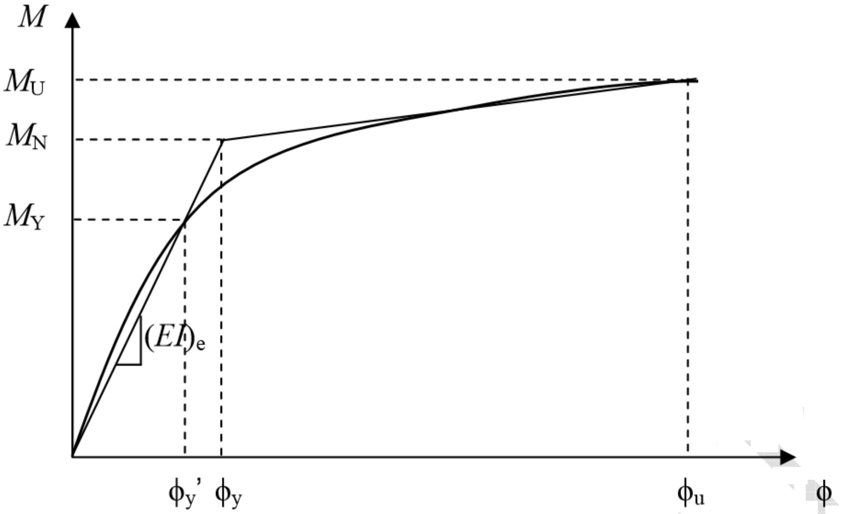


Figure 3.1: The moment- curvature relationship (ISDTB-2008).

- The confined concrete and steel reinforcement behavioural models defined in TEC-2007 Information Annex 7B can be used. The approval of the Independent Control Board is required for the use of concretes with strength higher than C50 in high rise buildings.
- In the preliminary design stage, the design strengths (f_d) of concrete, reinforcing steel and structural steel are defined by dividing the corresponding

characteristic strengths (f_k) by material safety factors. On the other hand, in the other subsequent design stages, these factors shall not be taken into consideration; however “the expected strength - (f_e)’’ will be used. The following relationships between average strength values and characteristic strength values can be taken into account:

For concrete $f_{ce} = 1.3f_{ck}$
For steel reinforcement $f_{ye} = 1.17f_{yk}$
For structural steel (S235) $f_{ye} = 1.5f_{yk}$ (3.5)
For structural steel (S275) $f_{ye} = 1.3f_{yk}$
For structural steel (S355) $f_{ye} = 1.1f_{yk}$

- Plastic hinges in the frame elements represented by a bi-linear backbone curve based on the nonlinear hysteretic relationships. In hysteretic behavior, the effects of the stiffness and strength degradation can be neglected in new constructed high rise buildings.
- Special care should be taken for the "Transfer floors" with sufficient in-plane stiffness and strength, when sudden changes in the horizontal stiffness of the vertical structural system elements (especially downward sudden increases) are present.
- The stiffness of the foundation soil and around the basement will be taken into account by the idealization methods deemed appropriate by the Independent Control Board. If necessary, the nonlinear behavior of the soil-foundation system can be utilized within the mentioned design stages.

3.3 Nonlinear Modeling of RC Components for Tall Buildings:

The nonlinear time-history analysis is the best way for predicting the building behaviour by depending on adequate models to simulate all modes of the structure’s deformation under various levels of ground shaking. To achieve this goal, some aspects related to components modeling should be taken into consideration. First of all, nonlinear models should include the stiffness, strength and deformation characteristics of the structuraaal components through depending on the expected properties of these components and their materials which represent an accurate and impartial measure of the expected response of the structure. Selection of component model type has also very important influence on modeling the elements of the structure and generally there are diverse models of the inelastic structural component which are categorized into groups due to factor called ‘‘degree of idealization’’. Figure 3.2 shows the comparison between three types of nonlinear model that are generally used for simulating the seismic behavior of RC beam-column, namely: continuum finite element model, lumped plasticity model and distributed inelasticity model. The finite elements included in a continuum model represent the nonlinear response of the materials and elements that constitute the member and in case of RC beam-column model, they represent the concrete and its associated actions, crushing, cracking, and dilation and also represent the longitudinal and shear reinforcement and their behaviour in terms of yielding, buckling and fracture as well as the bond transfer between concrete and steel. The continuum models cover the effects of stiffness, strength and deformation capacity of the member through the material properties so there is no need to define them in the model. On the other hand, the concentrated hinge (lumped plasticity) model are defined by the overall force-deformation response of the member and for RC beam-column component, the axial force-moment interaction is presented by taking the behavior and hysteretic test data of these components into consideration by following the inelastic deformation rules. Contrary to the concentrated hinge model, in the fiber model the spread of plasticity through the cross section and along the member is permitted so its called distributed inelasticity model.The most significant difference between these models is the ability of each one to capture the strength degradation effects where concentrated hinge models can capture the strength degradation while continuum and distributed inelasticity models cannot due to their limited ability (ATC-72-1, 2010).

The response features of an elastic hinge model for RC elements are idealized and presented by a backbone curve and hysteretic cyclic model. The backbone curve is a tool and reference to understand the behaviour of nonlinear system by defining the force-deformation (F-δ), and for plastic hinge region moment- rotation (M-θ) relationship limits which confine the hysteretic response of the component. The initial backbone curve is a term refers to the backbone curve with no occurred cyclic deterioration and in this case it is very close to the monotonic loading curve and both terms are interchangeable for practical purposes. Figure 3.3 illustrates the typical initial backbone curve with its parameters and their definitions (ATC-72-1/2010).

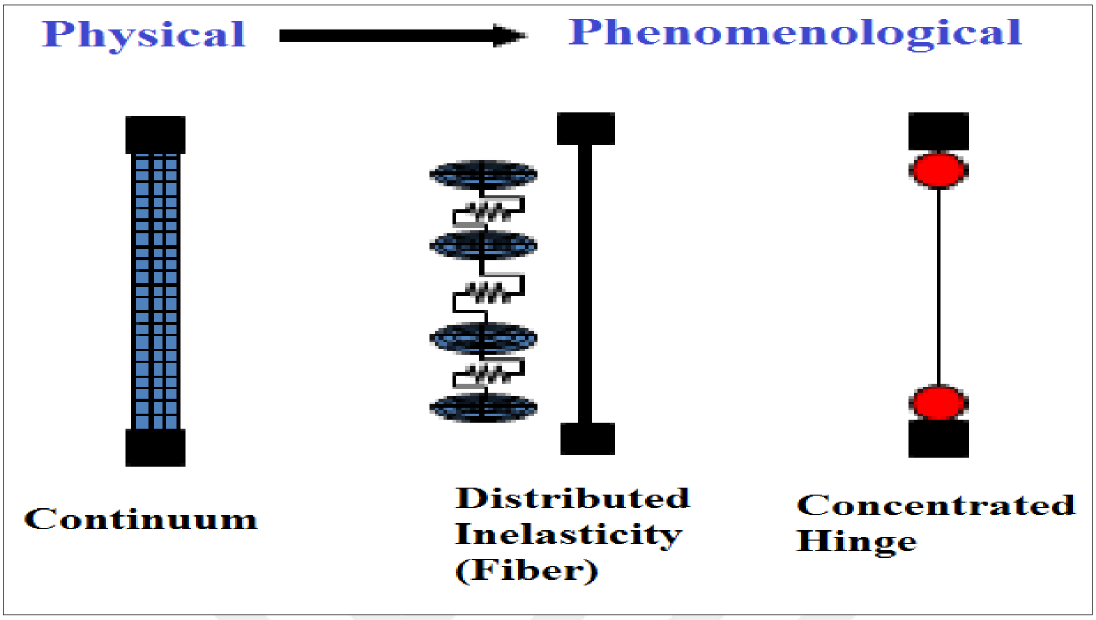
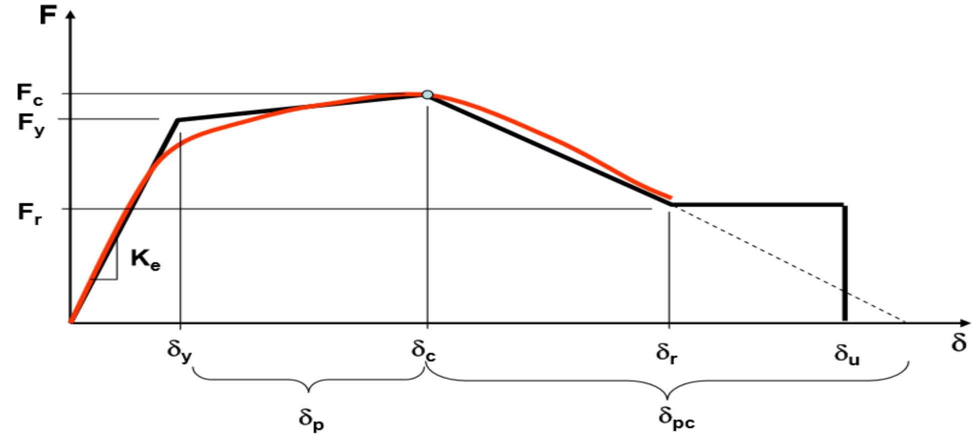


Figure 3.2: Nonlinear structural beam-column model types (ATC-72-1/2010).



- Effective yield strength and deformation (F_y and δ_y)
- Effective elastic stiffness, $K_e = F_y/\delta_y$
- Strength cap and associated deformation for monotonic loading (F_c and δ_c)
- Pre-capping plastic deformation for monotonic loading, δ_p
- Effective post-yield tangent stiffness, $K_p = (F_c\text{-}F_y)/\delta_p$
- Post-capping deformation range, δ_{pc}
- Effective post-capping tangent stiffness, $K_{pc} = F_c/\delta_{pc}$
- Residual strength, $F_r = \kappa F_y$
- Ultimate deformation, δ_u

Figure 3.3: The initial backbone curve of the Ibarra-Krawinkler model and its parameters (ATC-72-1/2010).

Regarding the hyteretic model, an appropriate basic rules should be followed to adapt the model for the deformation mode that represent the dominant cyclic behavior of the component, such as bilinear (with ignoring the stiffness deterioration), peak-oriented, or pinched hysteretic behaviour (with accounting the stiffness deterioration) as shown in Figure 3.4 (ATC-72-1, 2010). These hystertic models are developed over the years by a lot of researchers and Clough Model (1966) is considered the first acceptable model for bilinear hysteretic cyclic behaviour and after that a lot of modifications and refinements are carried out to create more purified models, such as trilinear models, which are developed by Fukada (1969) and the most sophisticated one was by Takeda (1970) and curvilinear models as illustrated in Figure 3.5 of modified Bouc-Wen (Foliente, 1995) model.

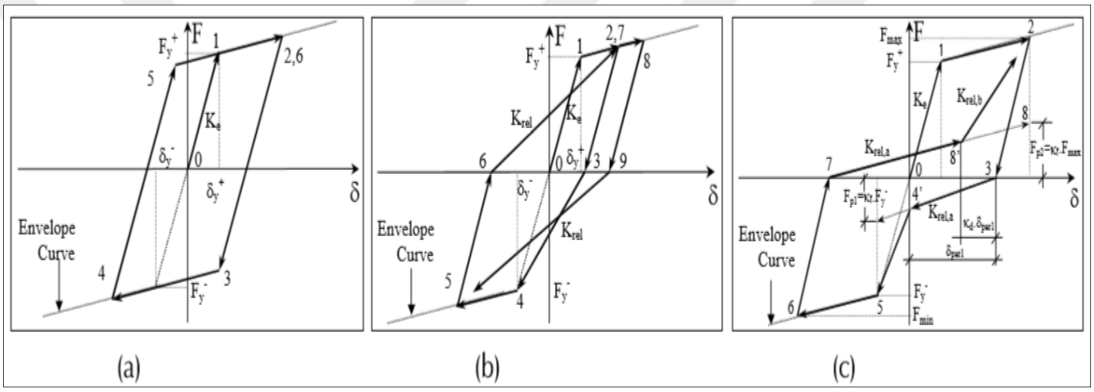


Figure 3.4: The basic rules for hysteretic behaviour models (a) bilinear; (b) peak oriented; and (c) pinched (Medina and Krawinkler, 2003).

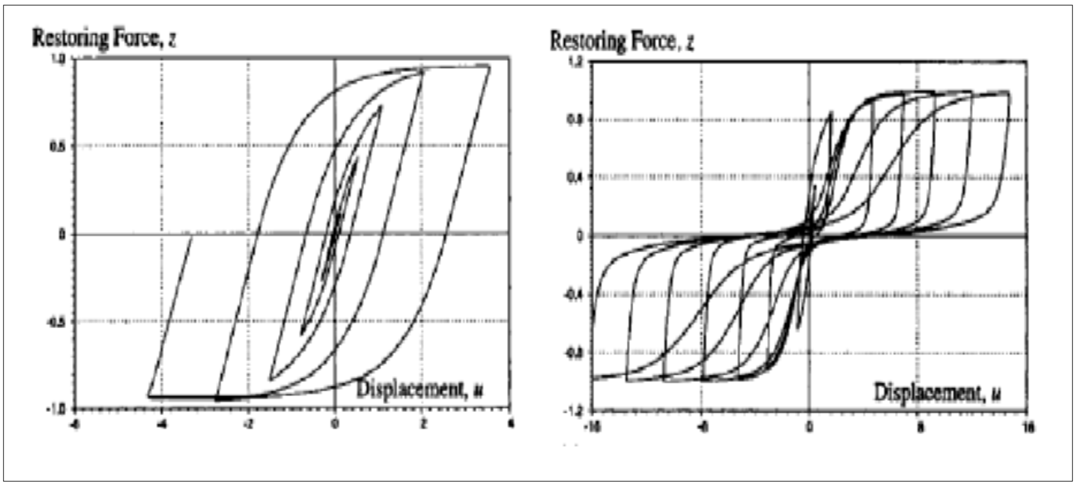


Figure 3.5: A modified Bouc-Wen model (Foliente, 1995).

Using these rules and features for modelling main RC elements in tall building will be discussed and summarized in the following points (more detailed recommendations about modeling components of tall building are discussed in detail in Chapter 2 of ATC 72, 2010).

3.3.1 Nonlinear modeling of RC frame components (beams, columns and beam-column joints):

Modeling of reinforced concrete flexural members; beams or columns, is idealized by a quasi-elastic element, for accounting the concrete cracking, bond slip, and other factors that occur before yielding by the elastic properties of the element, with concentrated inelastic hinges at each end, represented by rotational springs which have backbone and hysteretic cyclic deterioration properties based on results from experimental studies as shown in Figure 3.6 (ATC-72-1, 2010).

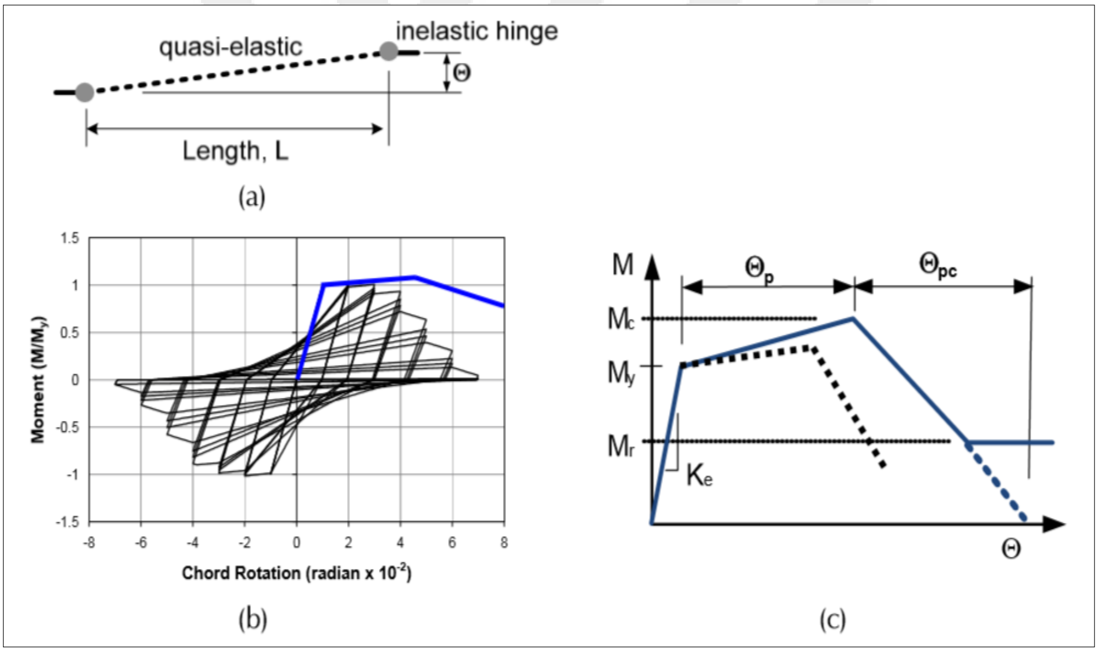


Figure 3.6: RC beam or column (flexural member) modeling: (a) the idealized flexural element; (b) the hysteretic response and monotonic backbone curve and (c) monotonic and modified backbone curves (ATC-72-1/2010).

Nonlinear modeling of beam-column joints is also considered very necessary due to their significant influence in the inelastic response of reinforced concrete beams and columns since the bond-slip and yield penetration occur into this region. The inelastic model of beam-column joint include five inelastic springs, as illustrated in Figure 3.7, four springs refer to the adjacent beam and column connected to the joint and they are modelled for representing the inelastic deformations of the concentrated plastic hinge member and the bond-slip as well as the yield penetration into the joint, same as the springs at each end of the flexural member, and the last spring at the center of the joint which is idealized to model the shear deformations that occur through the joint(ATC-72-1, 2010).

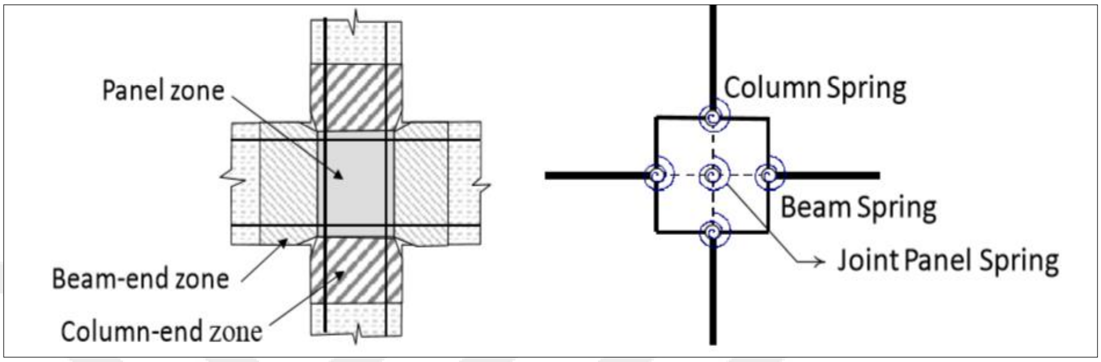


Figure 3.7: RC beam-column joint modeling (ATC-72-1/2010).

3.3.2 Nonlinear modeling of RC shear walls:

Modeling of reinforced concrete shear walls should reasonably include the load versus deformation responses caused by flexure, shear, and bond. A variety of approaches are used in modeling the shear wall, however three main approaches typically are employed in the analysis and design of core wall in tall buildings and they are explained in the following paragraphs (ATC-72-1, 2010).

First approach is called equivalent beam-column element (lumped plasticity) models and, as shown in Figure 3.8, the wall is idealized by elastic column at center of the wall connected to adjacent beams. This approach is not desirable due to shortcomings summarized by disregarding the migration of the neutral axis along the wall cross-section during loading and unloading; the interaction between the adjacent beams and slabs in the plane of the wall and perpendicular to it; the influence on the wall strength and stiffness due to the variation in axial load and inability to model T- or L-shaped cross-sections of shear walls. On the other hand beam-column element models and their parameters related to stiffness and plastic hinge rotation limits can be easily used and assigned and they are considered computationally efficient (ATC-72-1, 2010).

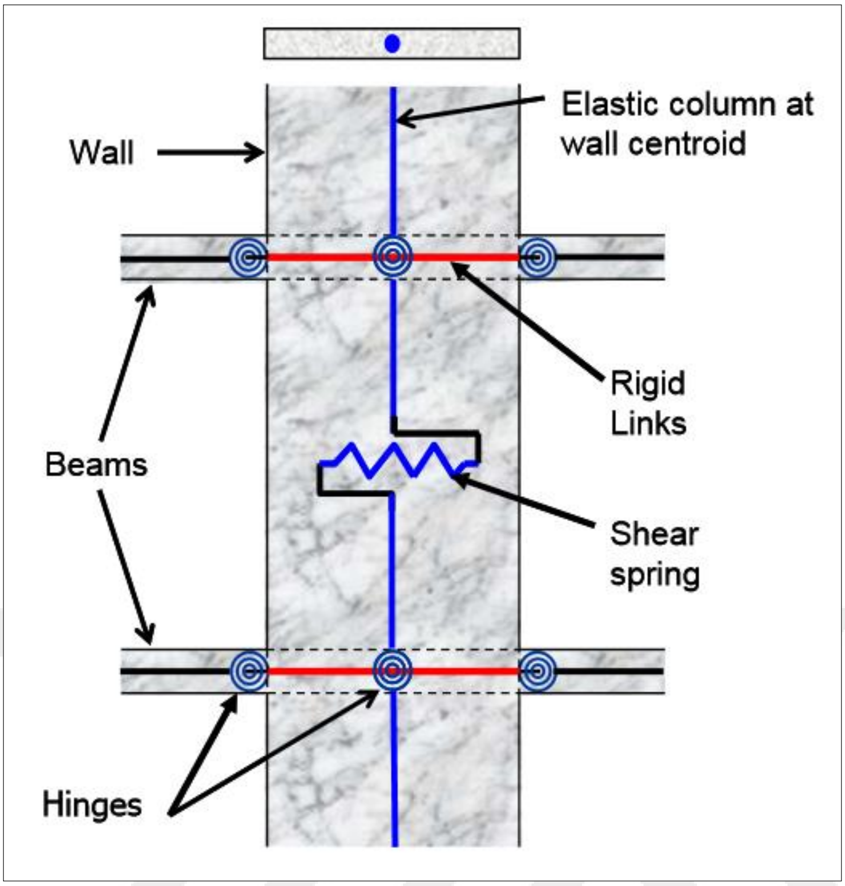


Figure 3.8: Modeling of a RC shear wall by equivalent beam-column element (ATC-72-1/2010).

Fiber (distributed inelasticity) beam-column modeling is considered a very acceptable approach in practice which eliminated the shortcomings regarding the equivalent beam-column models and it has been carried out in different analytical software. This approach include subdividing the wall section into a sufficient number of concrete fibers and steel fibers, along the cross-section, and elements over the height of the wall, as demonstrated in Figure 3.9. However in these models, the impact of selection of material and modeling parameters should be taken into account (ATC-72-1, 2010).

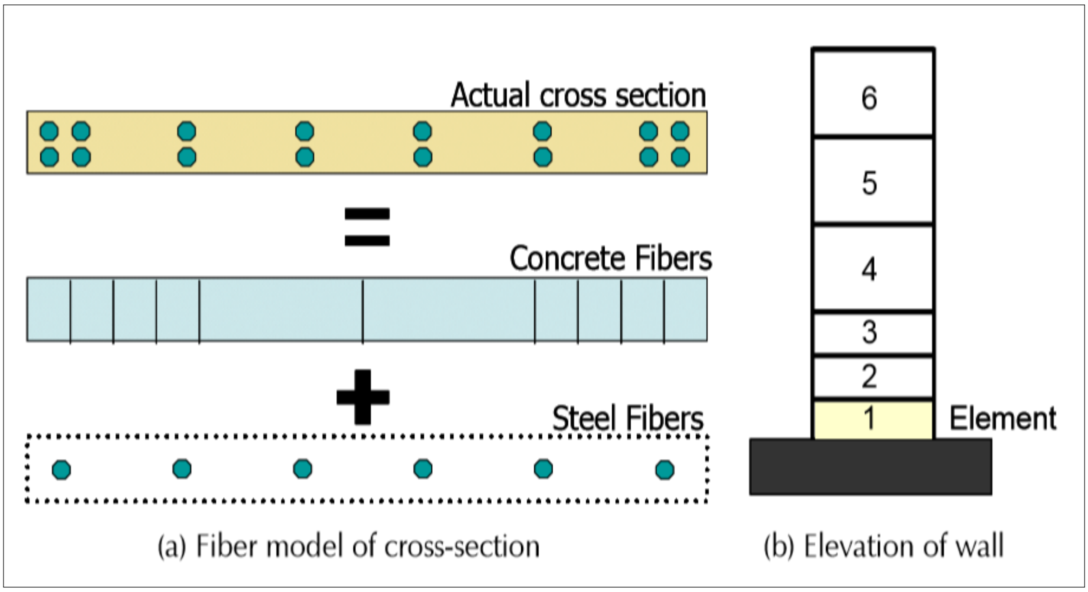


Figure 3.9: Fiber model of RC shear wall (ATC-72-1/2010).

Biaxial fiber models are considered the more sophisticated modeling approaches which are also available in some commercially software. Figure 3.10 illustrates an example of a two-dimensional uniaxial fiber model. According to this model, the yielding is permitted to occur in both directions; horizontal and vertical. Using the biaxial models is available for complicated geometry of structural systems; for instance, core walls and also floor diaphragms that subjected to bidirectional loads (ATC-72-1, 2010).

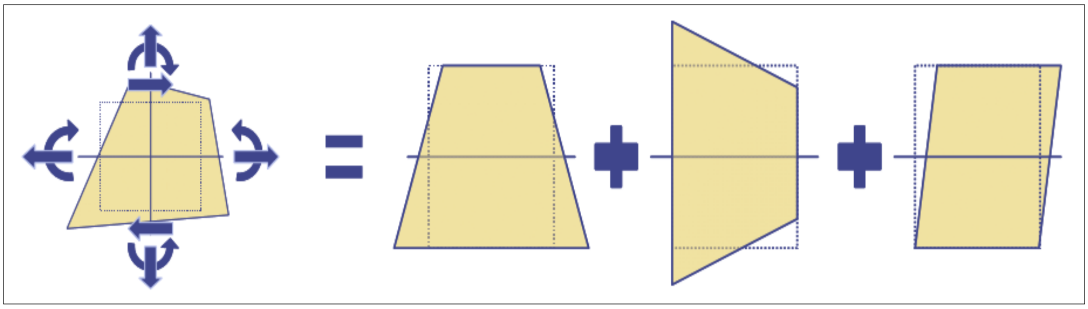


Figure 3.10: Two-dimensional biaxial fiber model (ATC-72-1/2010).

3.4 Ground Motion Selection:

Earthquake is produced by ground motion that occurs below the surface of the earth and releases a stored strain energy in form of seismic waves which cause shaking of the surface of the earth on where the buildings are constructed. The ground motion can be measured by recording one of the following parameters; acceleration, velocity or displacement. Through these parameters, the levels of ground shaking, at which the building is damaged, can be evaluated. For design engineers, it is very necessary to take the characteristics of the ground shaking into consideration due to its effect in controlling the response of the building under the earthquake in addition to the building characteristics. Consequently, instruments called accelerographs are employed to record the acceleration of the ground as a function of time during an earthquake at a location, where the instrument is placed which called station. These records obtained by accelerographs are called accelerograms. Accelerograms are affected by energy released, local soil, type of rupture at the bed rock and geology along the travel path from source to the earth’s surface. For that reason selecting and scaling the appropriate earthquake ground motions are very substantial for design and seismic performance assessment of buildings performing the nonlinear time-history analysis. PEER’s guidelines mentioned three steps to select and modify the records (accelerograms) for structural dynamic analyses, as follows:

(1) Identifying the types of earthquakes that control the ground motion hazard by probabilistic seismic hazard analysis to deaggregate the ground motion hazard for spectral accelerations of the required earthquake level at the structural natural periods of interest, which include minimum the first three translational periods of structural response in each direction of the structure’s principal orthogonal response.

(2) Selecting a minimum of seven accelerograms sets recorded during past earthquakes for response history analysis and they must consist of at least two horizontal components (the vertical component may also be included) and be compatible with the earthquake magnitude and site source conditions.

(3) Modifying those motions to achieve a match with the target spectrum (either the uniform hazard spectrum or conditional mean spectrum), by using spectral matching or direct amplitude scaling procedure. The spectral matching procedure adjusts the frequency content of accelerograms to fit the target response spectrum limits over a specific period of time. In contrast, the direct scaling procedure is applied to increase or decrease the amplitude of an accelerogram by determining a constant scale factor.

For an elastic behavior of the building subjected to earthquake shakings, the duration of these shakings may not make a difference, however, it makes a significant difference for buildings under inelastic conditions, in addition to the effect of peak ground acceleration and the frequency content. Figure 3.11 shows the differences in these features for a collection of ground motions scaled to the same time and amplitude axes.

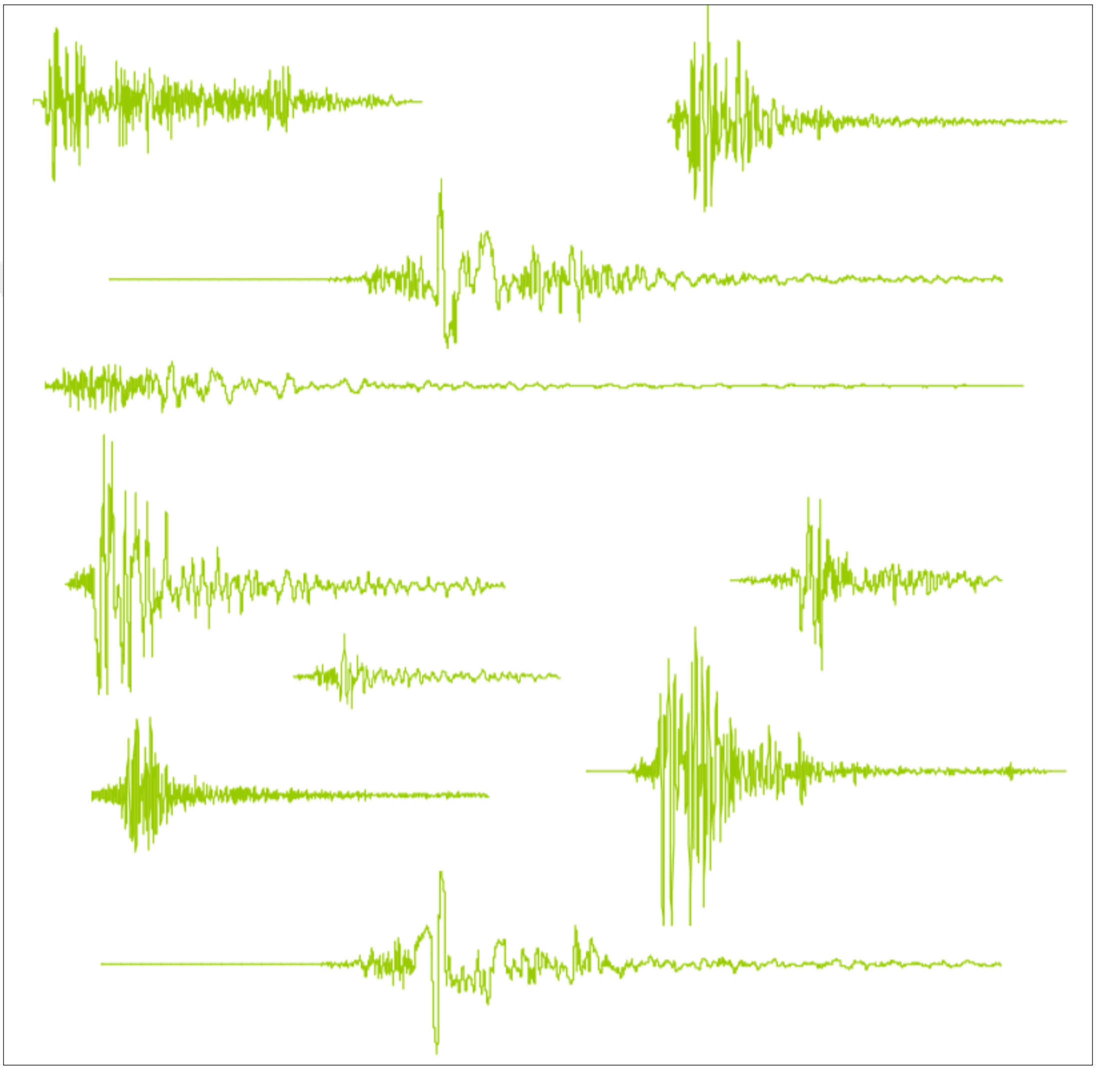


Figure 3.11: Qualitative difference in acceleration time histories of ground motions recorded during past earthquakes worldwide (Murty et al, (2012)).

3.5 Earthquake Design Stages of Tall Buildings According to ISDCTB-2008:

The recommendations and the performance objective criteria of each design stage are illustrated below in details:

3.5.1 Design stage (I-A): Controlled damage / linear analysis for performance objective of life safety:

- The buildings are checked for achieving the performance objective of this preliminary design stage under the effect of D2 earthquake level for normal class building and D3 earthquake level for special class building by using the linear elastic analysis and following the same regulations and rules of TEC-2007 chapters 2, 3 and 4.
- The Seismic Load Reduction Factor should be used to reduce the elastic earthquake loads and it is defined below in terms of the Structural System Behavior Factor (R) and the natural vibration period (T):

$$\begin{aligned} R_a(T) &= 1.5 + (R - 1.5)\frac{T}{T_s} && (0 \leq T \leq T_s) \\ R_a(T) &= R && (T_s < T) \end{aligned} \tag{3.6}$$

  Where T_s is defined previously in earthquake design spectrum.
- There are some conditions related to the building structural system approved by Independent Control Board for defining the Structural System Behavior Factor (R) which can be taken as R=7 at most in this design stage.
- Regardless of the used factor R, the base shear force to be considered in the preliminary design cannot be less than the value calculated by the following relation:

$$V_{t,min} = 0.04\, S_{MS(D2)} W \tag{3.7}$$

  Where $S_{MS(D2)}$ is the short period spectral acceleration defined for earthquake at D2 level, W represents the weight of the building.
- Additional eccentricity effects shall be taken into account as defined in TEC-2007 ‘‘by shifting the actual mass center by +5 % and −5 % of the floor length in perpendicular direction to the earthquake direction and to the mass center as an additional load.’’

- The internal forces of elements in the direction of principle axis a and b shall be calculated according to the following equations of TEC-2007:

$$B_a = \pm B_{ax} \pm 0.3B_{ay} \quad or \quad B_a = \pm 0.3B_{ax} \pm B_{ay}$$
$$B_b = \pm B_{bx} \pm 0.3B_{by} \quad or \quad B_b = \pm 0.3B_{bx} \pm B_{by} \tag{3.8}$$

- In each direction, the maximum effective relative story drifts of vertical bearing elements, columns or structural walls, shall be calculated according to TEC-2007 (2.10.1.) and satisfied the condition of this design phase as following:

$$\frac{(\delta_i)_{max}}{h_i} \leq 0.02 \tag{3.9}$$

Where:

h_i: The height of the i^{th} storey of the building.

$(\delta_i)_{max}$: The maximum value of effective relative story drift of the i^{th} storey which is obtained by applying the following equation:

$$\delta_i = R\Delta_i \qquad : \qquad \Delta_i = d_i - d_{i-1}$$

Δ_i: The reduced relative storey drift

$d_i \; and \; d_{i-1}$: The lateral displacements obtained from the analysis at the ends of any column or structural wall at stories i and (i - 1).

- For high ductility level of all RC members, the requirements of the minimum values for systems defined in TEC-2007 (Chapter 3) should be achieved.
- The capacity design principles given in Chapter 3 and / or Chapter 4 of TEC-2007 shall be applied in the same way for the shear safety of columns and beams.
- The dynamic magnification coefficient β_v, placed in the following equation of design shear force for structural walls defined in TEC-2007, should be taken as $\beta_v = 2$ for shear safety:

$$V_e = \beta_v \frac{(M_p)_t}{(M_d)_t} V_d \tag{3.10}$$

3.5.2 Design stage (I-B): Controlled damage / non-linear analysis for performance objective of life safety:

- More than 75m of height and after determining the preliminarily dimensions of the structural system of the tall building according to the first stage (preliminary design stage) under the effect of D2 earthquake level for normal class building and D3 earthquake level for special class building, the nonlinear analysis is performed in this stage according to the rules and conditions mentioned in 3.2 to achieve the life safety performance level under effects of same levels of earthquake. Additional eccentricity effects may not be considered in this analysis.

- As mentioned previously, the earthquake requests are obtained as an averages of the results calculated from at least 2 * 7 = 14 analysis and these results will be compared to the capacities defined below:
  - (a) The relative storey drift ratio (ratio of relative storey drift to the storey height) of each structural vertical load-bearing element for each storey should not exceed 0.025.
  - (b) For reinforced concrete sections which satisfy the conditions of transverse reinforcement defined in TEC-2007, the upper limits of the compression strain of the concrete of the outermost fiber inside the transverse reinforcement region and the steel reinforcement strain are given below:

$$\varepsilon_{cg} = 0.0135 \quad ; \quad \varepsilon_s = 0.04 \tag{3.11}$$

  - (c) Strain capacities of steel bar elements will be taken from Seismic Rehabilitation of Existing Buildings recommendations published by American Society of Civil Engineers, ASCE/SEI 41-06, for life safety performance level.
  - (d) The shear strength capacities of reinforced concrete structural members shall be calculated according to TEC-2007 based on the average strengths defined in 3.2 by (3.5) relations.
  - (e) If any of the previous conditions from (a) to (d) cannot be achieved, all design steps will be repeated with necessary changes in the load-bearing system.

3.5.3 Design stage (II): Minimum damage / verification by linear analysis for performance objective of immediate occupancy:

- After finishing the previous design stages, the minimum damage level of the building (immediate occupancy performance level) will be achieved in this stage under the effect of D1 earthquake level for normal class building and D2 earthquake level for special class building by applying linear analysis according to rules and conditions determined in 3.2. Additional eccentricity effects may not be considered in this analysis. The building design will be completed in this phase for buildings which are not more than 75 m in height.

- The internal forces of elements in the direction of principle axis a and b will be calculated according to equations 3.8.
- The principal internal forces will be obtained by dividing the internal forces obtained from the linear elastic analysis by Ra = 1.5 coefficient regardless the type of the load-bearing system. These forces shall not exceed the section bearing forces calculated on the basis of the average strengths defined by (3.5) relations.
- The relative storey drift ratio of each structural vertical load-bearing element for each storey should not exceed value of 0.01.
- If the last two conditions or one of them are not met, all design steps shall be repeated with necessary changes in the load-bearing system.

3.5.4 Design stage (III): Advanced damage / verification by nonlinear analysis for performance objective of collapse prevention:

- In this stage, the advanced damage level of the building (collapse prevention performance level) will be achieved under the effect of D3 earthquake level by using nonlinear analysis according to rules and conditions determined in 3.2. Additional eccentricity effects may not be considered in this analysis.
- The earthquake requests obtained as an averages of the results calculated from at least 2 * 7 = 14 analysis will be compared to the capacities described below:
  - (a) The relative storey drift ratio of each structural vertical load-bearing element for each storey should not exceed value of 0.035.
  - (b) For reinforced concrete sections which satisfy the conditions of transverse reinforcement defined in TEC-2007, the upper limits of the compression strain of the concrete of the outermost fiber inside the transverse reinforcement region and the steel reinforcement strain are given below:

$$\varepsilon_{cg} = 0.018 \quad ; \quad \varepsilon_s = 0.06 \tag{3.12}$$

(c) Strain capacities of steel bar elements will be taken from ASCE/SEI 41-06 for collapse prevention performance level.

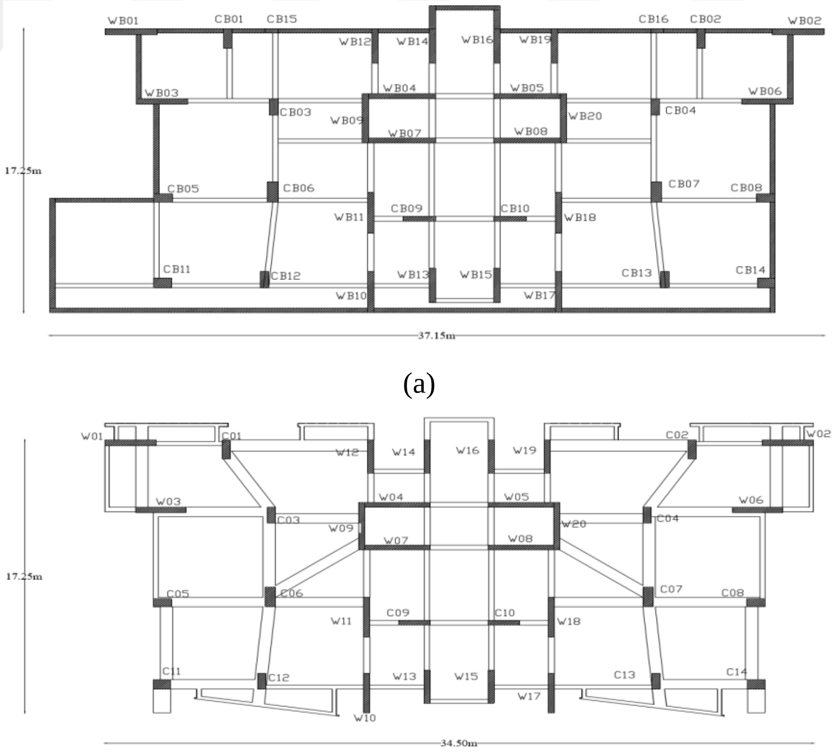
(d) The shear strength capacities of reinforced concrete structural members shall be calculated according to TEC-2007 based on the average strengths defined in 3.2 by (3.5) relations.

(e) If any of the previous conditions from (a) to (d) cannot be achieved, all design steps will be repeated with necessary changes in the load-bearing system.

4. CASE STUDY

4.1 Description of the Case Study Building

The case study is a twenty five-storey reinforced concrete tall building, including one basement, with a height of 3 m for each storey. The basement has different plan due to the external walls, which surrounds the building under the ground level, however, the typical floor plan is considered almost-symmetrical with dimensions of 17.25m x 34.5m. The plans of the building and the 3D view of the building model are shown in Figure 4.1 and Figure 4.2 respectively. This building is a real office project which is constructed in Turkey, in seismic zone 3 with conditions similar to selected location in northern part of Iraq and will be checked as if it is located in high seismicity region, zone 1, which is selected to be Istanbul for this study with same soil conditions.



(a)

(b)

Figure 4.1: (a) The basement plan and (b) the typical floor of the building.

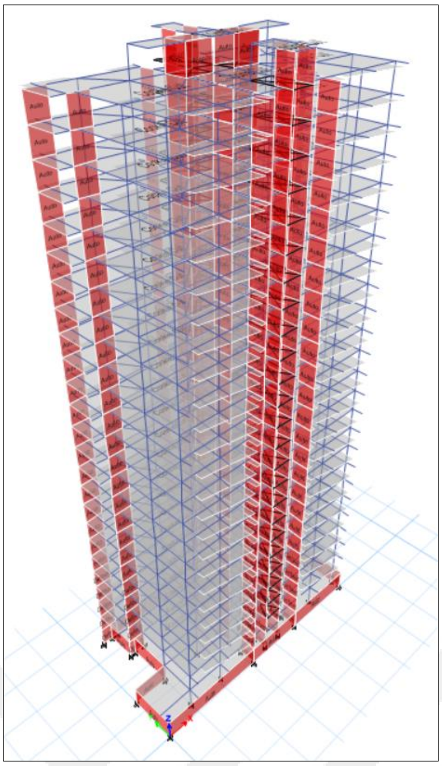


Figure 4.2: 3D model of the building in ETABS.

The properties selected during the modeling and design stage for both cases are listed in Table 4.1below. The linear elastic spectrum for each case is created according to values of S_S, S_1 and their design values for D2 earthquake which are defined in last draft version of Turkish Seismic Code (2016) and presented in Figure 4.3 for damping ratio of 5%.

Table 4.1: The selected properties of the case study in each location.

The design properties	Istanbul building	Iraq building
Seismic Zone	Zone 1	Zone 3
Soil Type (Site Coeffs.)	Type B (F_a=1, F_v=1)	Type B (F_a=1, F_v=1)
Building Importance Factor	1	1
Structural System Behavior Factor, R	7	7
Live Load Participation Factor	0.25	0.25

Table 4.1 (continued): The selected properties of the case study in each location.

The design properties	Istanbul building	Iraq building
S_S (S_{DS})	1.217 (1.460)	0.308 (0.400)
S_1 (S_{D1})	0.335 (0.502)	0.089 (0.134)
Strength of concrete (C30), f_{ck} (MPa)	30	30
Strength of reinforcement steel (S420), f_{yk} (MPa)	420	420

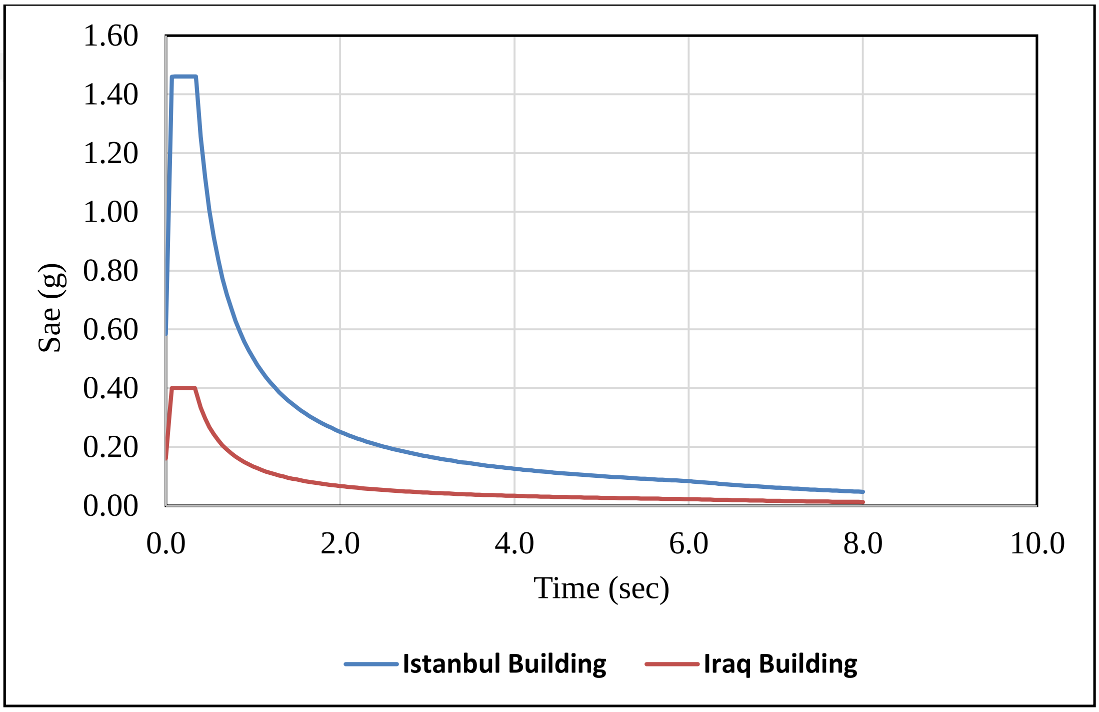


Figure 4.3: The linear elastic spectrum for each case.

The system consists of ribbed slabs based on a dual system comprised of shear walls and lateral resisting frames. The columns and shear walls have been labeled, as shown in Figure 4.1, from C01 to C14 (in addition to two columns in the basement; CB15 and CB16) and W01 to W20 respectively. The columns and shear walls dimensions are constant throughout the height of the building and they are presented in Table 4.2. Structural system and reinforcement details of the case study are described in Appendix B.

Table 4.2: The section dimensions of columns and shear walls.

Section labels	Dimensions, mm
C01/C02	400 x 1200
C03/C04/ C12/C13	400 x 1000
C05/C08	900 x 500
C06/C07	500 x 1250
C09/C10	1550 x 300
C11/C14	850 x 600
W01/W02	2500 x 350
W03/W06	2450 x 350
W04/W05/ W07/W08	3500 x 300
W09/W20	300 x 3000
W10/W17/ W11/W18	300 x 2500
W12/W13/W14/W15/W16/W19	300 x 2100

4.2 Modeling of the Case Study Building

The analytical model of the case study building is generated by using ETABS 2015. Honestly, this software has some restrictions for a nonlinear model which seems to be in confusion about the acceptance and the reliability of the nonlinear analysis method. Some assumptions that are made during nonlinear modeling of the case study, are illustrated below:

- Three main stages were employed for this case study. Firstly, the building is designed according to TEC-2007 and ISCTB-2008 in the preliminary stage. Secondly, the nonlinear time history analysis is employed for D2 level evaluation under a suite set of seven earthquake shakings with a return period of 43 years which is considered as 10% probability of exceedance. Finally, the results have been evaluated in compliance with the predetermined target performance levels.

- The expected strengths of materials are taken into account in the nonlinear dynamic analysis. In this regard and depending on relationships mentioned in 3.2, the expected strengths are taken as $1.17f_{yk}$ for the yield strength of reinforcement steel and $1.3f_{ck}$ for the compressive strength of concrete.
- Beams and columns are modeled as quasi-elastic elements as presented previously in 3.3.1 with M3 plastic hinges for beams and P-M2-M3 plastic hinges for columns defined at 0.1 and 0.9 of the relative length of all members.
- For calculating the effective flexural stiffness of beams and columns, section moment of inertia (Ig) for each member is reduced by multiplying the gross inertias of beams with 0.4 and the gross inertias of columns with 0.8.
- Reinforced concrete shear wall members are modeled by using inelastic fiber sections for the first five stories and above this level the shear wall members are elastic. Each fiber wall element consists of seven concrete fibers and six steel fibers.
- Rigid diaphragms are assigned to each story level.
- P-Delta effects are considered in this case study in addition to accidental eccentricity.
- Earthquake load combinations in evaluation the seismic performance are selected as 1.0 D + 0.25 L + 1.0 E.

4.3 Seismic Input

The seismic data which are used in the case study for seismic performance evaluation include seven different horizontal component pairs of ground motions. Table 4.3 shows some properties of the selected ground motions. Spectral matching method in time domain is employed in order to generate suitable ground motions time series in accordance with the defined target spectrum by employing ETABS 2015 software program and create the modified response spectrum for each ground motion and each building case by utilizing SeismoSignal v. (2016) software program as presented in Figure 4.4 and 4.5 below.

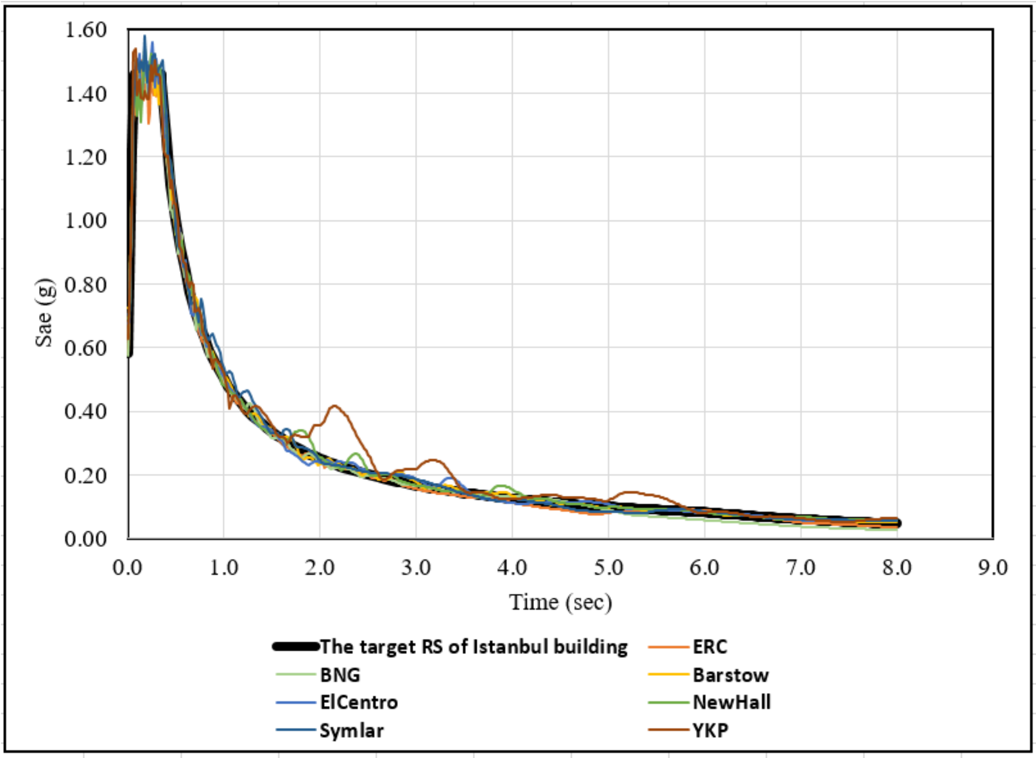


Figure 4.4: Acceleration response spectrum of generated ground motions and the target spectrum for Istanbul building.

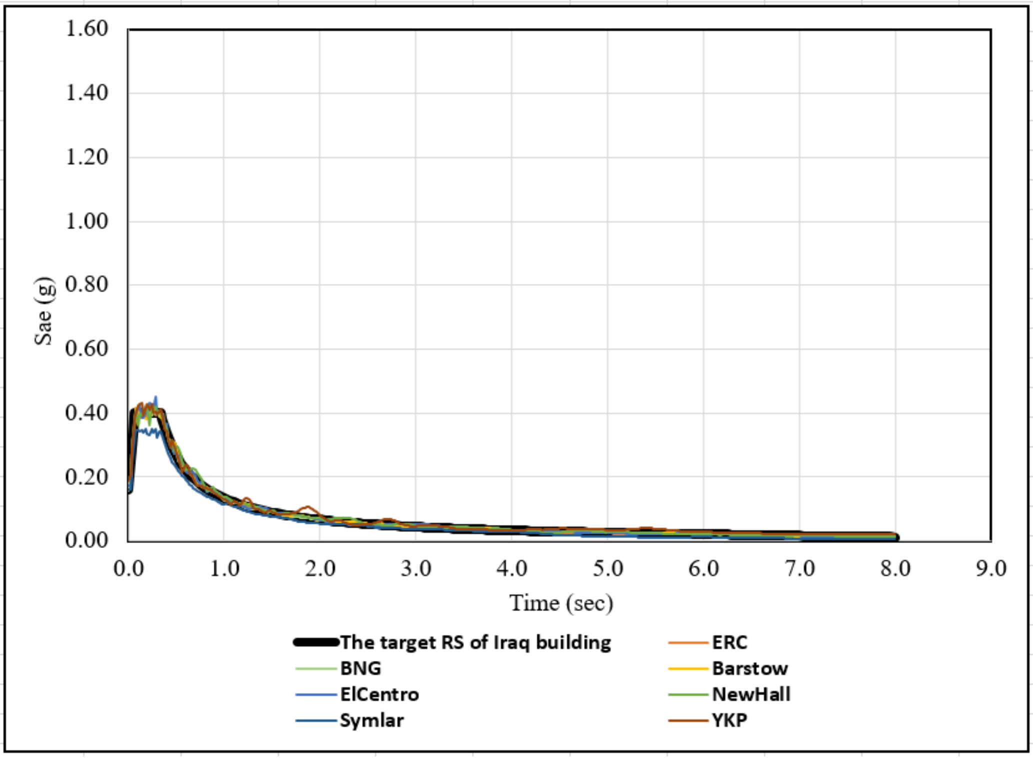


Figure 4.5: Acceleration response spectrum of generated ground motions and the target spectrum for Iraq building.

Table 4.3: The selected ground motion properties.

No.	The earthquake (abbreviation name) – component dir.	Moment Magnitude	PGA (cm/sec^2)	PGV (cm/sec)	PGD (cm)
1	ERZINCAN (ERC) - EW	6.9	488.3	85.9	53.9
	ERZINCAN (ERC) - NS		390.3	102.2	313.9
2	BINGOL (BNG) - EW	6.1	276.8	21.7	56.7
	BINGOL (BNG) - NS		545.5	37.0	56.3
3	ELCENTRO (ElCentro) - EW	6.9	452.0	62.4	27.7
	ELCENTRO (ElCentro) - NS		662.8	59.8	14.2
4	BARSTOW (Barstow) - EW	7.3	412.9	66.0	33.1
	BARSTOW (Barstow) - NS		417.4	65.6	39.5
5	NEWHALL (NewHall) - EW	6.7	664.9	95.5	19.8
	NEWHALL (NewHall) - NS		644.4	80.9	35.5
6	SYMLAR (Symlar) - EW	6.7	558.4	80.1	17.3
	SYMLAR (Symlar) - NS		801.4	118.9	26.9
7	YARIMCA (YKP) - EW	7.4	35.5	7.4	23.0
	YARIMCA (YKP) - NS		41.0	8.5	48.0

4.4 Presentation of Analysis Results for the Case Study

In this section, the analysis results of each building case are explained including the free vibration of the system, the base shear value, the storey shear values, the maximum effective relative story drifts, hinge states and the internal forces of some critical sections of shear walls and columns.

4.4.1 Free vibration (eigenvalue) analysis

Free vibration properties of the building are obtained depending on the un-cracked section properties. The natural periods of vibration of the system as well as the modal participation mass ratios for the first fifteen mode shapes are presented in Table 4.4.

Table 4.4: The natural periods of vibration of the system and the modal participation mass ratios.

Mode	T_n (sec)	Modal participation mass ratio in X-dir. (%)	Modal participation mass ratio in Y-dir. (%)
1	3.085	21.04	0.00
2	2.792	50.18	0.00
3	2.423	0.00	69.92
4	0.955	2.09	0.00
5	0.796	10.54	0.00
6	0.686	0.00	14.53
7	0.517	0.55	0.00
8	0.391	3.83	0.00
9	0.342	0.26	0.36
10	0.341	0.02	4.15
11	0.248	0.12	0.00
12	0.24	2.25	0.00
13	0.221	0.00	2.41
14	0.192	0.09	0.00
15	0.164	1.44	0.00
Sum (required)=		92.41 (>90%)	91.37 (>90%)

4.4.2 Base shear and storey shear forces

The mean values of the base shear forces for each building case caused by the selected seven pairs of earthquake shakings are presented in Table 4.5. Figure 4.6 and 4.7 illustrates the mean value of story shear forces affect the buildings in each direction.

Table 4.5: The base shear force.

Earthquake	Istanbul Building		Iraq Building	
	Abs. Max value in X-dir. (tonf)	Abs. Max value in Y-dir. (tonf)	Abs. Max value in X-dir. (tonf)	Abs. Max value in Y-dir. (tonf)
ERC	5228.32	4653.27	1452.27	1340.72
BNG	5421.37	4323.13	1590.21	1111.62
ElCentro	5808.68	3628.11	1887.15	1106.49
Barstow	5189.20	4130.66	1845.45	2030.35
Symlar	7696.74	5102.94	1456.09	971.85
NewHall	5735.98	4884.09	1623.02	858.82
YKP	5560.26	4299.77	1575.06	1278.45
Average values	5805.79	4431.71	1632.75	1242.61

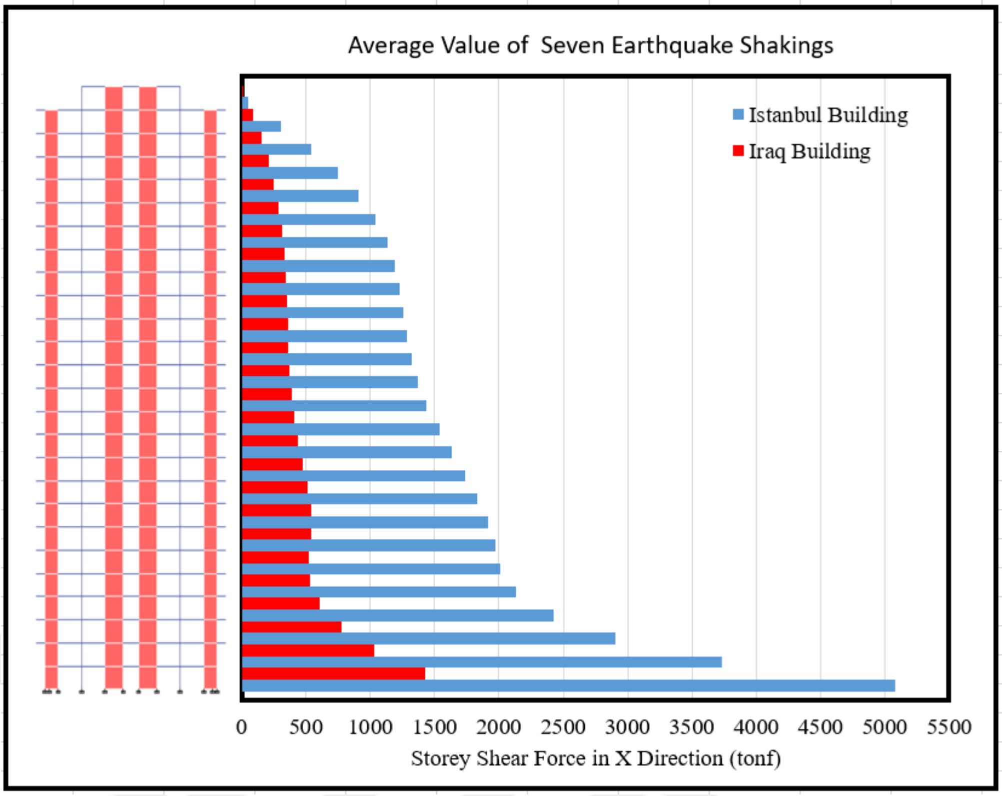


Figure 4.6: Storey shear forces in X-dir. for both cases.

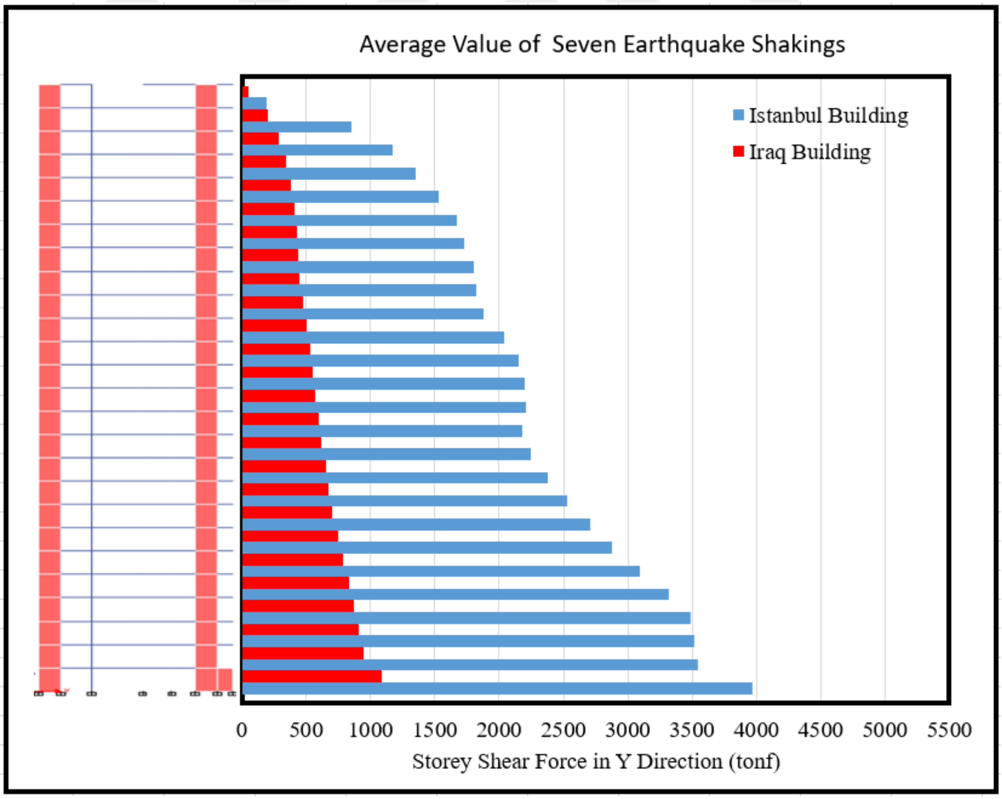


Figure 4.7: Storey shear forces in Y-dir. for both cases.

4.4.3 Displacement and maximum effective relative storey drift demands

The variation of the top story displacement of each building case due to effect of Erzincan earthquake is illustrated in Figure 4.8 and Figure 4.9. Figure 4.10 explains the maximum effective relative story drifts throughout the height of the building under effects of seven pairs of earthquake in each direction and a comparison of the mean values for each building case is shown in Figure 4.11.

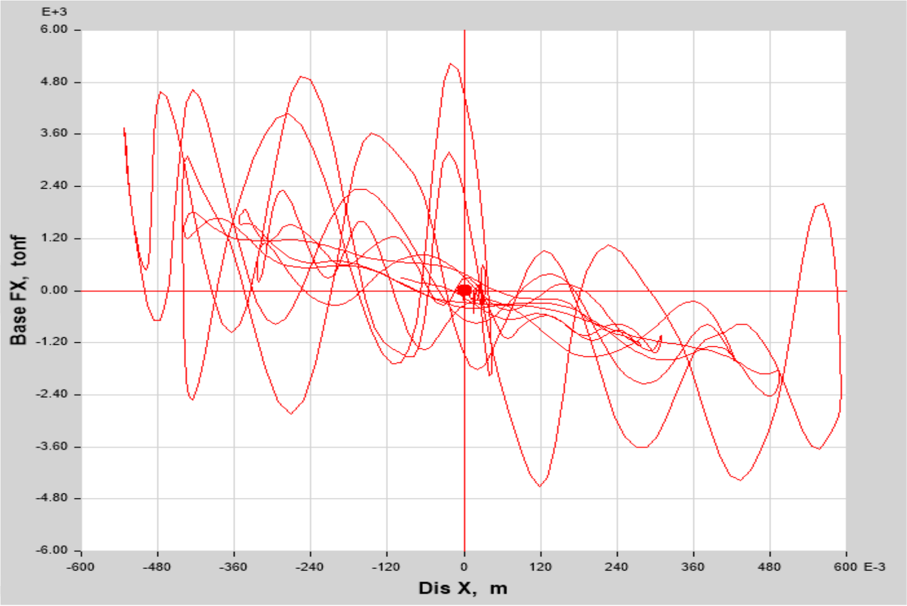
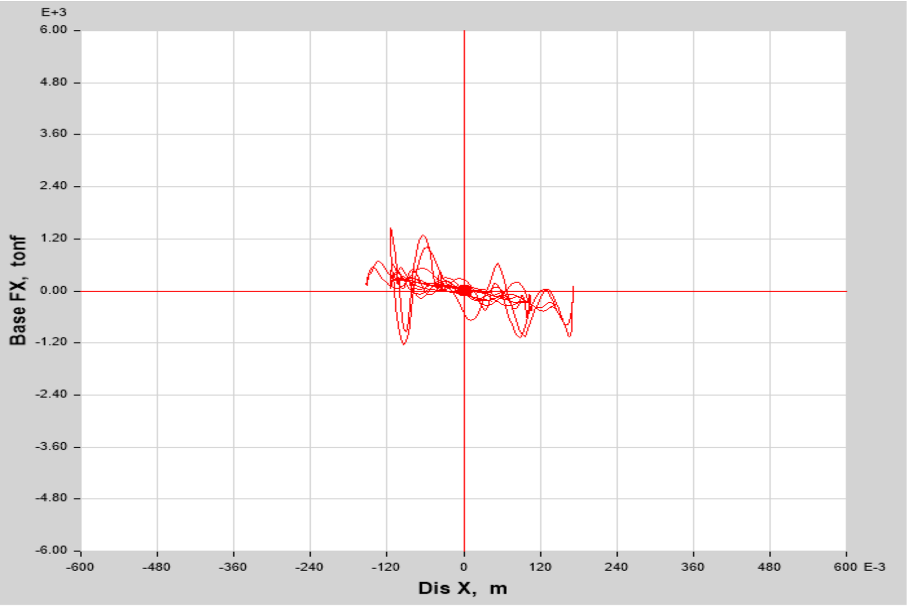
Istanbul Building	
Iraq Building	

Figure 4.8: Base shear force vs top storey displacement in X-dir.

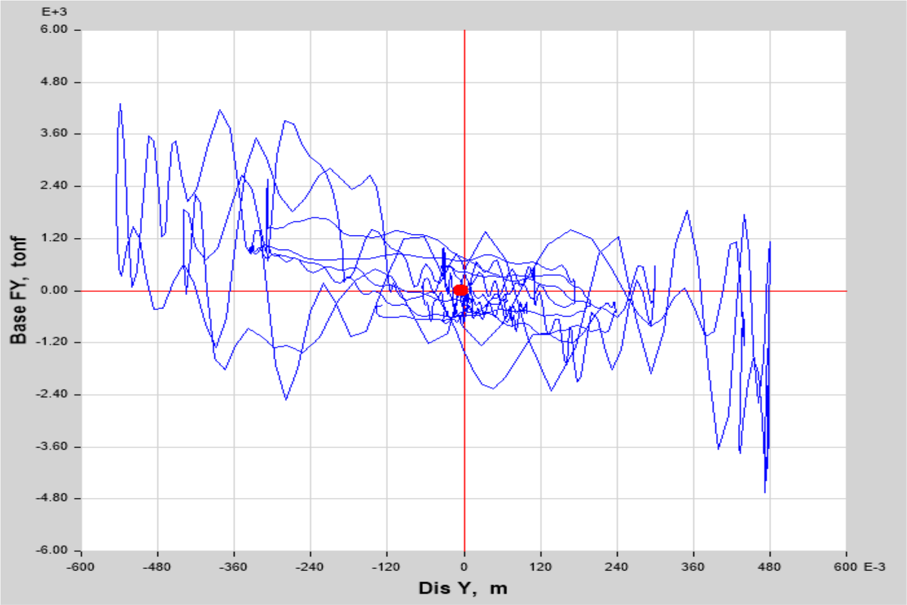
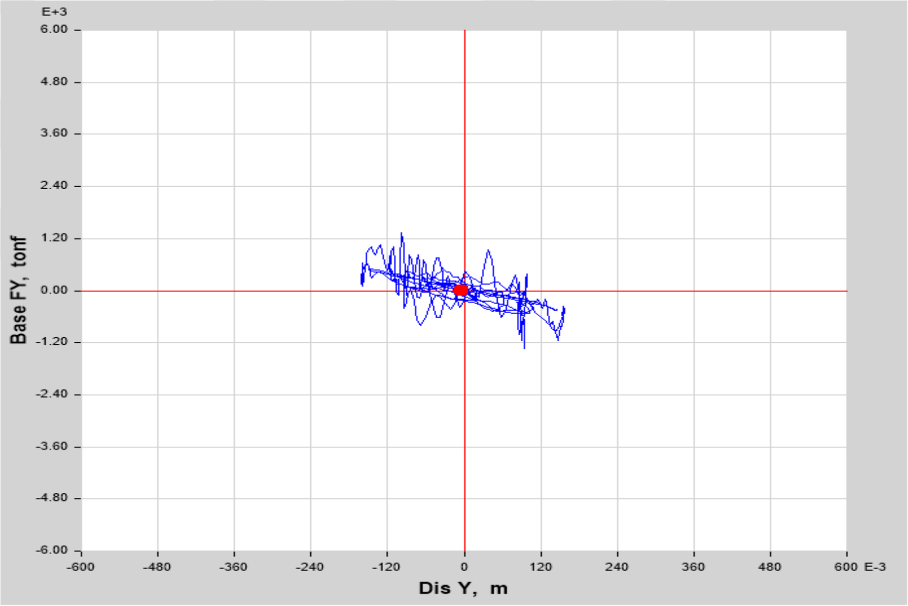
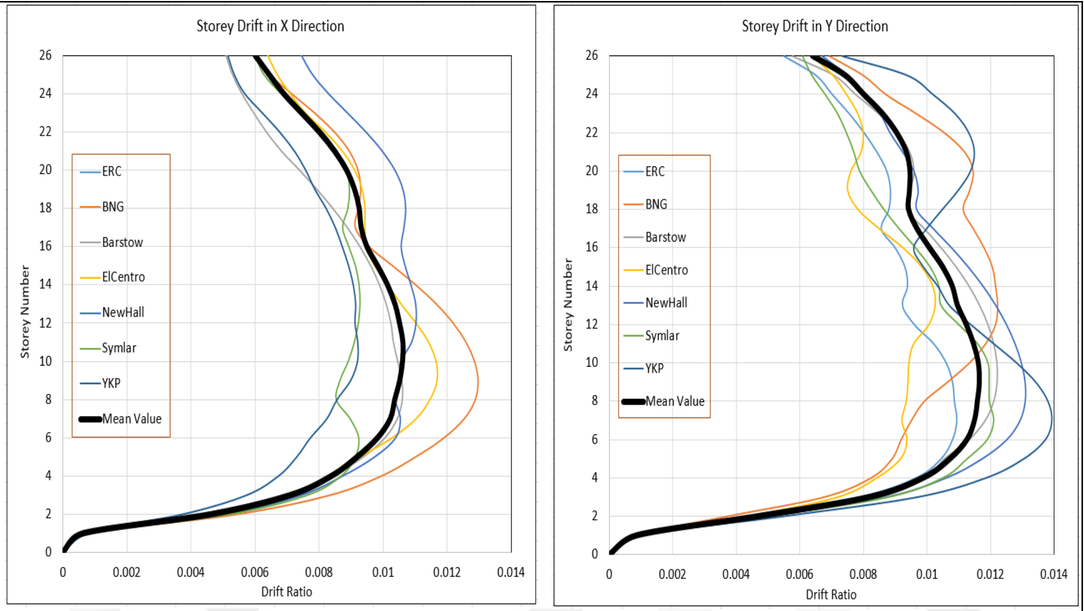
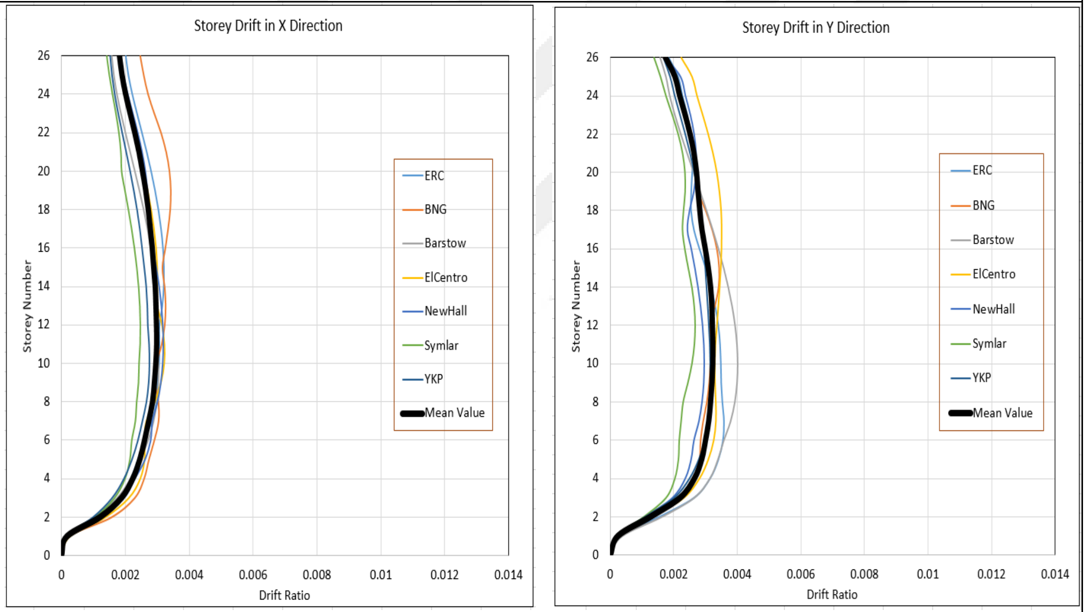
Istanbul Building	
Iraq Building	

Figure 4.9: Base shear force vs top storey displacement in Y-dir.

The hysteresis behavior of the building under the effect of each ground motion is presented in Appendix C.



Istanbul Building



Iraq Building

Figure 4.10: The maximum effective relative storey drifts.

In this design stage, the maximum effective relative story drift should not exceed 0.025 for performance objective of life safety and that’s achieved in both building cases.

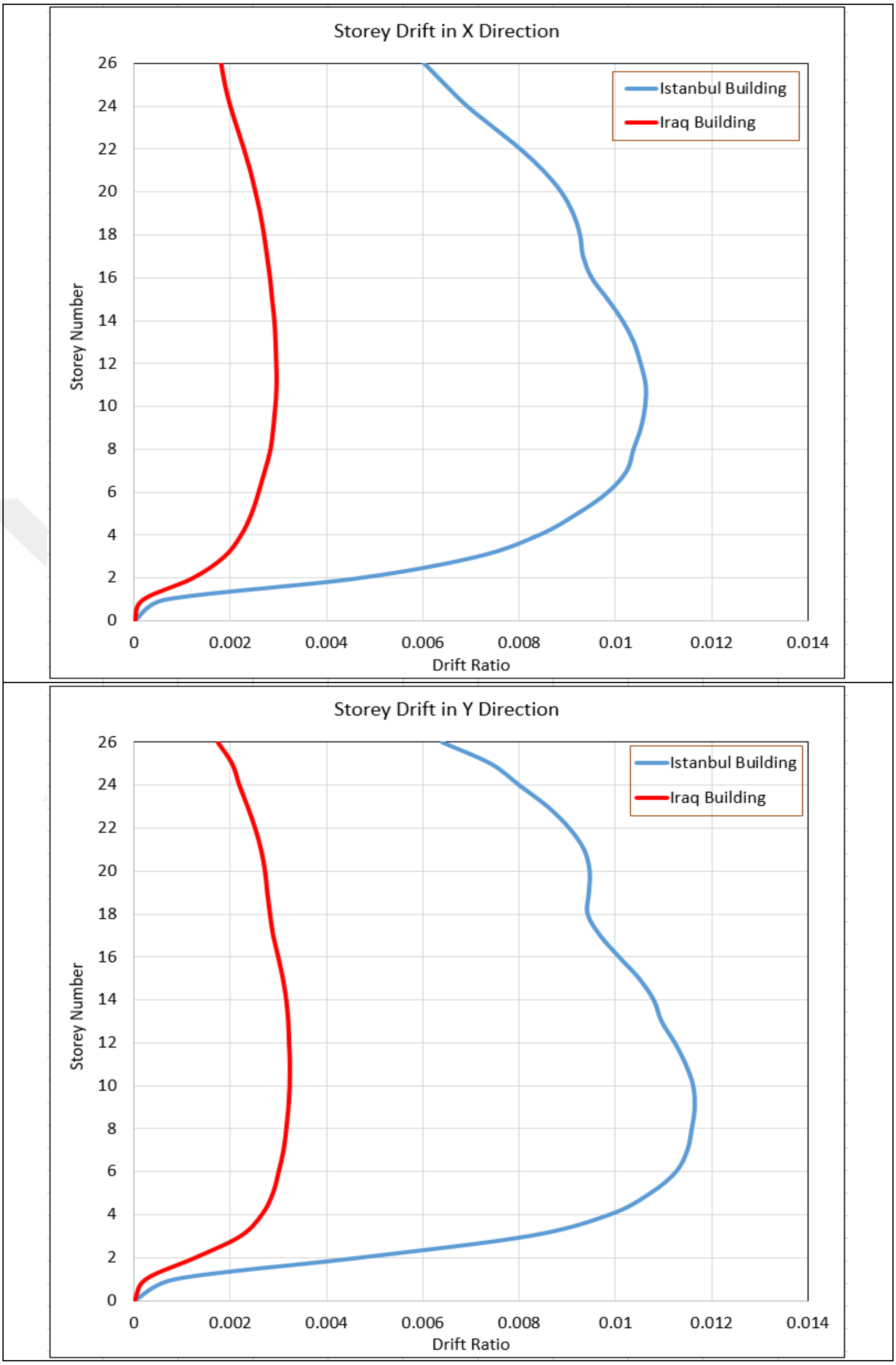


Figure 4.11: Mean values of the effective relative storey drifts.

4.4.4 Hinge states

As mentioned previously in modeling the case study, all columns and beams are assigned as quasi-elastic elements with concentrated plastic hinges at 0.1 and 0.9 of the relative distance of the member and fiber hinges are assigned to the continuous shear walls of the first five storeys. The analysis results showed that all columns still elastic under the effects of the earthquakes, on the other hand, a lot of beams passed slightly the yield point, B, without exceeding the acceptance criteria of immediate occupancy performance level which is defined, in addition to the other performance levels, on plot of the backbone curve as shown in Figure 4.12. Figure 4.13 illustrates the elements that exceeded the yield point for both building cases under Erzincan earthquake shakings as a comparison. The fiber hinges of the shear walls exhibited a linear elastic behavior with a very slight rotation as shown in Figure 4.14 and Figure 4.15 for both buildings under the effect of Erzincan earthquake.

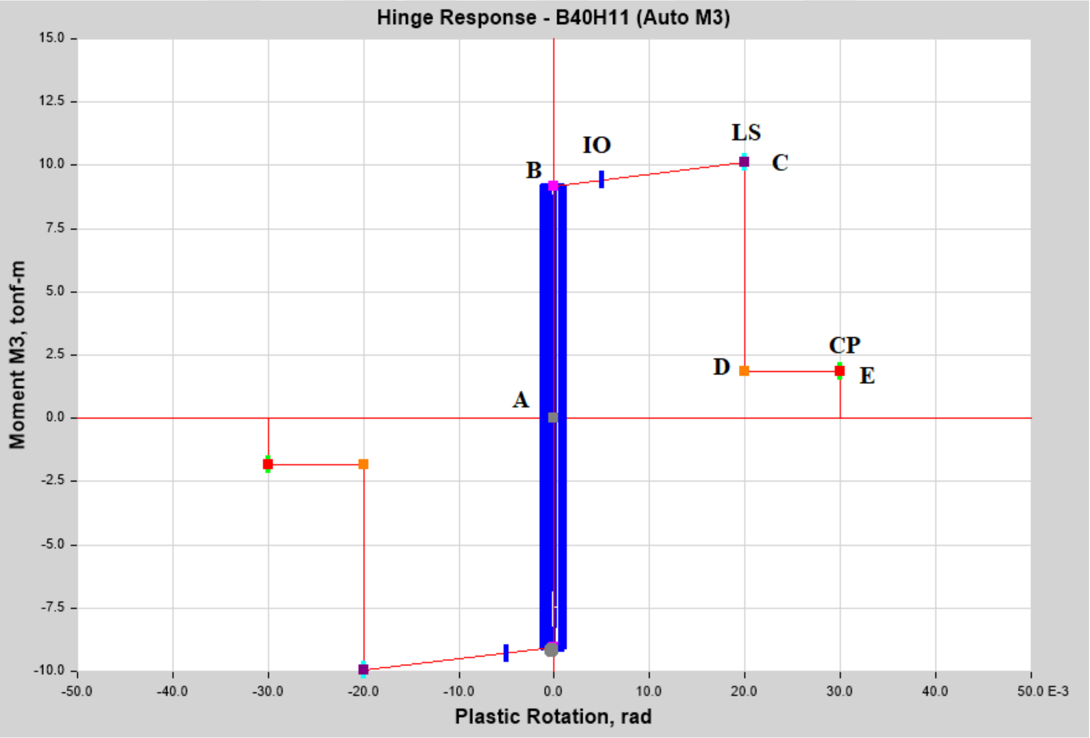
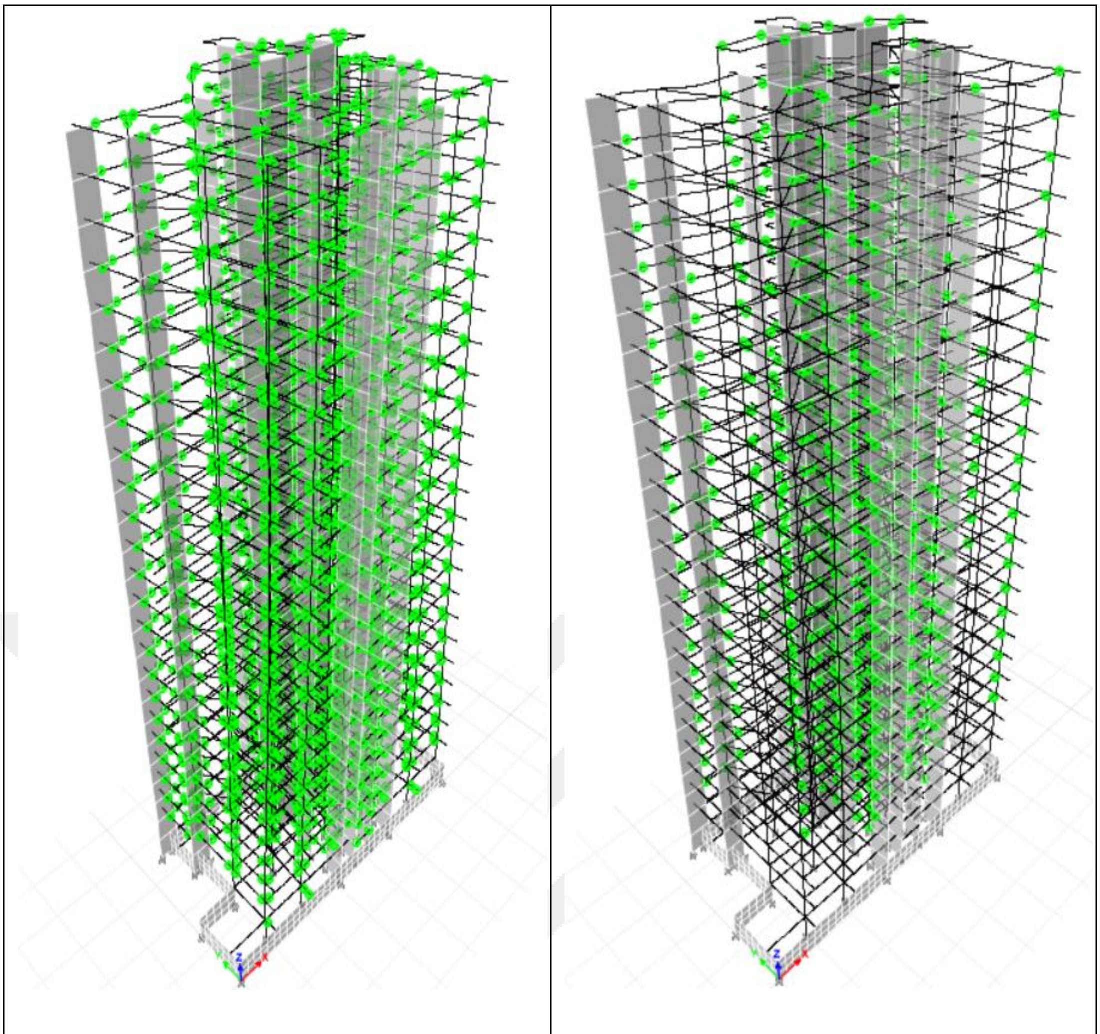


Figure 4.12: The general backbone curve with points represent the acceptance criteria of performance levels and the hinge response for one of couple beams at third storey level in Istanbul building under the effect of Erzincan earthquake.



Istanbul Building

Iraq Building

Figure 4.13: Beam elements exceeded the yield point, B, under the effect of Erzincan earthquake.

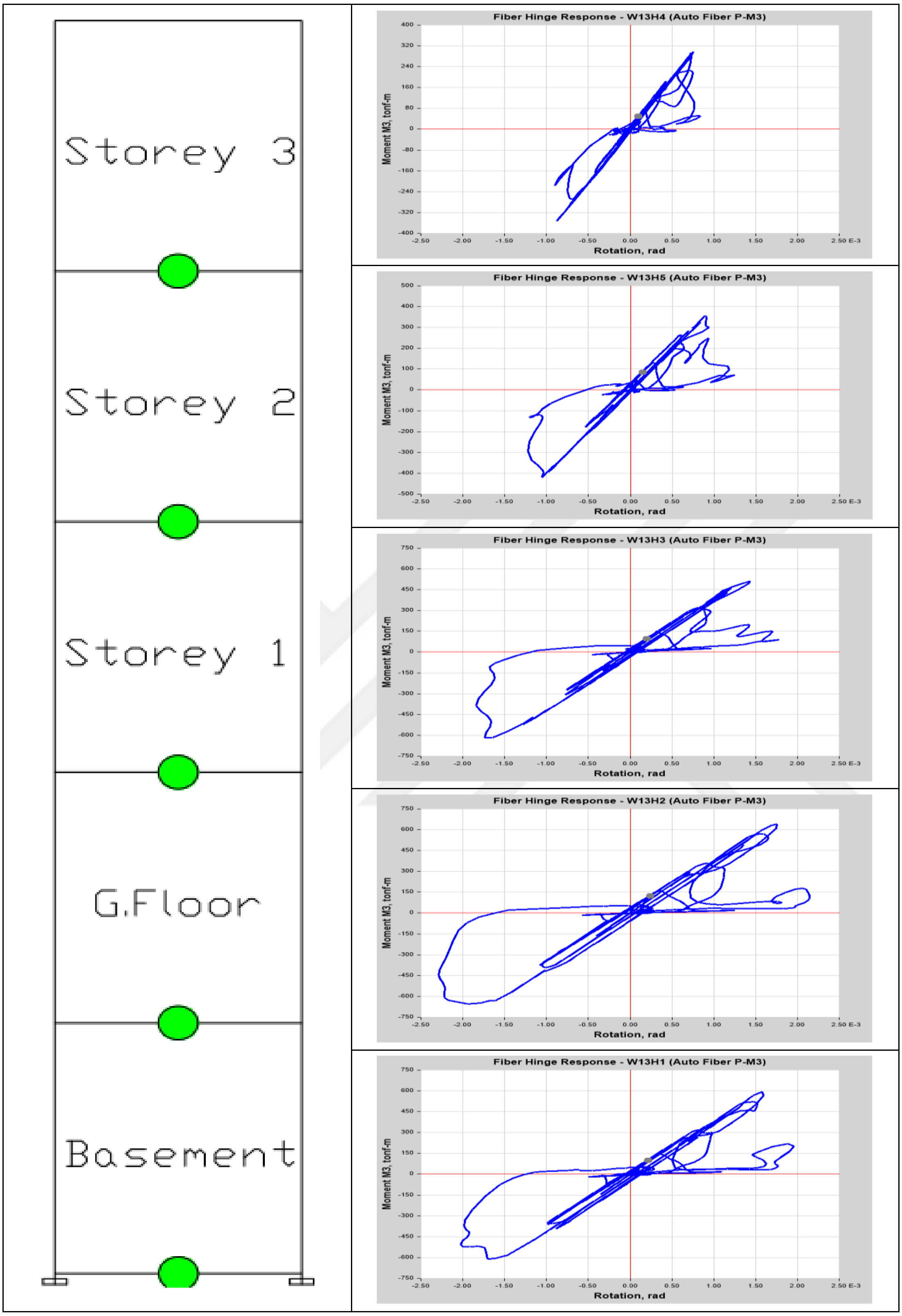


Figure 4.14: Fiber hinge response of W09 under effect of Erzincan earthquake for Istanbul building.

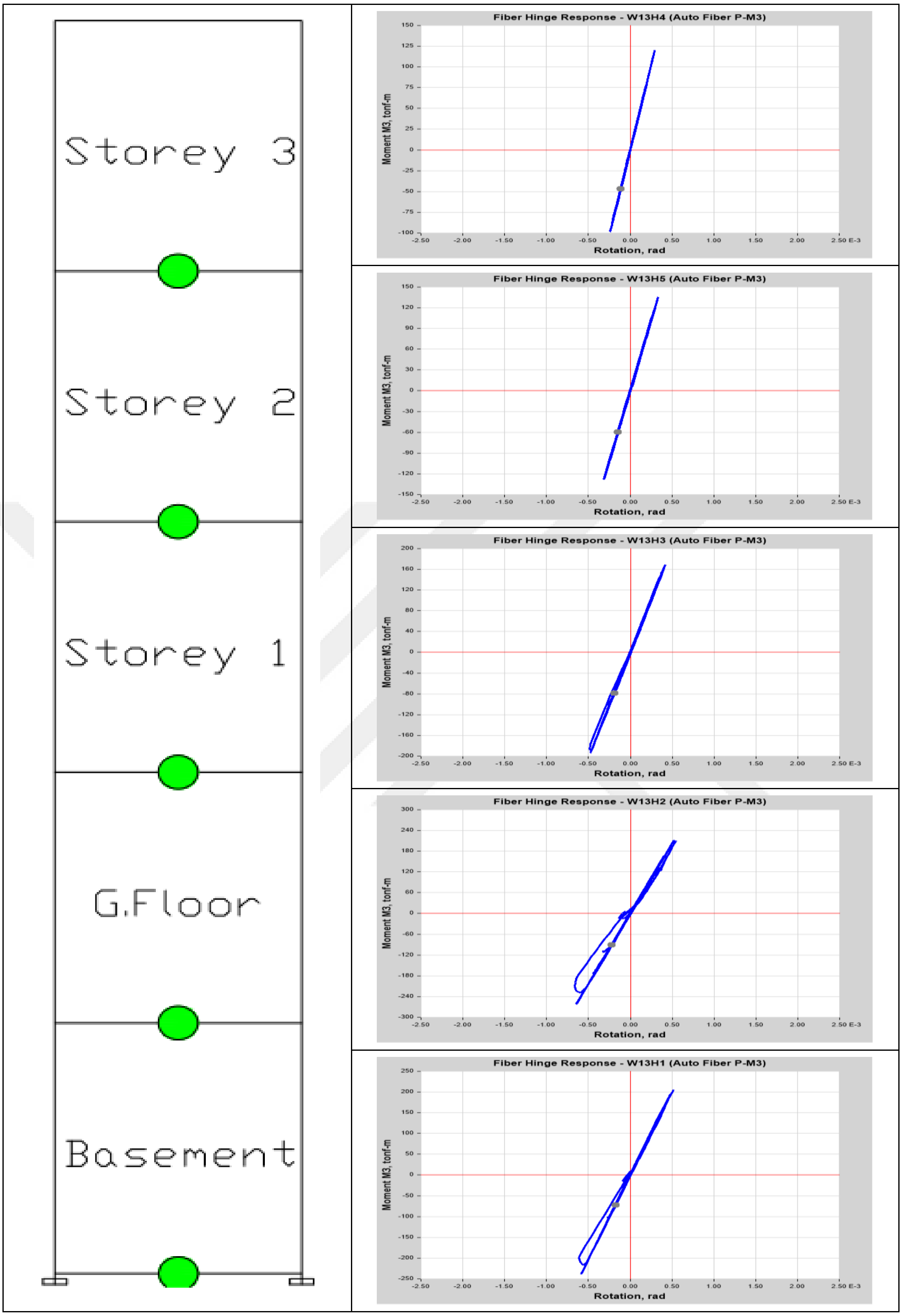


Figure 4.15: Fiber hinge response of W09 under the effect of Erzincan earthquake for Iraq building.

4.4.5 Shear stress of shear walls and axial load of columns

In this paragraph, average shear stress results of shear walls and maximum axial load of columns are explained. Due to the symmetry of the plan, results for two critical sections will be discussed for each member; W07 and W09 for shear walls and C06 and C09 for columns under the effect of load combination including Erzincan earthquake shakings. Generally shear failure of structural members are not desired, for that reason, designer should meet some provisions and limitations provided by seismic codes and guidelines in addition to those of axial loads. In this study, all members meet the requirements and limitations which are defined in TEC-2007 for shear and axial forces. Figure 4.16 explained the average shear stress of the shear walls and Figure 4.17 showed the maximum axial loads of the columns.

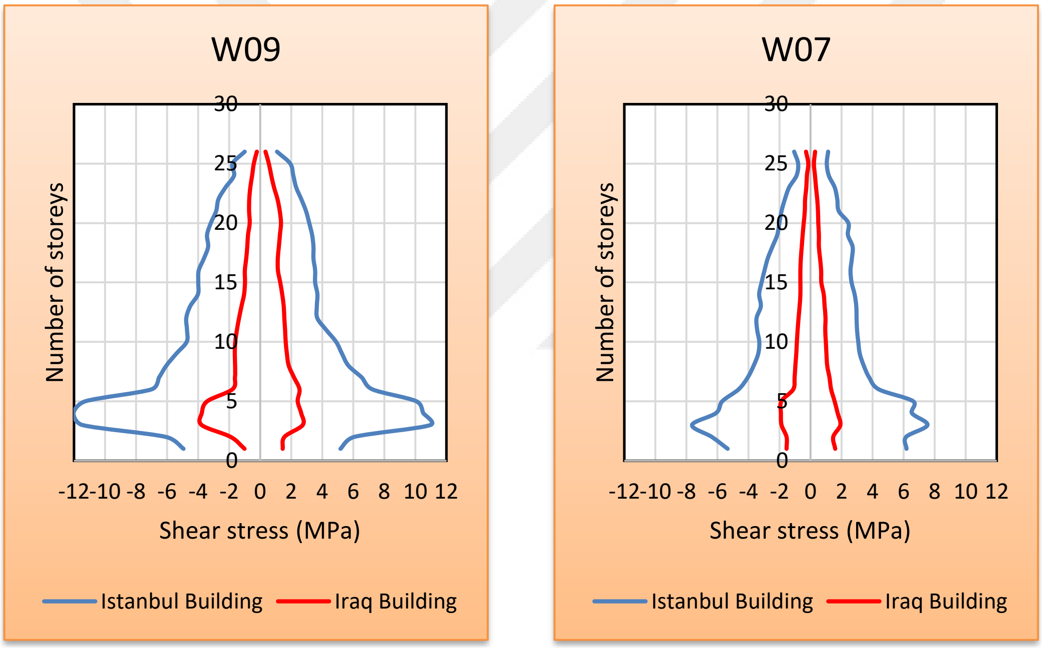


Figure 4.16: Average shear stresses of W09 and W07 under effect of Erzincan earthquake.

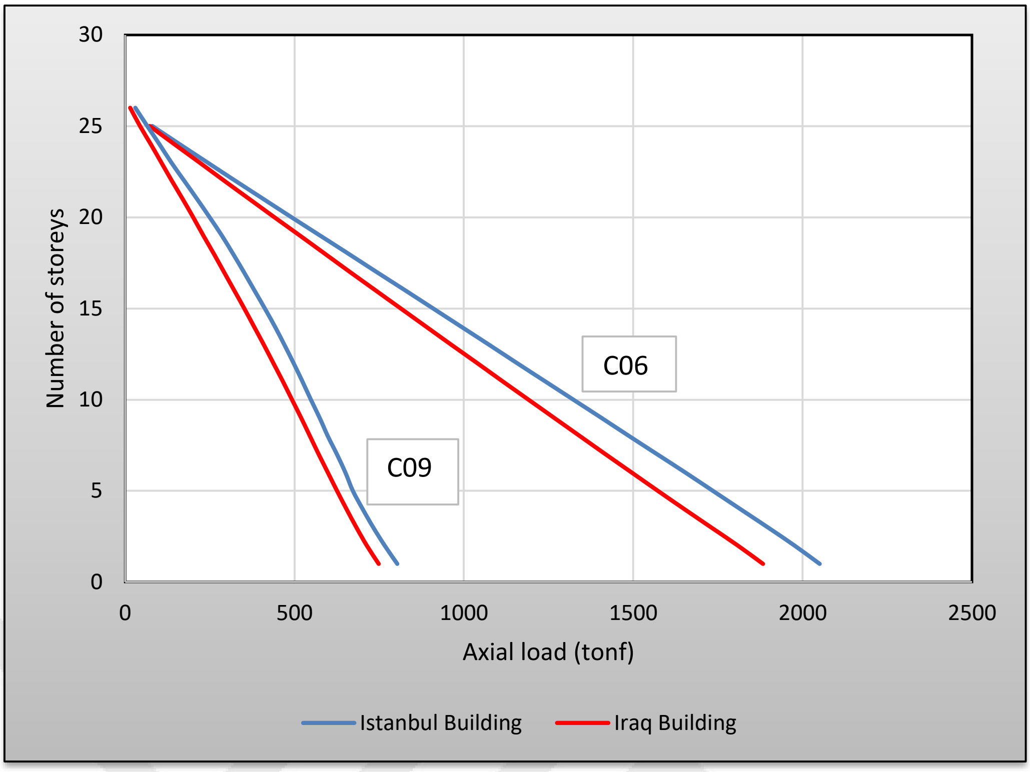


Figure 4.17: Maximum axial loads of C09 and C06 under effect of Erzincan earthquake.

4.5 Modified Model for the Case Study in Iraq

The previous results indicated that the case study building has a well-designed structural system which gives a very satisfied results for a region with high seismicity like Istanbul. However, construction of same system in Iraq is considered, from a structural engineering viewpoint, exaggerated and uneconomic. For this purpose, some proposed changes are carried out on the plan of the building as following:

- W01, W02, W03, W06, W10 and W17 are turned into column sections with dimensions of 900 x 500 mm and for the first four sections and 400 x 1000 mm for the last two sections.
- Bar size of the longitudinal reinforcement of the sections is altered from 18mm to 16mm for all columns.
- The spacing of the transverse reinforcement became 20 cm for all columns.

After these modifications have been carried out, the case study is re-analysed and nonlinear dynamic analysis has been performed to evaluate the new seismic behavior of the building through the previously selected results which are given sometimes as a comparison with the previous original design in the following parts.

4.5.1 Free vibration (eigenvalue) analysis

The natural periods of vibration of the elastic system as well as the modal participation mass ratios for the first fifteen mode shapes are presented in Table 4.6.

Table 4.6: The natural periods of vibration of the system and the modal participation mass ratios.

Mode	T_n (sec)	Modal participation mass ratio in X-dir. (%)	Modal participation mass ratio in Y-dir. (%)
1	3.155	14.97	0.00
2	2.877	55.75	0.00
3	2.534	0.00	69.47
4	0.968	1.87	0.00
5	0.827	10.84	0.00
6	0.714	0.00	14.28
7	0.523	0.46	0.00
8	0.409	3.88	0.00
9	0.352	0.00	4.52
10	0.343	0.25	0.00
11	0.252	2.03	0.00
12	0.246	0.32	0.00
13	0.223	0.00	2.46
14	0.187	0.05	0.00
15	0.171	1.45	0.00
Sum (required)=		91.87 (>90%)	90.73 (>90%)

4.5.2 Base shear forces

The mean values of the base shear forces for both designs of the building in Iraq caused by the selected seven pairs of earthquake shakings are presented in Table 4.7.

Table 4.7: The base shear force.

Earthquake	Iraq Building (original design)		Iraq Building (modified design)	
	Abs. Max value in X-dir. (tonf)	Abs. Max value in Y-dir. (tonf)	Abs. Max value in X-dir. (tonf)	Abs. Max value in Y-dir. (tonf)
ERC	1452.27	1340.72	1204.714	1740.734
BNG	1590.21	1111.62	1394.025	1832.936
ElCentro	1887.15	1106.49	1118.137	1924.059
Barstow	1845.45	2030.35	1112.917	1628.459
Symlar	1456.09	971.85	984.752	1298.973
NewHall	1623.02	858.82	1588.774	1052.526
YKP	1575.06	1278.45	1032.745	1606.187
Average values	1632.75	1242.61	1205.152	1583.411

4.5.3 Displacement and maximum effective relative story drift demands

The variation of the top story displacement of each design case due to the effect of Erzincan earthquake is illustrated in Figure 4.18 and Figure 4.19. Figure 4.20 explains the maximum effective relative story drifts throughout the height of the building under effects of seven pairs of the earthquake in each direction.

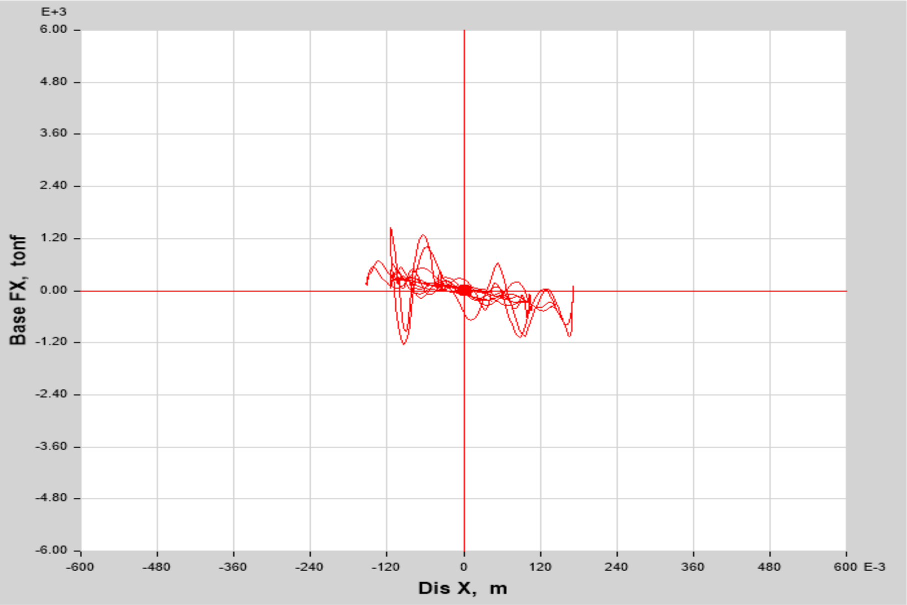
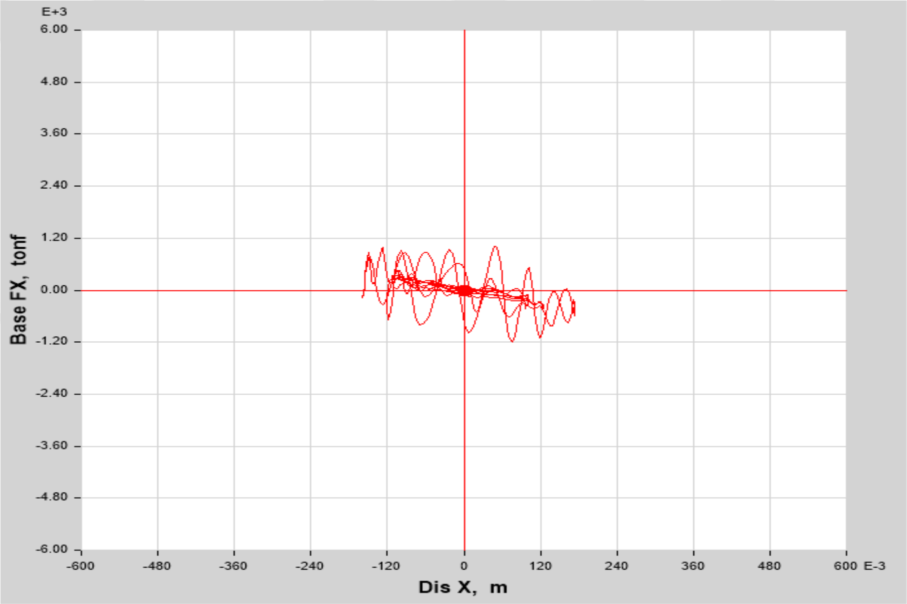
Iraq Building (original design)	
Iraq Building (modified design)	

Figure 4.18: Base shear force vs top storey displacement in X-dir.

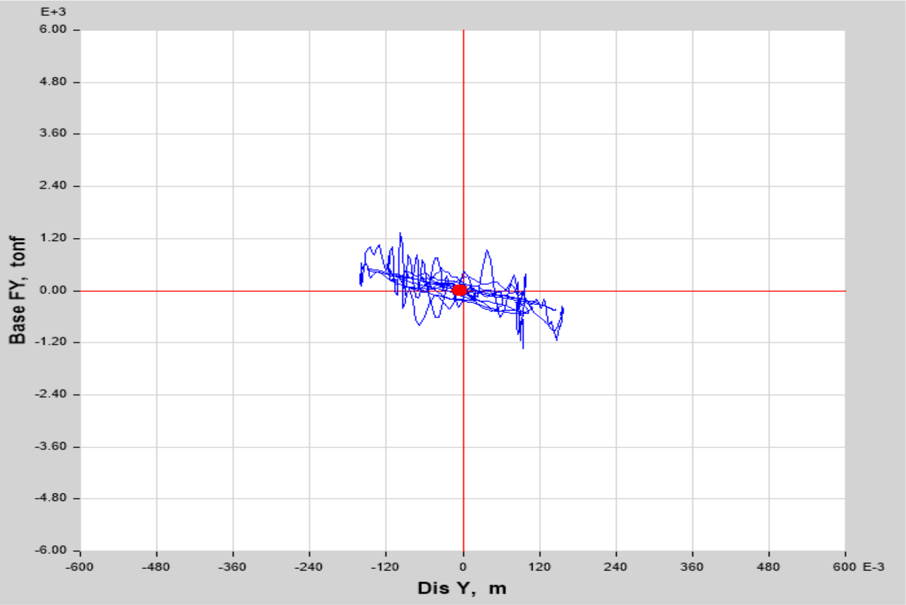
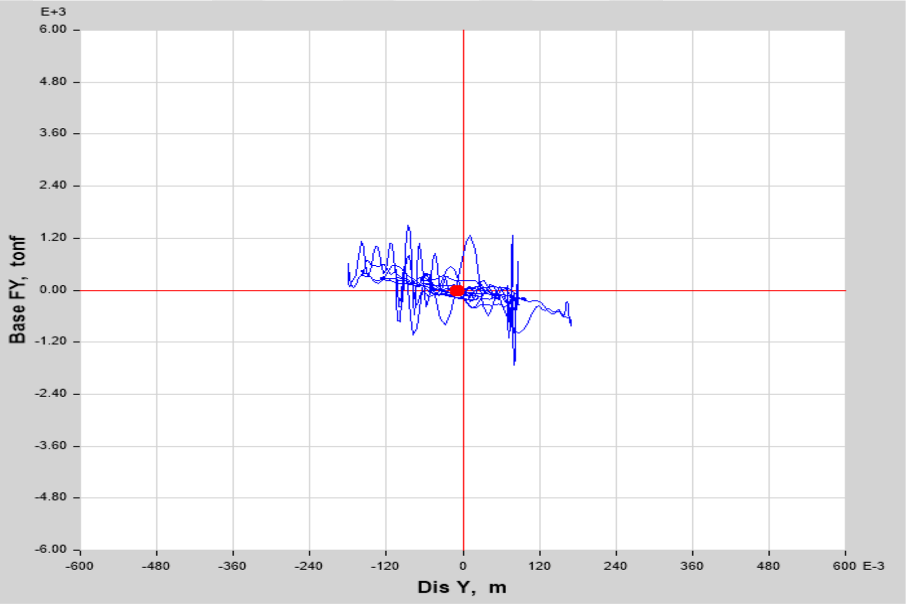
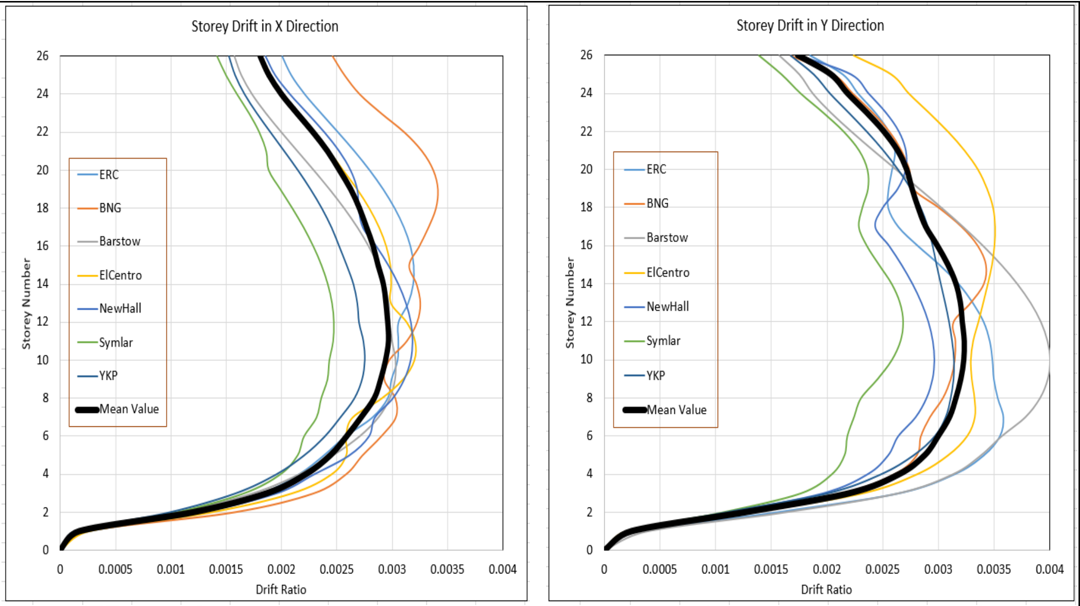
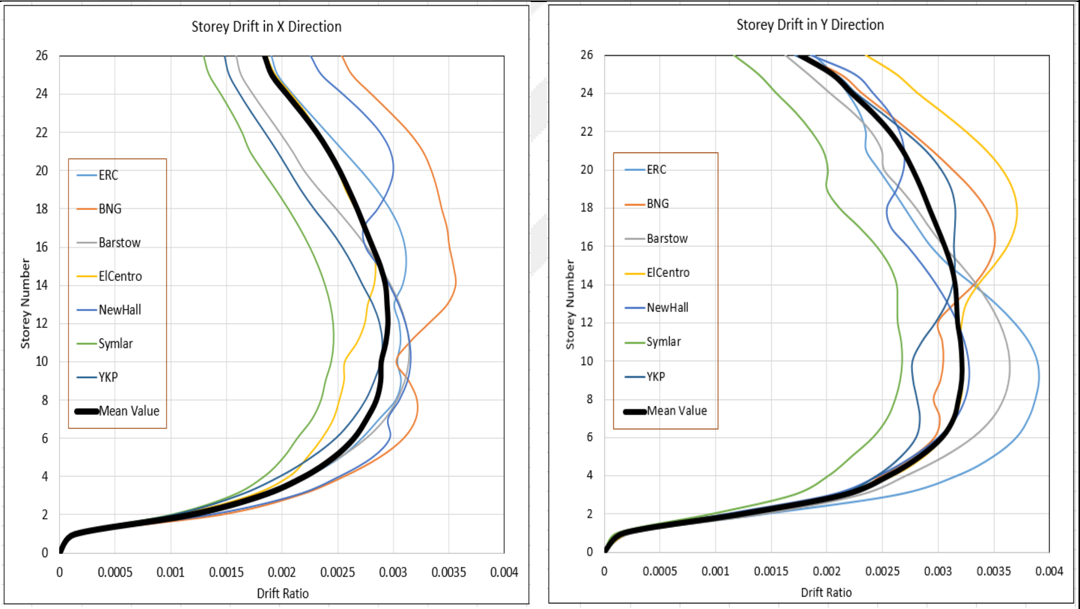
Iraq Building (original design)	
Iraq Building (modified design)	

Figure 4.19: Base shear force vs top storey displacement in Y-dir.



Iraq Building (original design)



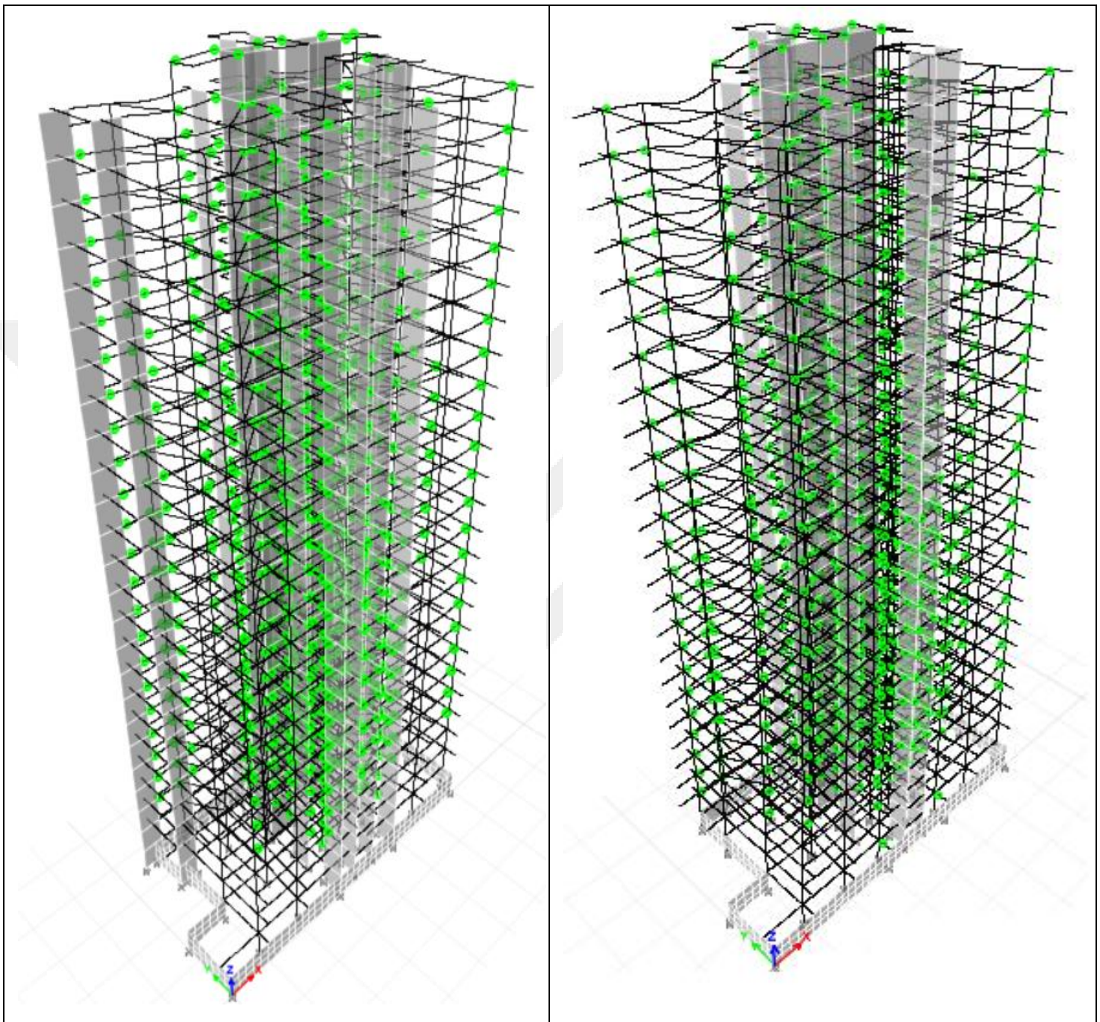
Iraq Building (modified design)

Figure 4.20: The maximum effective relative storey drifts.

In this design stage, the maximum effective relative story drift should not exceed 0.025 for performance objective of life safety and that's achieved in both design cases which are very close to each other.

4.5.4 Hinge states

The Same approach has been followed in assigning thee plastic hinges for elements. The analysis results showed that all columns still elastic under the effects of the earthquakes, on the other hand, a lot of beams passed slightly the yield point, B, without exceeding the acceptance criteria of immediate occupancy performance level. Figure 4.21 illustrates the elements that exceeded the yield point for both design cases and the number of these elements slightly increased in the new design case. The fiber hinges of the shear walls exhibited a linear elastic behavior with a very slight rotation as shown in Figure 4.22.



Iraq Building (original design)

Iraq Building (modified design)

Figure 4.21: Beam elements exceeded the yield point, B, under effect of Erzincan earthquake.

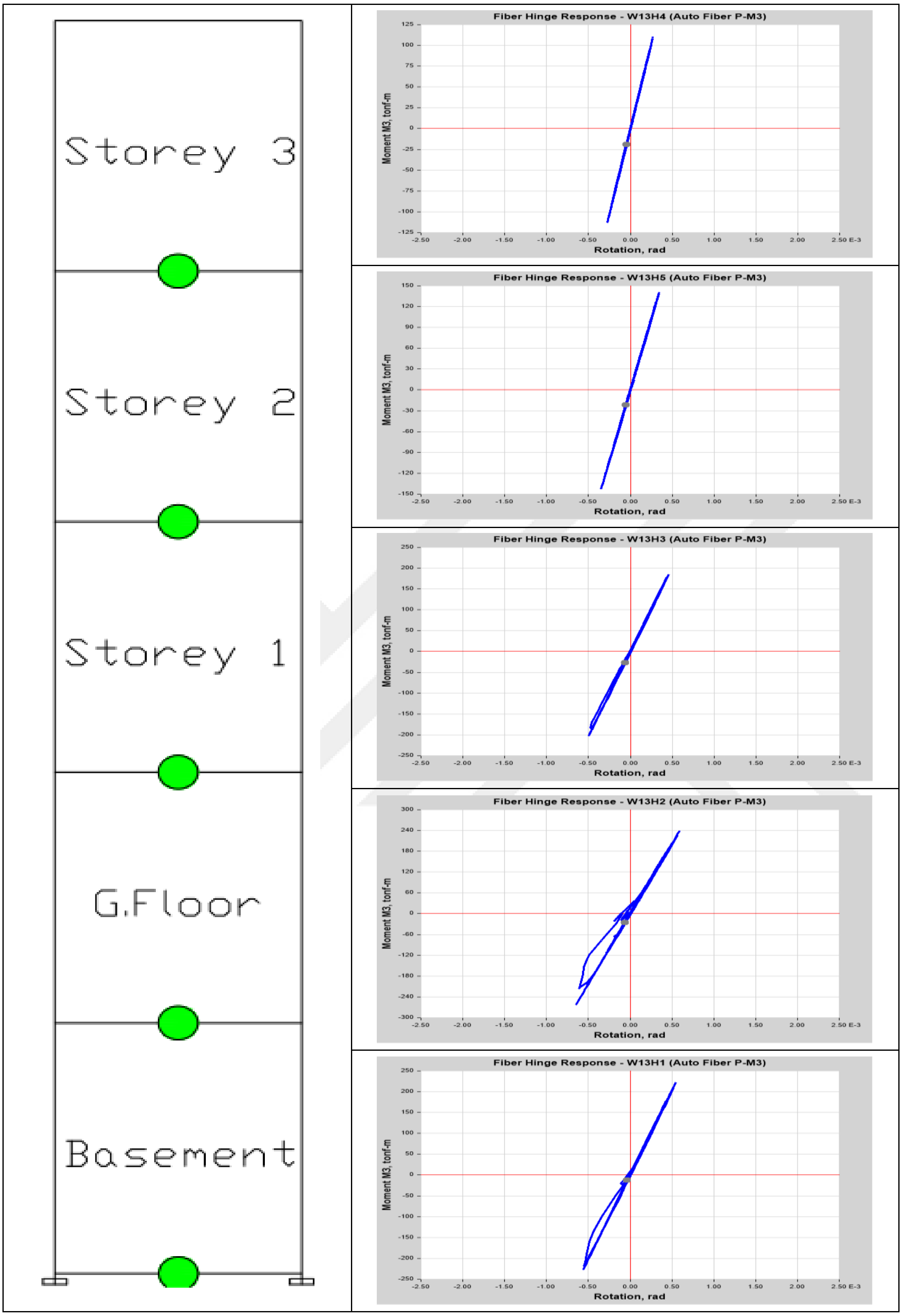


Figure 4.22: Fiber hinge response of W09 under effect of Erzincan earthquake for Istanbul building.

4.5.5 Shear stress of shear walls and axial load of columns

The results for the selected critical sections are presented in Figure 4.23 and Figure 4.24 as a comparison with the previous results of the case study under effect of load combination including Erzincan earthquake shakings.

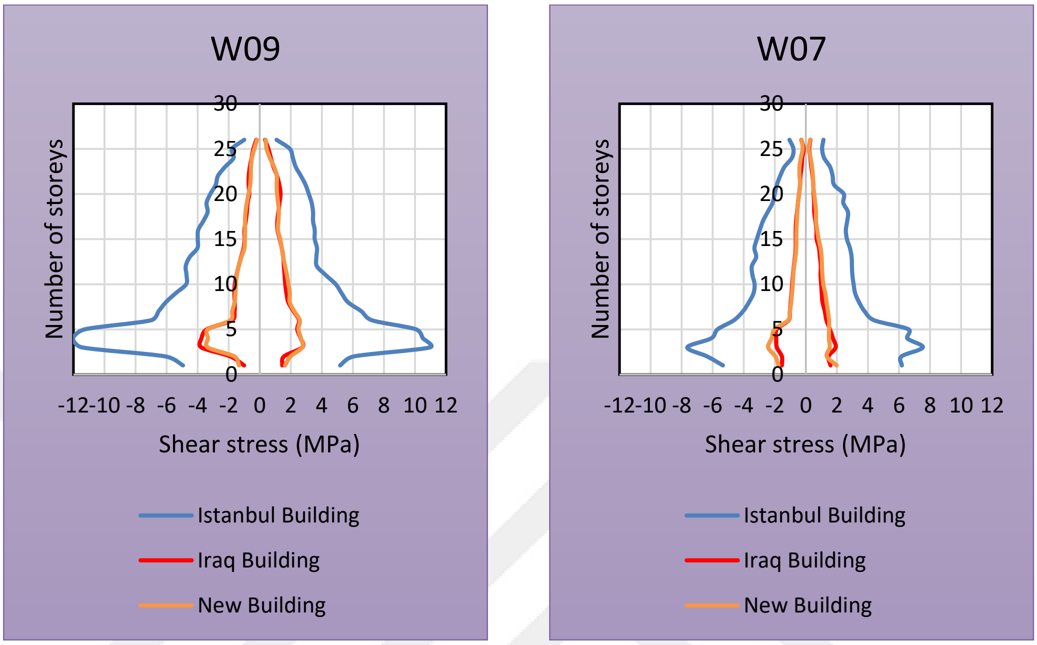


Figure 4.23: Average shear stresses of W09 and W07 under effect of Erzincan earthquake.

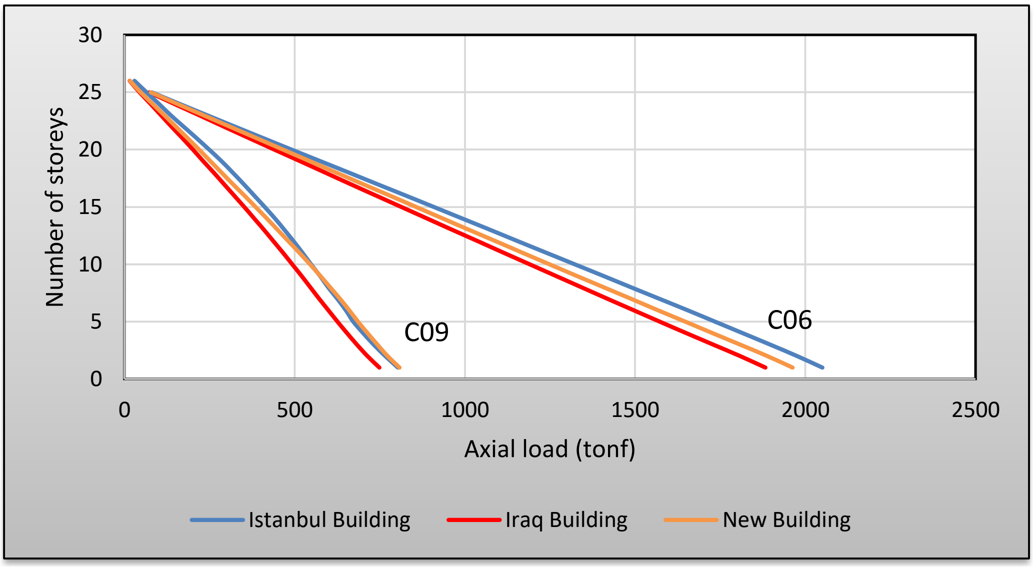


Figure 4.24: Maximum axial loads of C09 and C06 under effect of Erzincan earthquake.

The results of the new design of the building to be constructed in Iraq demonstrated that the performed changes generally don't have a significant influence on the seismic behaviour of the building and that related to some aspects, one of them is obviously the locations of the changed sections which have a slight effect on the overall stiffness of the building but the designer should not disregard the economic side effects which should be taken into account as one of the fundamental principles of the engineering design.

4.6 Verification Stage

The last design stage is a verification stage for evaluating the seismic performance of the building. According to ISCTB-2008, this case study is considered a normal building with height does not exceed 75m thus allowing the verification to be performed by employing only the linear response spectrum for D1 earthquake level with value of structural system behaviour coefficient, R, equal to 1.5. In this study, the verification is performed only for Istanbul building, which is considered as a special class of building, by two more challenging ways as following:

1- By employing the linear response spectrum for D2 earthquake level with R=1.5.

2- By performing the nonlinear time history analysis for D3 level of Erzincan earthquake.

Storey drift ratio is introduced as an acceptance criterion to evaluate the required performance objective of the building in each step above, where the immediate occupancy is required in first step with limit of drift ratio ≤ 1%, however, the collapse prevention is required in second step with limit of drift ratio ≤ 3.5%. Figure 4.25 presented the two cases with their limits and it is obvious that the result meet the acceptance criteria of the required performance level with a slight passing for the immediate occupancy performance level. Figure 4.26 showed the target response spectrum for D3 level of earthquake with the matched spectrum of Erzincan earthquake.

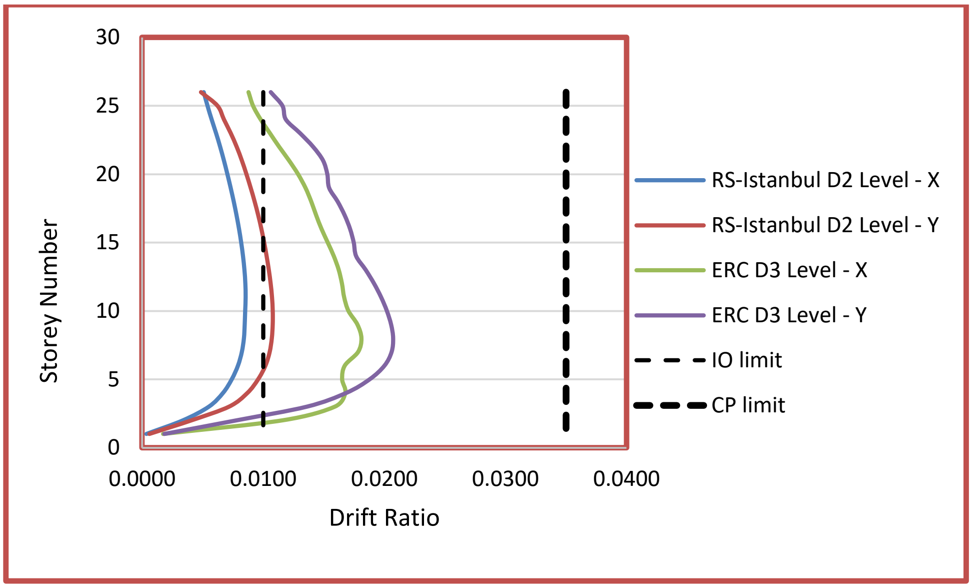


Figure 4.25: Maximum relative storey drift ratio for verification stage.

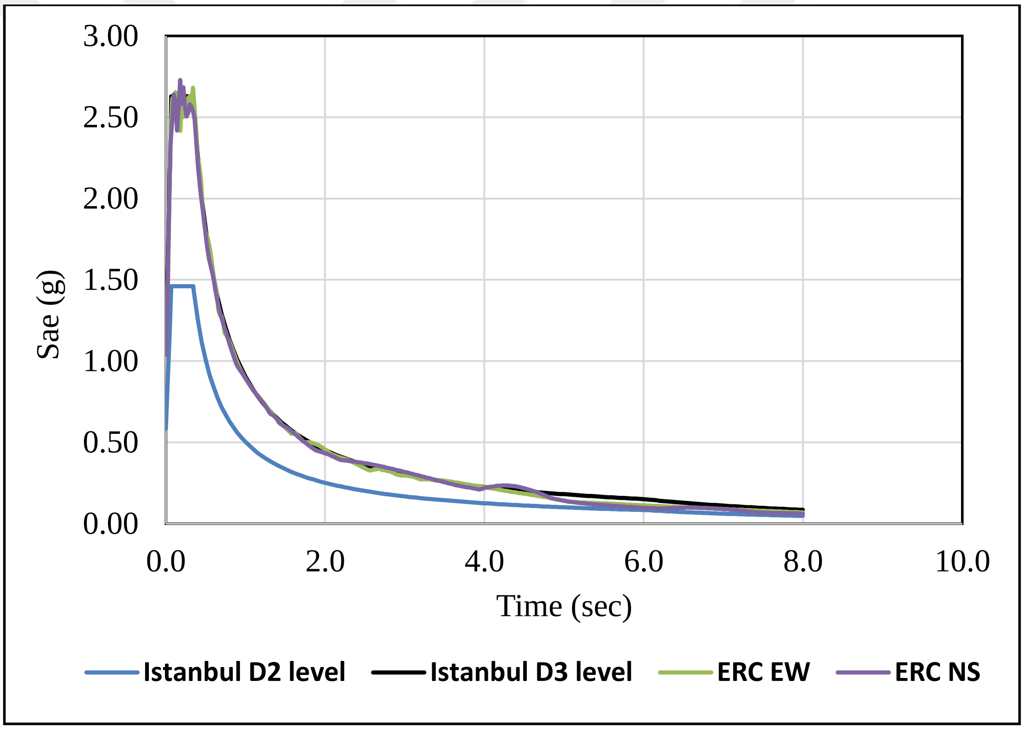


Figure 4.26: The target response spectrum for D3 level of earthquake with the matched spectrum of Erzincan earthquake.

5. DISCUSSIONS AND CONCLUSIONS

This thesis is prepared to study the seismic behaviour of tall buildings by following the performance-based design approach. This approach employs the dynamic nonlinear time history analysis to evaluate the seismic performance of the structure. Some discussion and conclusion points are extracted through preparing this study and depending on the obtained results, as following:

- The necessity of constructing tall buildings is growing consistently all over the world especially in urban areas. Turkey is one of these locations but for Iraq, this subject is considered a new era.
- Although there are a lot of studies that carried out in this field, the lack of experience still exists in understanding the actual behaviour of the tall buildings which are long period structures thus the contribution of higher modes are substantial.
- In USA, a lot of organizations and research centers have been working in this field to introduce an adequate and agreed guidelines to design engineers. PEER center is considered a significant reference in establishing the performance-based design guidelines for tall buildings.
- In Turkey, the interest in this field is considered a new orientation through the last ten years due to the increase of construction of these buildings especially in high seismic zones like Istanbul. ISCTB-2008 draft document is considered a first attempt in this field but it has not published yet due to some obstacles including the restriction of these guidelines for only Istanbul. The new draft version of Turkish Seismic Code-2016 has a chapter related to tall buildings for using in different seismic zones in Turkey.
- Performance-based design has some problems and challenges that design engineers face when using this approach in practice specially in the

transformation stage from the prescriptive code design to a more non-prescriptive performance-based design.

- In tall buildings, estimating the level of damage depending on the internal deformations and internal forces in the members are important to evaluate the seismic performance of structure. However, current conventional seismic codes do not permit estimating distribution of the expected damage level and internal forces accurately.
- Through the analysis, some technical problems were faced in utilizing ETABS software program. The direct integration method that should be used in time history analysis got stuck in analysing the 25-storey building after taking a really long time. For a model of three storeys, the analysis took one day to finish the analysing, however, for seven-storey building, it took three days with some null steps that affect the final results of the analysis thus the increasing in number of elements and joints is very complicated for this software.
- Nonlinear modal time-history analysis, also known as Fast Nonlinear Analysis (FNA), is performed in dynamic analysis of the building. This method is considered more accurate and efficient than direct-integration time-history analysis according to the viewpoint of the program designers and the accuracy of FNA depends upon the sufficiency of suitable mode shapes. The mystery of mathematical procedure that followed in this method make it not desirable, on the other hand, using a number of data in analysis and comparing the results with each other give a good feeling of confidence in using this method.
- Some analysis results of the case study are presented to evaluate the performance of the building in two different seismic zones. The results demonstrated the difference in seismic performance and loading case of both buildings due to the difference in seismic hazard of each location.
- Relative story drift ratio explains the overall performance of the building. For Istanbul building, it was around 1.2 %, however, for Iraq building, it was around 0.35% and both cases meet the required limits of 2.5% at that design stage.

- The plastic hinge states describe the internal deformation that control the performance objective of the building through the performance level of the sections. The results show that all columns and shear walls remained elastic under the effect of seven earthquake pairs. However a number of beams passed slightly the yield point without exceeding the immediate occupancy limit. Probably the main reason for this behaviour is related to the plenty of shear walls in the plan which absorbed the greatest influence of the earthquake.

- The results indicated that this case study is a well-designed building even if it will be constructed in high seismic zone area. On the other hand, employing the same structural system for building to be built in low seismic zone is considered, from a structural engineering viewpoint, exaggerated and uneconomic. For that reason an alternative system is created after some changes and analysed for checking the new performance of the building. The results showed that there is no significant influence to the seismic performance of the building but this alternative design saved around 10,000 \$ (35,000 TL) through decreasing about 187 m^3 of concrete and 2 m^3 of reinforcement steel and gave a motivation to the designer to reconsider and find an adequate design that provides an acceptable performance with taking the economic side into account.

- The last design stage which is considered a verification stage is performed to Istanbul building and the overall performance of the building is evaluated through the relative story drift which meet the acceptance criteria of the required performance level with a slight passing for the immediate occupancy performance level under subject of linear response spectrum for D2 earthquake level in Y direction.

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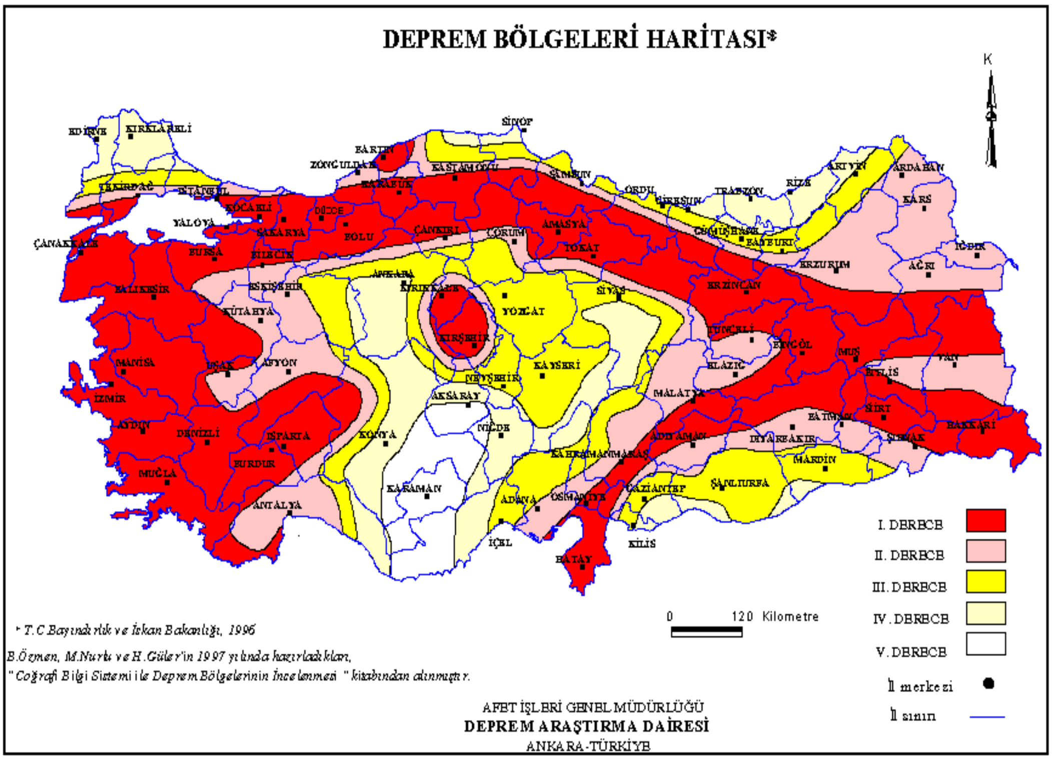
APPENDICES

APPENDIX A: Maps

APPENDIX B: Structural System and Reinforcement Details of the Case Study

APPENDIX C: Analysis Data and Results

APPENDIX A

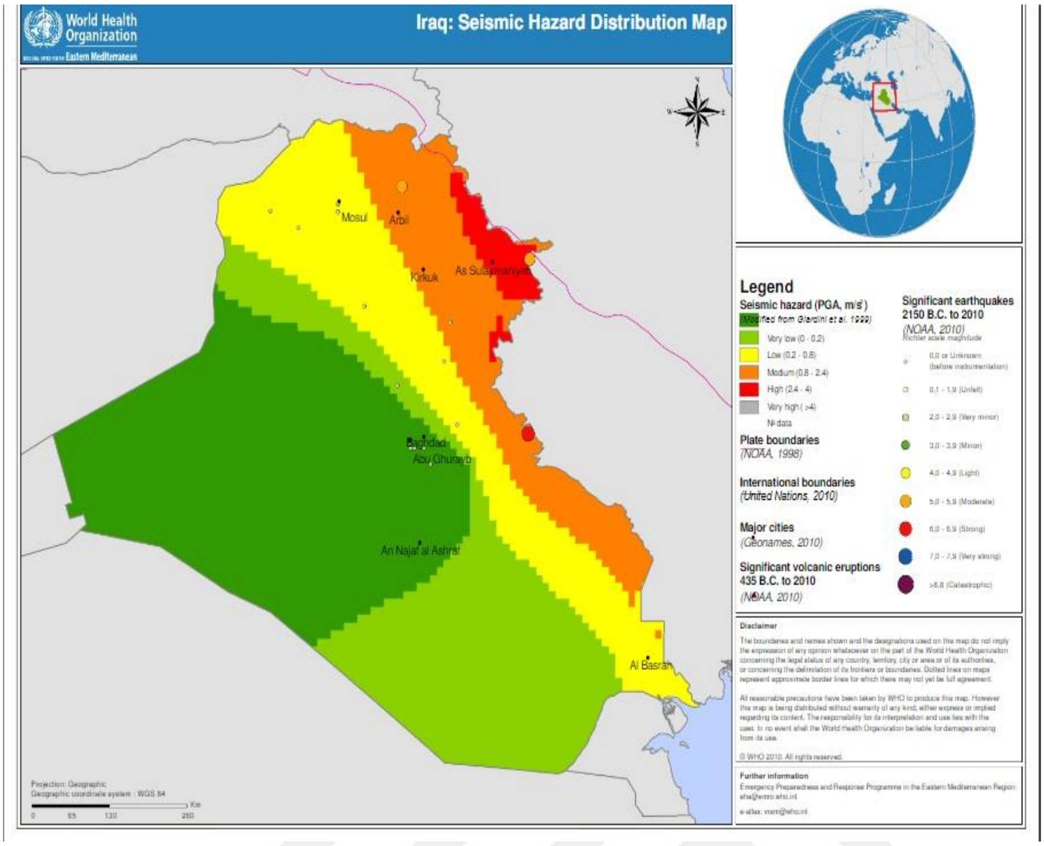


* T.C Bayındırlık ve İskan Bakanlığı, 1996

B.Özmen, M.Nurlu ve H.Güler'in 1997 yılında hazırladıkları, "Coğrafi Bilgi Sistemi ile Deprem Bölgelerinin İncelenmesi" kitabından alınmıştır.

AFET İŞLERİ GENEL MÜDÜRLÜĞÜ
DEPREM ARAŞTIRMA DAİRESİ
ANKARA-TÜRKİYE

Figure A.1: Map of earthquake zones of Turkey.



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eha@emro.who.int
e-atlas: [illegible]@who.int

Figure A.2: Seismic hazard distribution map of Iraq.

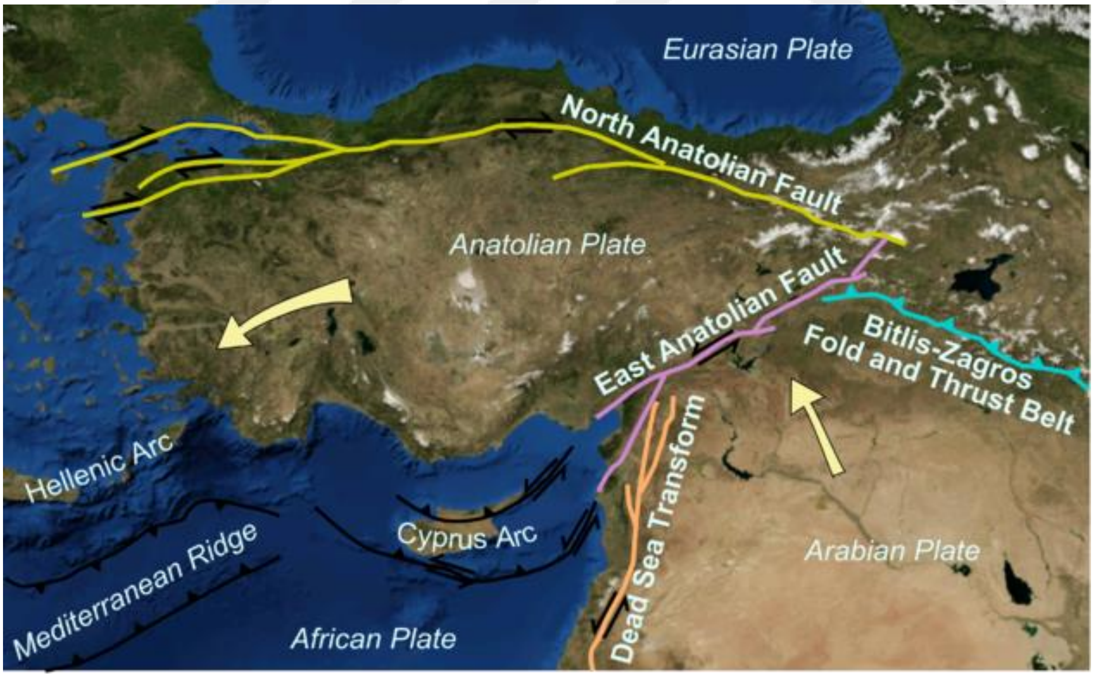

Figure A.3: Map of plate boundaries affecting Turkey and northern part of Iraq.

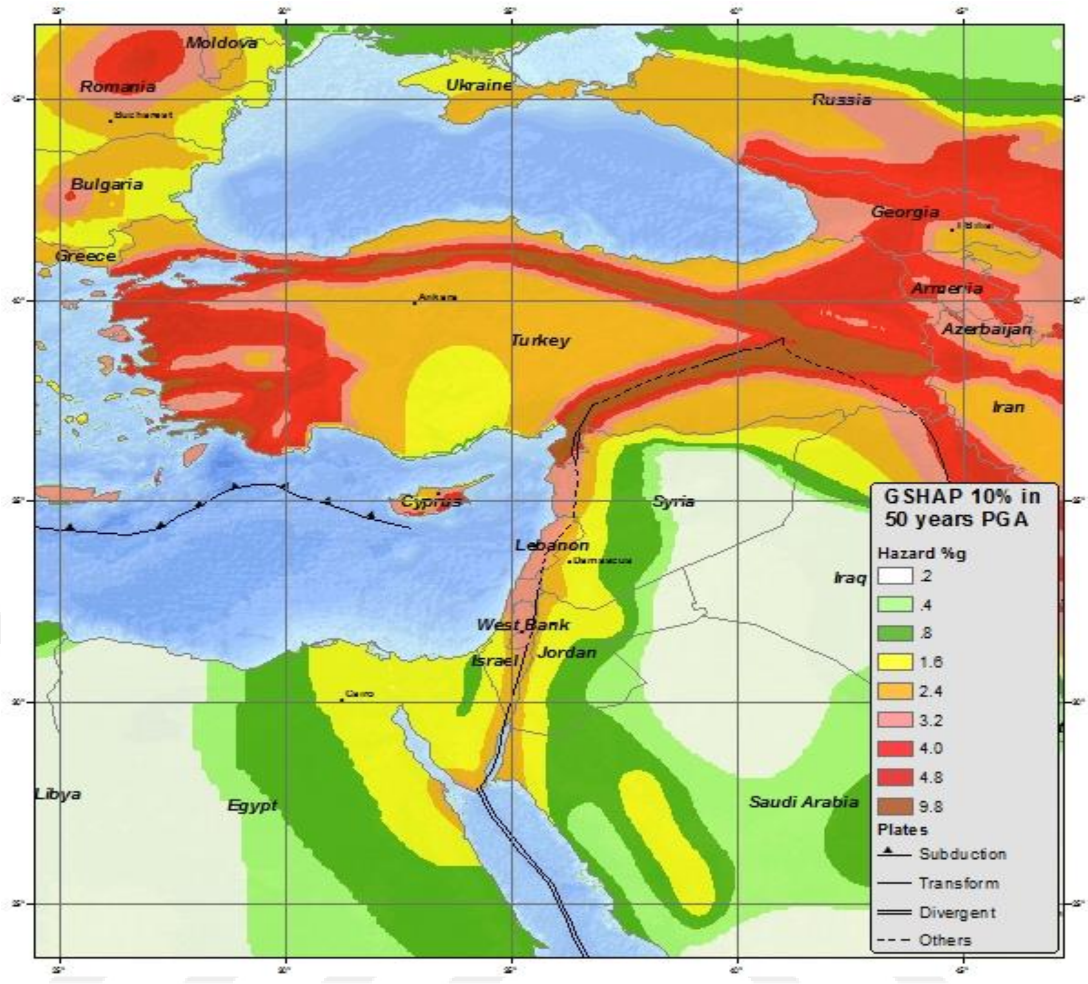

Figure A.4: Map of seismic hazard for Turkey and the Middle East.

APPENDIX B

Table B.1: The section dimensions of columns and shear walls.

Section label	Dimensions, mm
C01/C02	400 x 1200
C03/C04/ C12/C13	400 x 1000
C05/C08	900 x 500
C06/C07	500 x 1250
C09/C10	1550 x 300
C11/C14	850 x 600
W01/W02	2500 x 350
W03/W06	2450 x 350
W04/W05/ W07/W08	3500 x 300
W09/W20	300 x 3000
W10/W17/ W11/W18	300 x 2500
W12/W13/W14/W15/W16/W19	300 x 2100

Table B.2: Beam sections details.

Beam label.	Dimensions, mm
KZ01, 02, 05, 06, 07, 26, 27, 28, 29, 31, 36, 45, 46, 47, 50, 51, 52, 65	250 x 500
KZ03, 04	700 x 330
KZ08, 11, 14, 17, 22, 23, 24, 41, 42, 43, 44, 48, 49, 53, 54, 55, 56, 57, 58	300 x 500
KZ09, 10, 12, 13, 15, 16, 18, 19, 20, 21, 25, 30, 32, 33, 35, 37, 38, 39, 40, 59, 60, 61, 62, 64, 66, 67	600 x 330
KZ34, 63	850 x 330

Table B.3: Slab sections details.

Slab label.	Type	Depth, mm
NZ01, 02, 03, 04, 05, 06, 07, 08, 09, 10, 11, 12, 13, 17, 18, 19, 20, 21, 25, 26, 27, 28, 29, 31, 33, 34, 35, 36, 37, 38.	Ribbed slab	80
NZ14, 15, 16, 22, 23, 24, 30, 32.	Solid slab	160

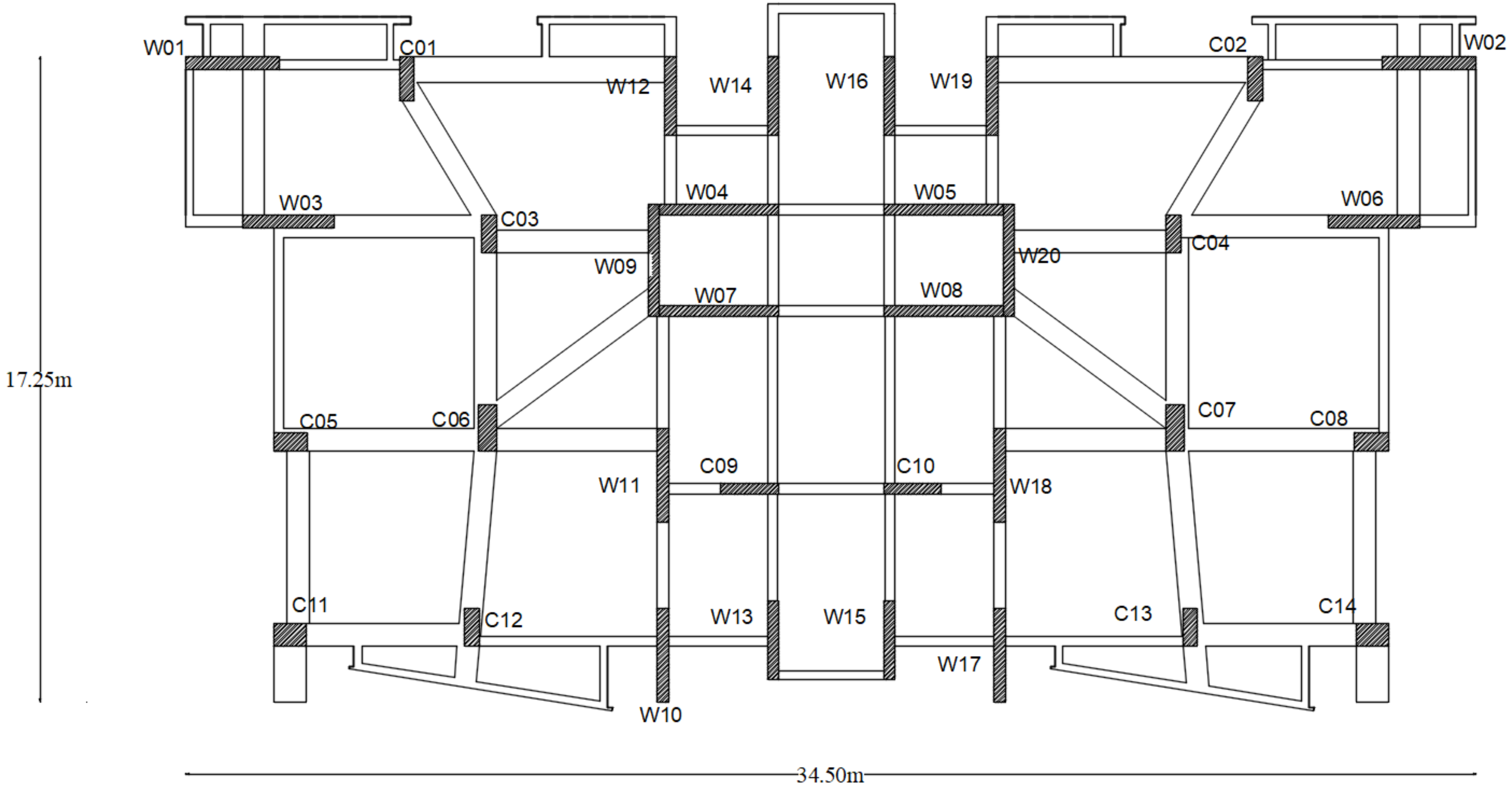

Figure B.1: Distribution of columns and shear walls in the ground and typical floor of the building.

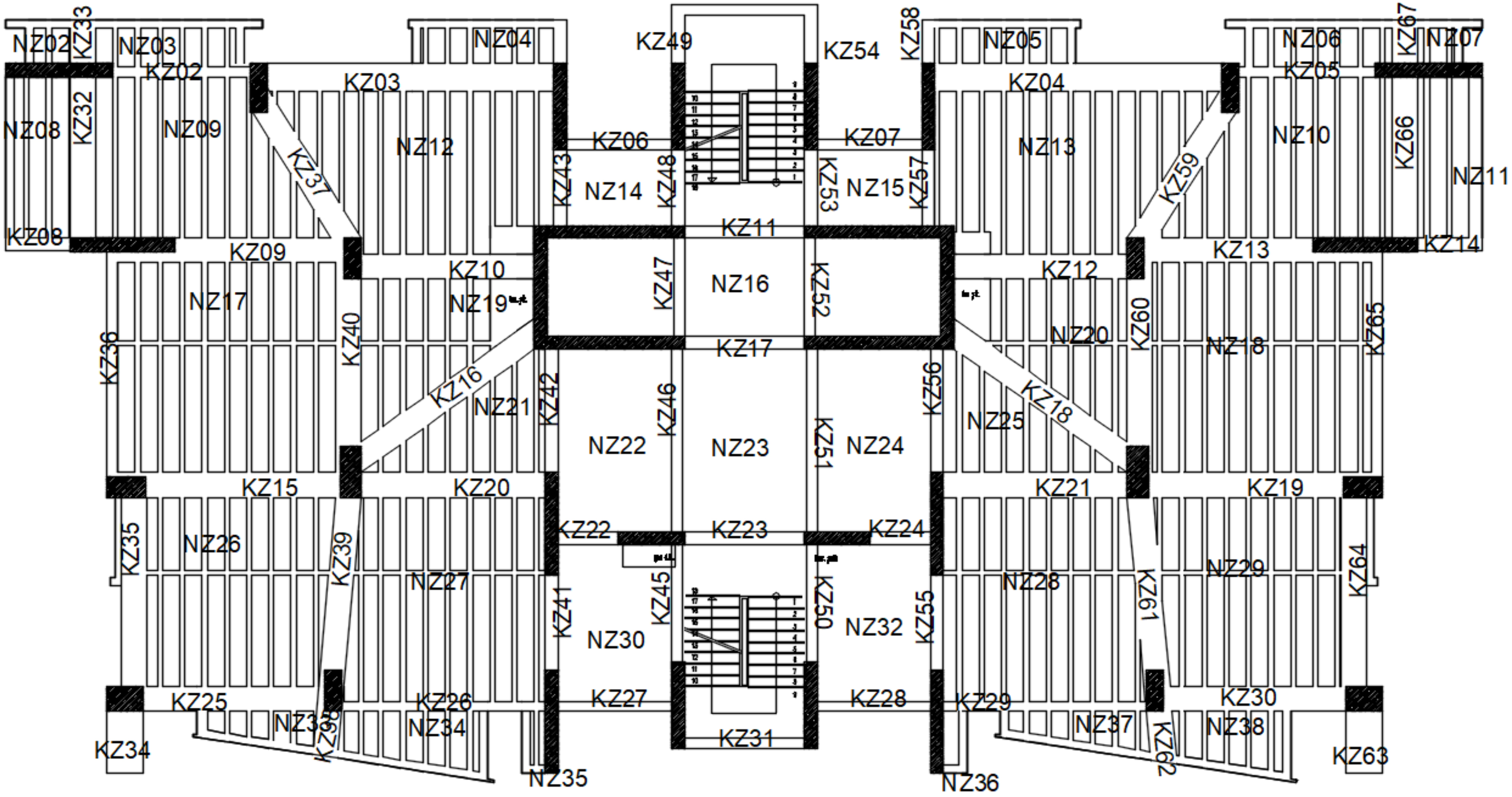

Figure B.2: Beam and slab sections of the ground and typical floor of the building.

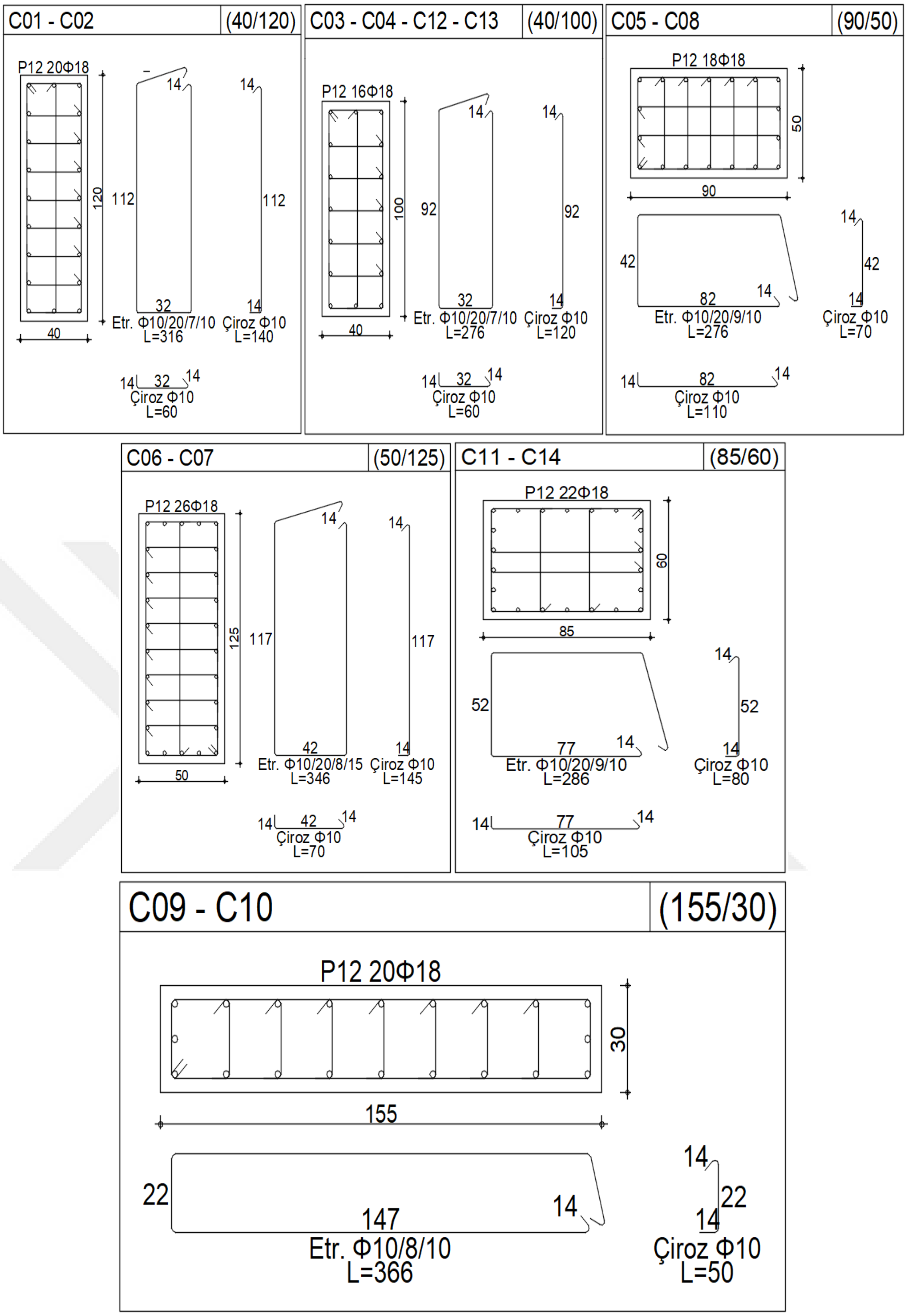

Figure B.3: Reinforcement details of column sections.

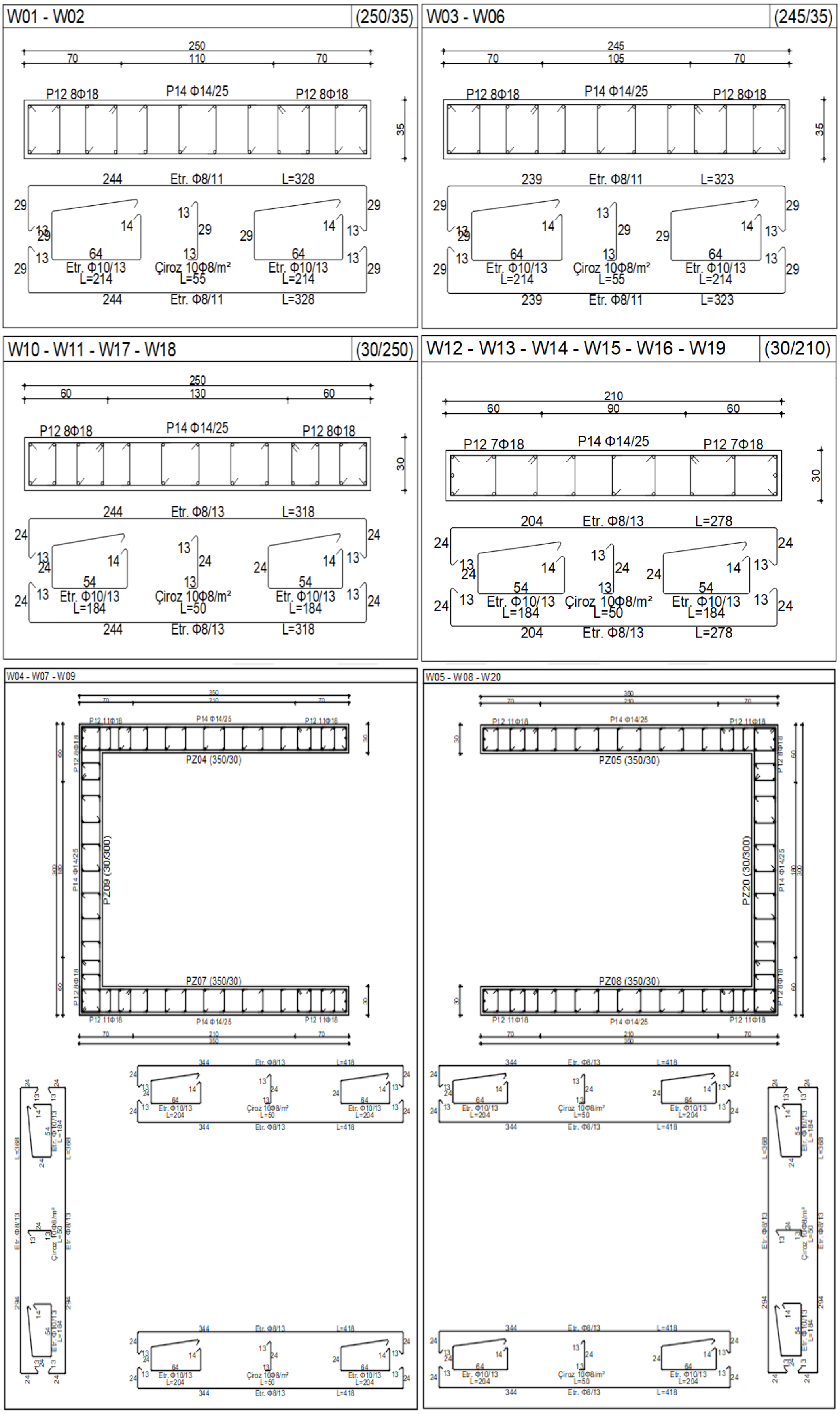

Figure B.4: Reinforcement details of shear wall sections.

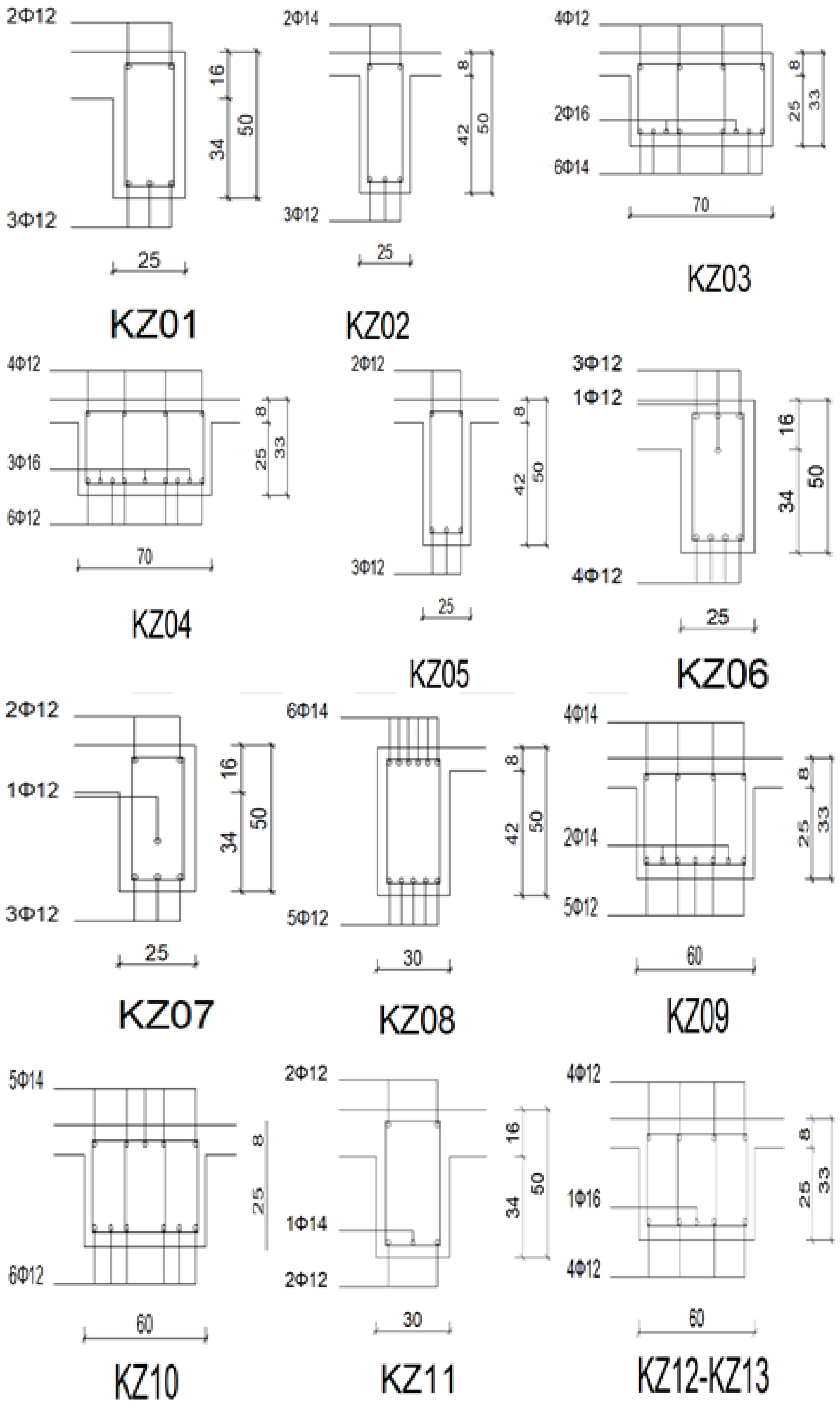

Figure B.5: Reinforcement details of beam sections (KZ01-KZ13).

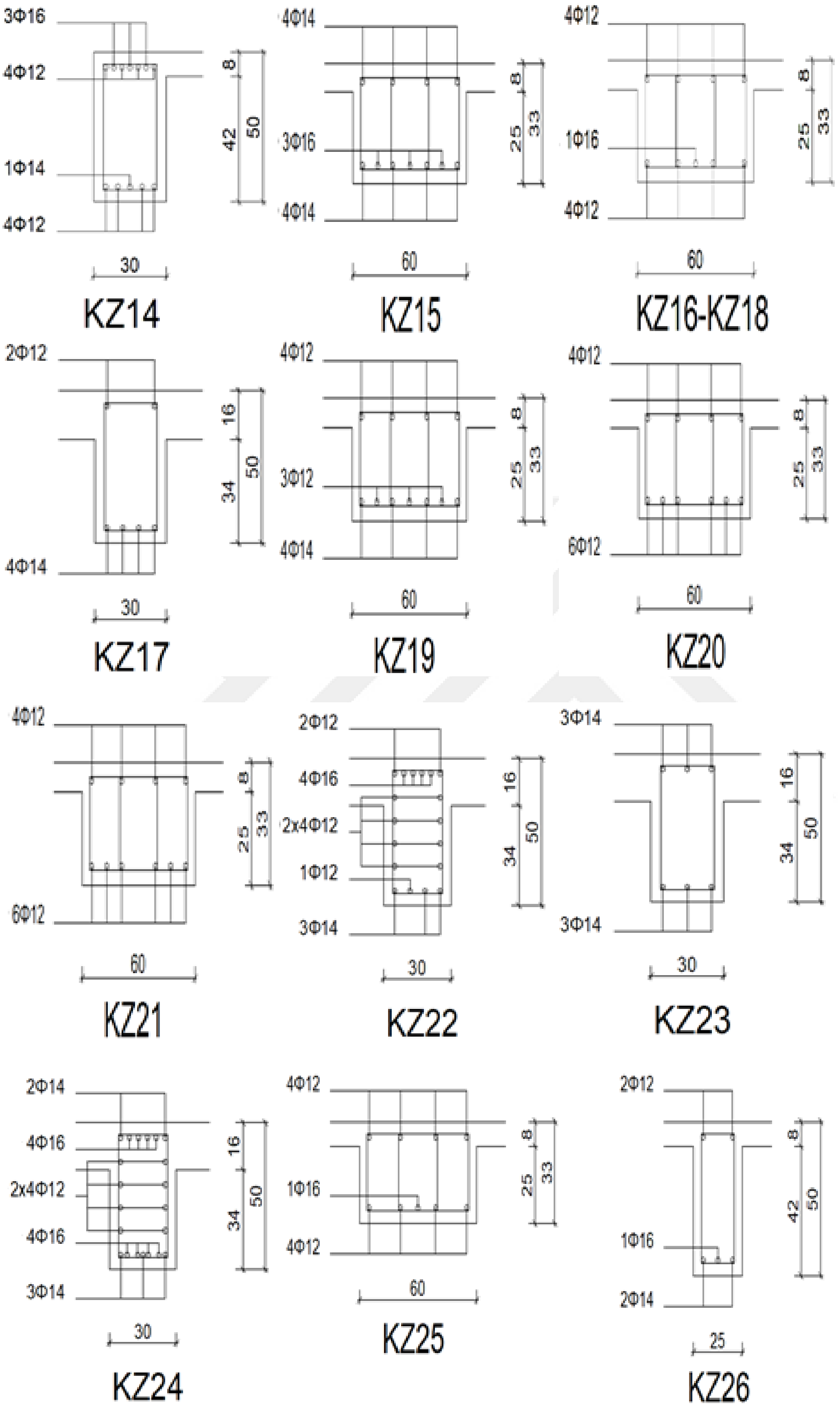

Figure B.6: Reinforcement details of beam sections (KZ14-KZ26).

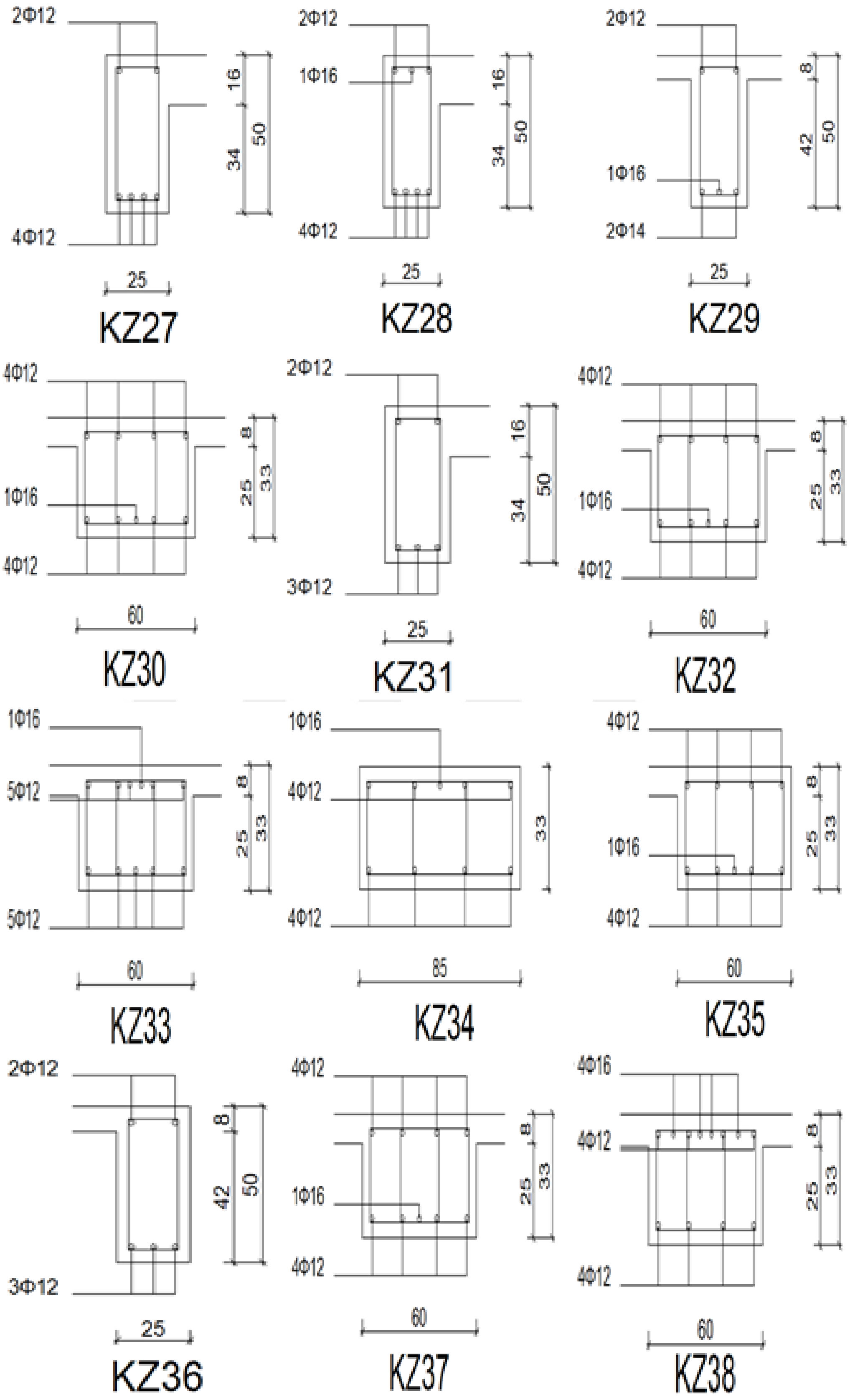

Figure B.7: Reinforcement details of beam sections (KZ27-KZ38).

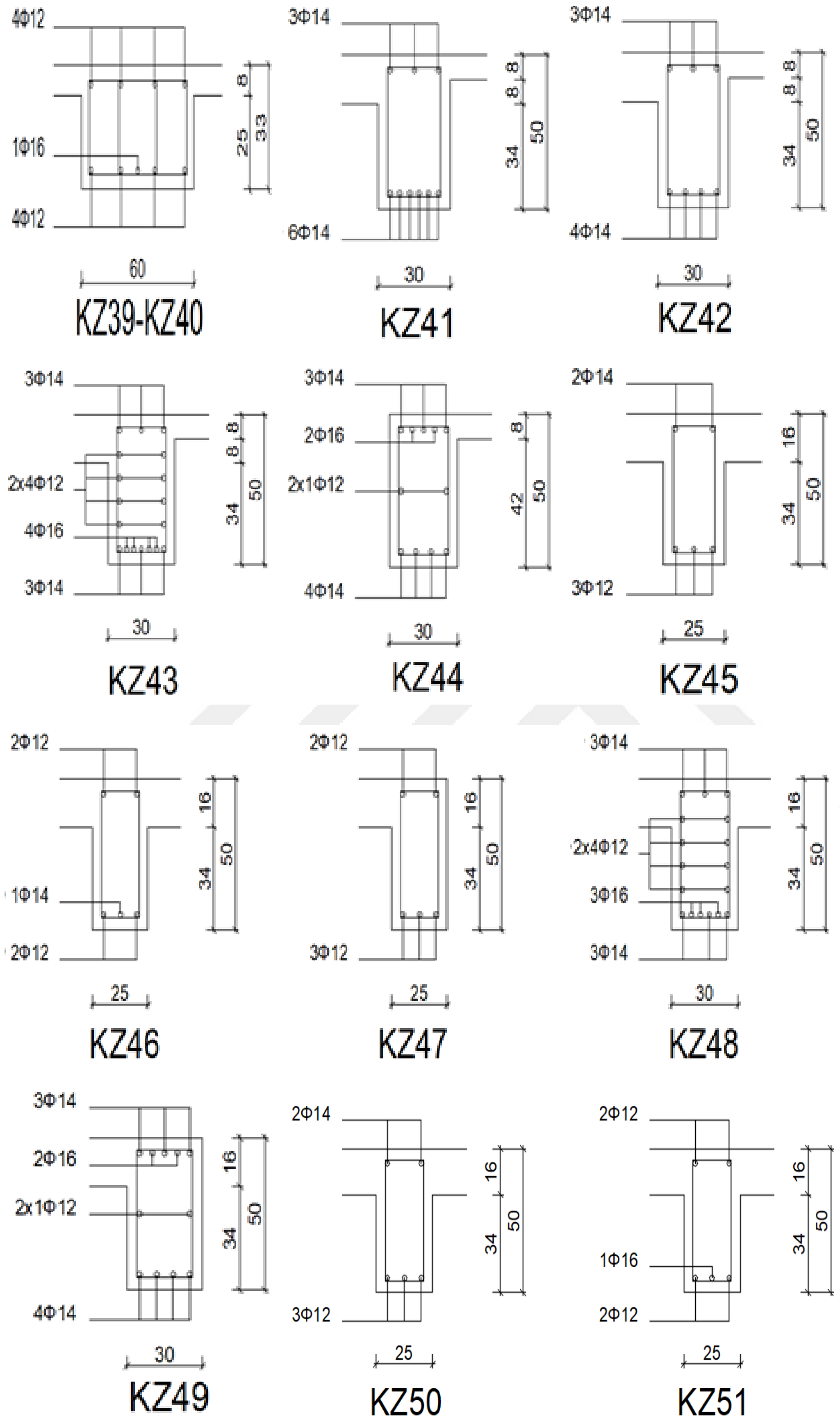

Figure B.8: Reinforcement details of beam sections (KZ39-KZ51).

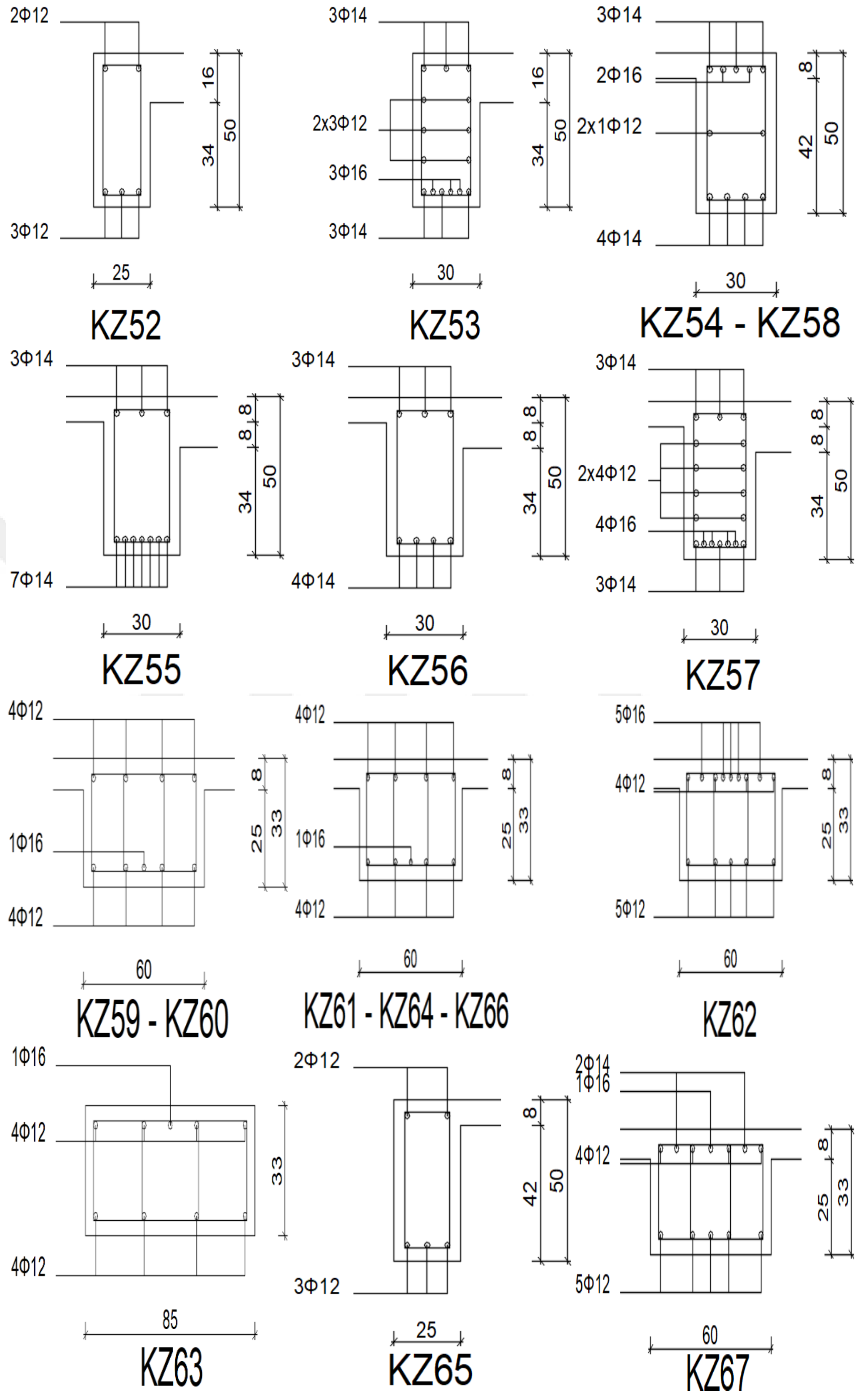

Figure B.9: Reinforcement details of beam sections (KZ52-KZ67).

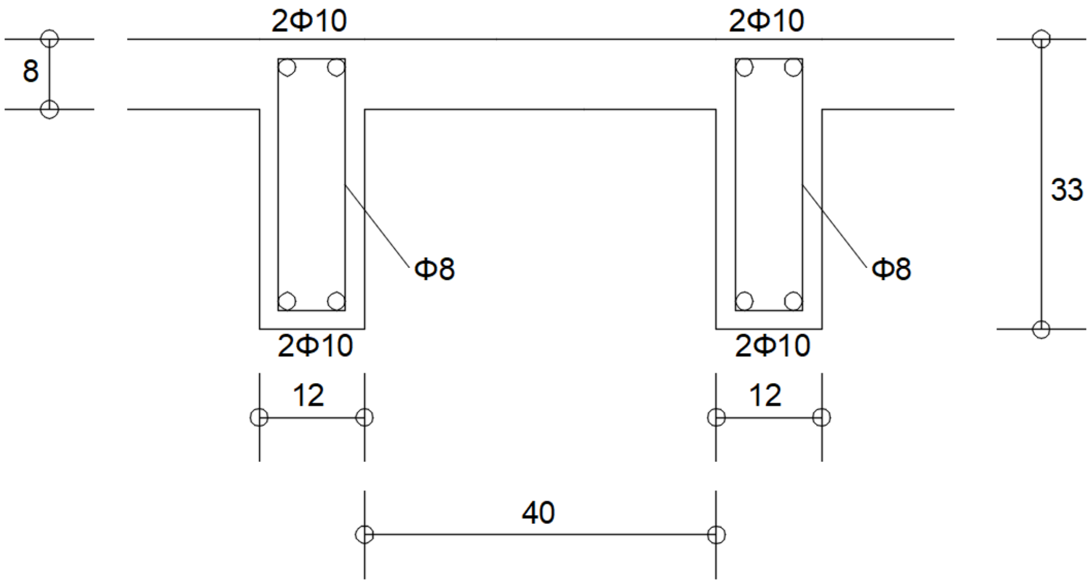

Figure B.10: Section details of the ribbed slab.

APPENDIX C

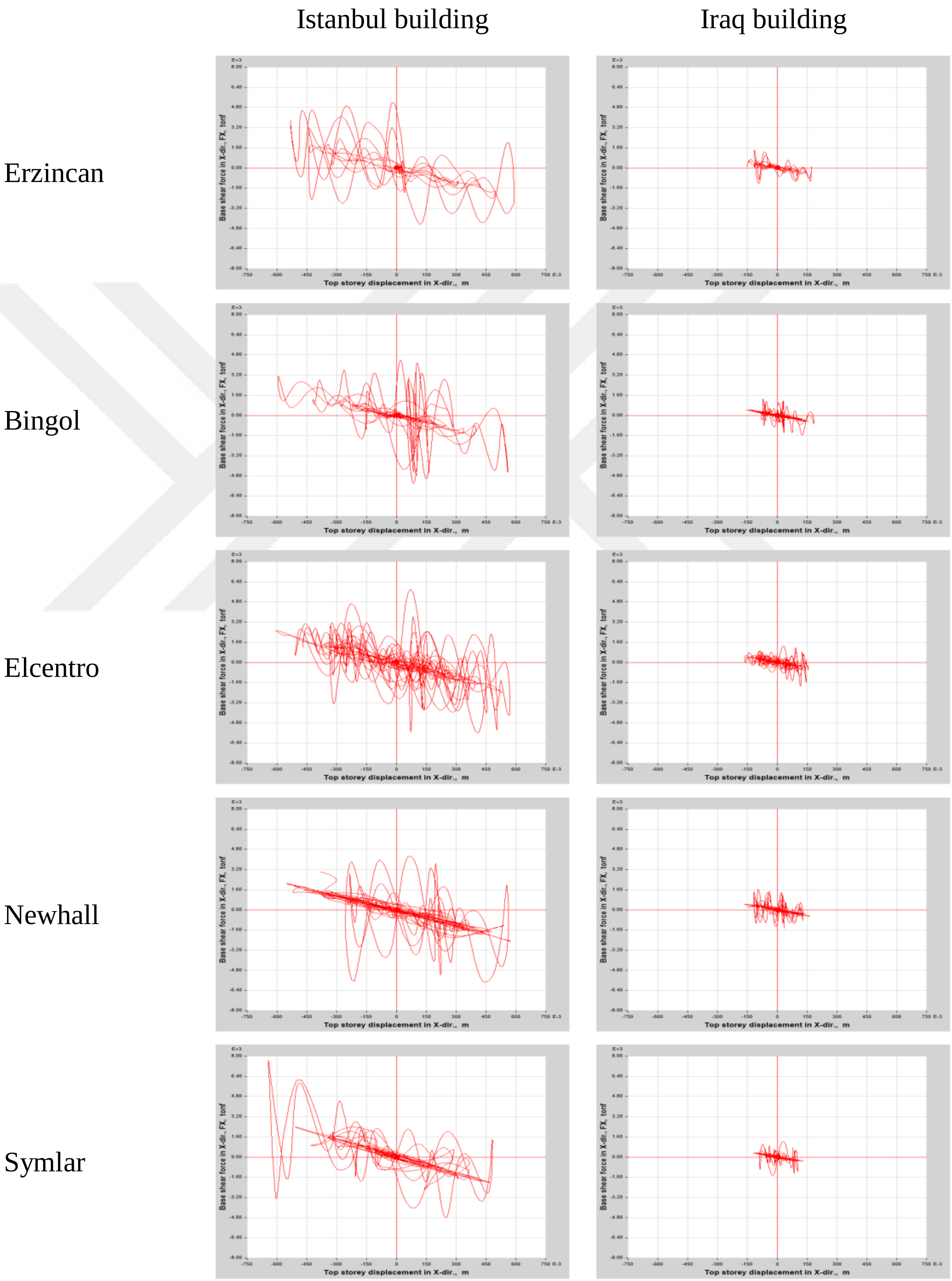

Figure C.1: Base shear force vs top storey displacement of the building in X-direction.

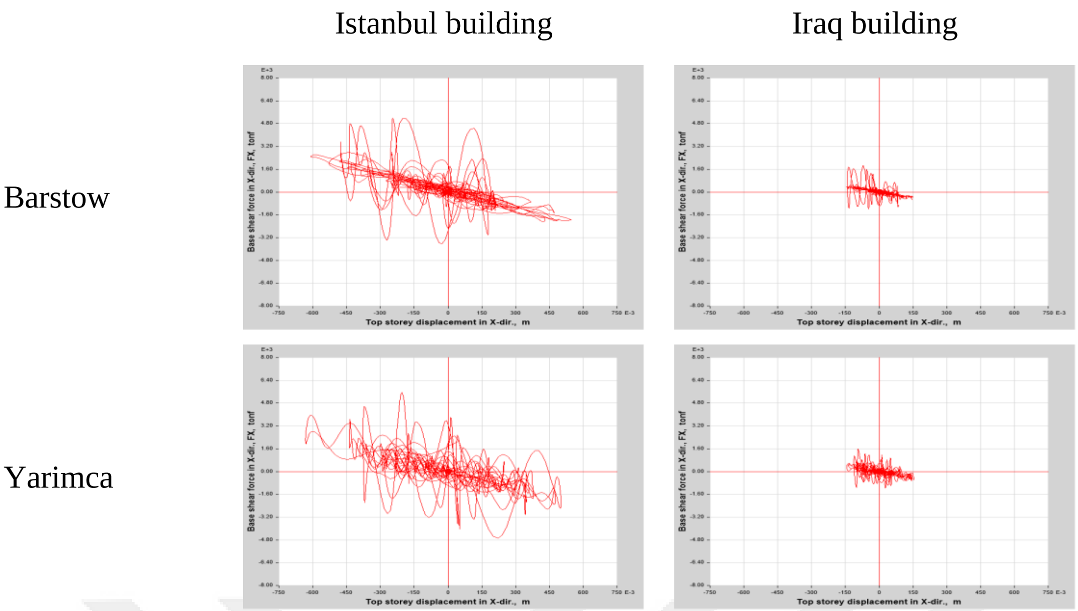

Figure C.1 (continued): Base shear force vs top storey displacement of the building in X-direction.

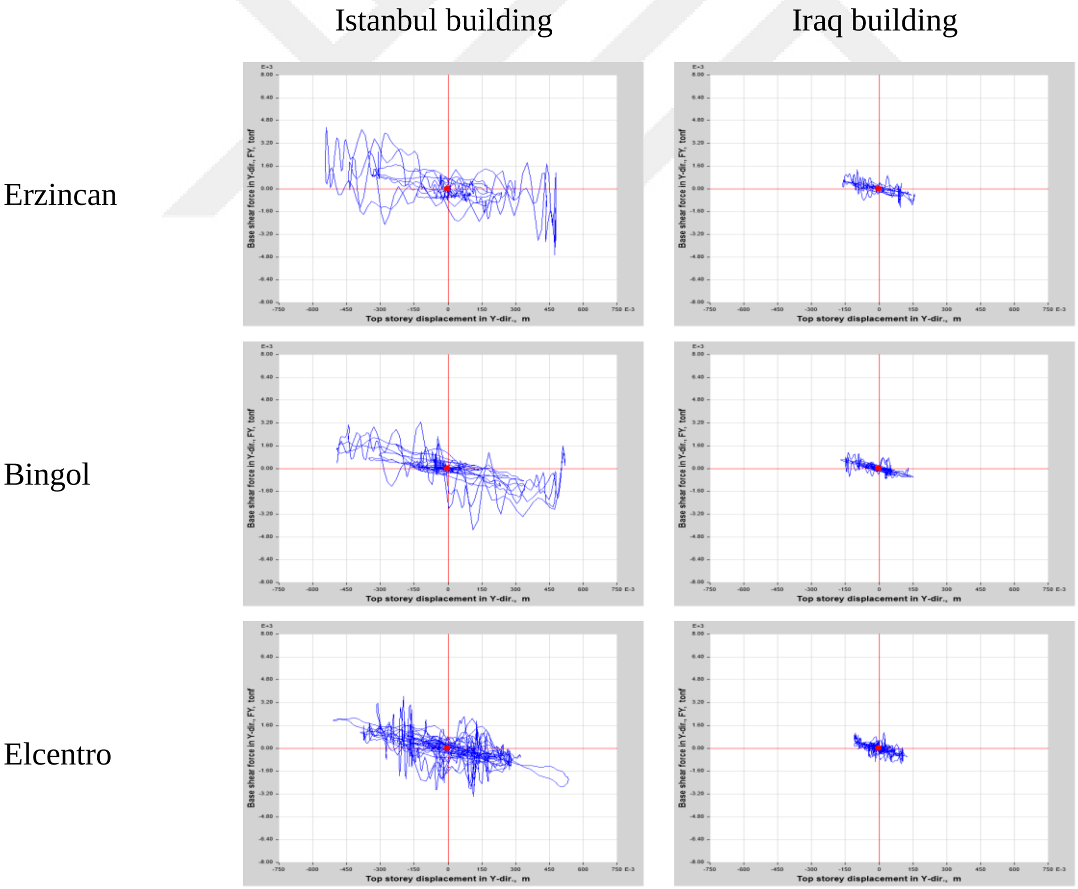

Figure C.2: Base shear force vs top storey displacement of the building in Y-direction.

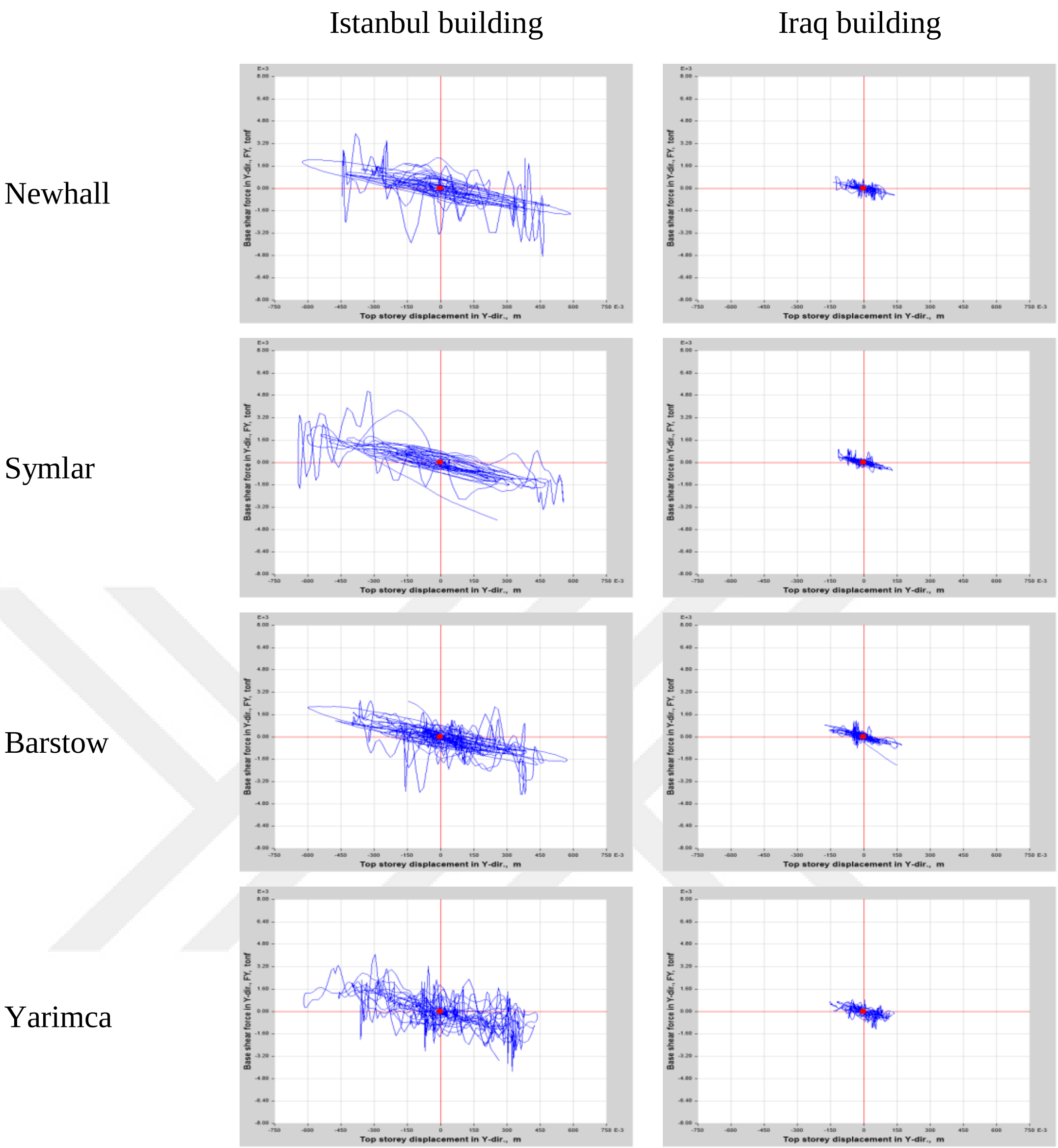

Figure C.2 (continued): Base shear force vs top storey displacement of the building in Y-direction.

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