ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

SEISMIC PERFORMANCE EVALUATION OF 24 STORY RC BUILDING BY NONLINEAR TIME HISTORY ANALYSIS UTILIZING TBDY2018 AND EC8

M.Sc. THESIS

Jeton BUZUKU

Department of Civil Engineering

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ISTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

24 KATLI BETONARME BİR BİNANIN DEPREM PERFORMANSININ ZAMAN TANIM ALANINDA LİNEER OLMAYAN YÖNTEMLE TBDY2018 VE EC8'E GÖRE BELİRLENMESİ

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Aralık, 2019

Jeton BUZUKU, a M.Sc. student of İTU Graduate School of Science Engineering and Technology student ID 501171024, successfully defended the thesis/dissertation entitled "SEISMIC PERFORMANCE EVALUATION OF 24 STORY RC BUILDING BY NONLINEAR TIME HISTORY ANALYSIS UTILIZING TBDY2018 AND EC8" 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 : 15 November 2019
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To my family,

FOREWORD

I would first like to thank my thesis advisor Dr. Beyza TAŞKIN. The door to Prof. Beyza office was always open whenever I ran into a trouble spot or had a question about my research or writing. She consistently allowed this paper to be my own work, but steered me in the right direction whenever she thought I needed it.

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December 2019

Jeton BUZUKU
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ABBREVIATIONS

BHB	**:** Significant Damage Zone
BKS	**:** Object Usage Level
BYS	**:** Object Height Level
DD	**:** Earthquake Level
DGT	**:** Strength Based Design
DTS	**:** Earthquake Design Level
Etabs	**:** Extended Three-Dimensional Analysis of Building Systems
Eurocode8	**:** Seismic Design of Buildings
GB	**:** Collapse Zone
GÖ/NC	**:** Near collapse
I	**:** Object Importance Coefficient
IHB/SD	**:** Advanced Damage Zone
BHB	**:** Controlled Damage
KK	**:** Uninterrupted Usage
PEER	: Pacific Earthquake Engineering Research Center
PGA	**:** Peak Ground Acceleration
SH	**:** Limited Damage Performance Level
BHB/DL	**:** Limited Damage zone
ŞDGT	**:** Deformation Based Design
TBDY	**:** Turkish Earthquake Structural Code
XTRACT	**:** Cross-sectional X Structural Analysis of Components

SYMBOLS

A_c	: Concrete gross area
Δ	: Story Displacements
$\Delta t(s)$	: Earthquake data time interval
(EI)e	: Cracked Section bending stiffness
ε_c	: Concrete strain value
ε_{cu}	: Confined concrete ultimate strain
ε_{sh}	: Steel strain at hardening point
ε_{su}	: Steel ultimate strain
ε_{sy}	: Steel yield strain
f_c	: Confined concrete stress
f_{cc}	: Confined concrete strength
f_{cm}	: Existing concrete strength
f_{co}	: Unconfined concrete strength
f_s	: Reinforcing steel stress
f_{sy}	: Reinforcing steel yield strength
f_{su}	: Reinforcing steel ultimate strength
F_1	: Soil effect coefficient for 1s period
F_s	: Soil effect coefficient for short period
G	: Dead load
h	: Height
h_i	: Story height
L_p	: Plastic hinge length
m_t	: Structure total mass
M_y	: Reinforcing yield moment
M_p	: Plastic moment
N_D	: Column axial force
$S_{ae}(T)$	: Horizontal elastic design acceleration
S_{DS}	: Design spectral acceleration coefficient for short period
S_{D1}	: Design spectral acceleration coefficient for 1 s period
S_S	: Map spectral acceleration coefficient for short period

T_A	: Horizontal elastic spectrum edge period
T_B	: Horizontal elastic spectrum edge period
T_L	: Horizontal elastic spectrum starting point of constant deformation spectrum
T_P	: Structures vibration period
$T_{1,x}$	: First mode X direction period
$T_{1,y}$	: First mode Y direction period
u_t	: Top lateral displacement
V_D	: Design or shear wall shear force
V_{tE}	: Total equivalent seismic load
$Ø_P$	: Plastic curvature
$Ø_t$	: Total curvature
$Ø_y$	: Yield equivalent curvature
θ_P	: Plastic rotation
θ_t	: Total rotation
Q	: Live load

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SEISMIC PERFORMANCE EVALUATION OF 24 STORY RC BUILDING BY NONLINEAR TIME HISTORY ANALYSIS UTILIZING TBDY2018 AND EC8

SUMMARY

Last years we were witnessing that earthquakes are causing enormous damages to the structures and communities. These damages are the reason for the necessity to evaluate performance analysis. Performance analysis aims to check the availability of the structure. Seismic Design Codes, depending on seismic risk and the structure itself, provide different methods to perform the analysis.

The position of Turkey in a manner of faults is considered as one of the most hazardous countries in the world. This fact caused the researchers to become more active in the fields of Structural and Earthquake Engineering. Last year they came up with an updated Seismic Code called TBDY2018. The code has some fundamental differences when compared to the old design code.

This thesis discusses the evaluation of performance analysis of 24 Story High-rise Structure, which is supposed to be constructed in Kağıthane district in Istanbul. The performance analysis is done using Time History analysis according to TBDY2018 and EC8.

The thesis includes six chapters where the first chapter is the entrance. In the first chapter, some necessary information about the structure and the purpose of the thesis are discussed.

The second chapter is the chapter about performance analysis. All information about the performance analysis definitions, according to TBDY2018, are given. It starts with the material models of concrete and reinforcement steel, defining their characteristics and stress-strain graphics. The following parts deal with the behavior of RC elements under bending moment loads also defining moment-curvature relations. It continues with the plastic hinge hypothesis explaining it in detail. Another topic argued in this chapter is the information about the structure required to perform the analysis. The damage limits, damage zones and performance targets of the structure are also defined in this part. The last part of this chapter talks about the methods used to perform the analysis. They are divided into two main groups: Linear and Non-Linear methods.

The third chapter is the chapter of seismic loads. In the beginning, Seismic ground motion levels are defined following with horizontal elastic spectrum defined using TBDY2018. The following part explains everything about the seismic loads used for performance analysis. In order to perform the non-linear time history analysis, 11 different earthquake strong ground motion data are required. Data were taken from the Pacific Earthquake Engineering Research Center (PEER) database, and they are matched to the response spectrum using Seismomatch Software.

Forth chapter is a summary of the second and third chapters, according to EC8. Only the differences of EC8 are discussed.

The fifth part is the most crucial part of the thesis. This chapter discusses the performance evaluation of the 24 Story high-rise structure, whose project was given by the mentor of the thesis. Firstly, general information about the structure are given such as section properties and material properties. The following part shows the mathematical model of the structure that was done using ETABS software. After that model response spectrum is defined using TBDY2018 and EC8. The plastic hinge hypothesis is discussed in details. Firstly, it was calculated manually using XTRACT, and the result is compared to the ETABS automatic assignments. It is considered that ETABS calculations are acceptable and manual assignments are not required. This procedure was done for columns, beams, and shear-walls. After defining the vertical and 11x2 seismic loads as non-linear time history loads, the analysis was performed. The results were discussed in two fields, based on structure performance and based on elements performance. Performance evaluation based on elements was calculated for every element and every element type, a calculation example was shown.

In the last chapter, after evaluating the performance of every element using the rules provided by TBDY2018 and EC8, it was concluded to which damage zone the structure belongs. Also, there are given some recommendations for this type of analysis.

24 KATLI BETONARME BİR BİNANIN DEPREM PERFORMANSININ ZAMAN TANIM ALANINDA LİNEER OLMAYAN YÖNTEMLE TBDY2018 VE EC8'E GÖRE BELİRLENMESİ

ÖZET

Türkiye'nin son aktif bir sismik bölgede olması ve yıkıcı depremler yaşanması, depreme dayanıklı yapı tasarımının bir zorunluluk olduğunu göstermiştir. Bu depremlerle mevcut doğrusal elastik analiz yönteminin yeterli olmadığını da anlaşılmıştır. Ülkemiz aktif fay hattında bulunmaktadır. Bu nedenle, yakın geçmişte meydana gelen depremlerin yıkıcı zararları, mevcut binaların büyük çoğunluğunun kanunlara uygun şekilde inşa edilmediğini göstermiştir. Ayrıca, depremlerden sonra yapılan incelemeler ve gözlemler bu yapılarda kullanılan beton dayanımın çok düşük olduğunu ve donatı kalitesinin yetersiz olduğunu göstermiştir. Ayrıca, iyi imalat işçiliğinin olmaması, yapılarımızın depremlerin etkisinde zayıf kalmasına neden olmuştur. Mevcut tüm bina stoklarının ulusal ekonomi ve sosyal yaşam koşulları açısından imha edilmesi ve yeniden yapılandırılması mümkün olmadığından, bu tür yapıların çeşitli güçlendirme yöntemleri ile güvence altına alınması mümkündür. Bu güçlendirme tekniklerinden biri olan yapının uygun kısımlarına betonarme dolgu duvar eklenmesi, yapı sağlamlığına önemli bir katkı sağlayacak ve yapı performansını artıracaktır. Ayrıca, yapı mühendisliği ve gelişen bilgisayar teknolojisinde yapılan çalışmalar binanın performans yeterliliğini daha gerçekçi bir yaklaşımla değerlendirmeyi mümkün kılmıştır. Bu tezde, 2007 DBYBHY göre inşa edilen bir beton yapının performans analizi yapılmıştır. Yapısal davranışlara elastik davranışların ötesine dikkat etmek, yapıların deprem sırasındaki hasarları ve dayanımı hakkında daha fazla fikir verir, sonuç olarak doğrusal analize ek olarak, doğrusal olmayan analiz yöntemi geliştirilmiştir. Mevcut yapının doğrusal elastik değerlendirilmesi, çöküş önleme performans seviyesine neden oldu. Altı bölümden oluşan tezin birinci bölümü, İstanbul Kağıthane semtinde inşa edıleceği 24 katlı bir gökdelenin Yapı performans analizinin değerlendirilmesini tartışmaktadır. Performans analizi, TBDY2018 ve EC8'e göre Geçmiş Zaman analizi kullanılarak yapıldı. Bu tez altı bölümden olup, birinci bölümün giriş olmaktadır. Birinci bölümde, tezin yapısı ve amacı hakkında bazı gerekli bilgiler tartışıldı.

İkinci bölümde, TBDY-2018 Bölüm 15'te, depremden etkilenen mevcut binaların davranışlarının ve bu binaların güçlendirilmesi için hesaplama ilkelerinin belirtildiği belirtilmiştir. Düzenleme ilkelerine göre, mevcut binaların değerlendirilmesi için bilgilerin toplanması, yapı elemanlarının hasar sınırlarının ve bölgelerinin belirlenmesi, bina için performans seviyeleri ve yöntemlerin temel ilkeleri ve hesap kuralları analiz edilmek üzere sunulur. Binanın mevcut projeleri ve yerinde yeterli bilginin sağlanması sonucunda bilgi seviyesi yoğun olarak belirlenmiş ve bilgi seviyesi bir katsayı olarak alınmıştır. Bilgi seviyesi katsayısı 1 olduğundan, yapıdaki mevcut malzemelerin analizleri herhangi bir azalma olmadan yapılır.

Üçüncü bölümde, TBDY-2018'in yer hareketlerine ilişkin tanımları ve ilgili ilkeleri tartışılmış ve performans analizinde kullanılacak zaman geçmişinde doğrusal olmayan dinamik analiz için gerekli olan deprem kayıtlarının ölçeklendirilmesi anlatılmıştır. Bu ilkelere göre, seçilen kayıtların yapı için elde edilen yatay elastik tasarım spektrumuna uyacak şekilde ölçeklendirilmesi gerekir. Seismomatch yazılımı yardımıyla ivme kayıtlarının sıklık içeriği değiştirilerek ölçeklendirme yapıldı. Böylece, kayıtların, deprem hareketinin fiziksel özelliklerini kaybetmemesi ve neredeyse önceden belirlenen belirlenmiş bir zaman aralığında tasarım spektrumuna uyması sağlanır. TBDY-2018 ilkelerine göre yapılan analizler sonucunda her deprem kaydından elde edilen sonuçların anlamlı bir fark yaratmadığı tespit edilmiştir. Böylece seçilen gerçek deprem kayıtlarının uygun olduğu ve ölçeklendirme işleminin doğru yapıldığı görülmektedir. Bu deprem kayıtları, zaman diliminde 0.2T ile 1.5T periyotları arasında ölçeklendirilmekte ve ölçeklemenin sebepleri ayrıntılı olarak açıklanmaktadır. Her deprem olayının yer hareketi kayıtları Pasifik Deprem Mühendisliği Araştırma Merkezi'nden alınmıştır. Tepki spektrumu eğrileri, Seismomatch Computer Software kullanılarak hesaplanır ve sonuç spektrumu, earthqauke kayıtlarının tepki spektrumu eğrileri kullanılarak kareler yönteminin toplamının karekökü kullanılarak hesaplanır. Elde edilen bu spektrum bir ölçek faktörü ile büyütülür. Tüm ölçeklendirilmiş cevap spektrum ivme değerlerinin ortalaması, yanıt spektrumunun ivme değerlerinin 1,3 katından büyüktür.

Dördüncü bölüm, EC8'e yöntemine göre ikinci ve üçüncü bölümlerin bir özetidir. Sadece EC8'in farklılıkları tartışılmaktadır.

Beşinci bölümde betonarme binanın doğrusal olmayan dinamik analize göre performans analizi yapılmıştır. Yönetmeliğin gereklerine uygun olarak kaydedilen onbir deprem iki yönde uygulanmış ve çözümler üretilmiştir. Yığılı plastik mafsal özellikleri tüm kirişlerde ve kolonlarda, perde duvarlarında ise yayılı plastik mafsal özellikleri tanımlanmıştır. Sütunlar eksenel yük - çift eksenli eğilme momenti etkileşimi diyagramları sütunların plastik menteşe davranışında göz önünde bulundurulur. Perde duvarlarının sınır bölgeleri iki fiber elemana ayrılır ve bu fiber elemanlar için sınırlı ve sınırsız özellikler göz önünde bulundurulur. Perde duvarların ağ bölümünde, betonarme beton özellikleri dikkate alınmıştır. Lineer olmayan doğrudan entegrasyon analizlerinde mander beton modeli, beton davranış olarak sınırlandırılmıştır. Bölümün ilk kısmında, yapısal sistemin tanıtımı ve doğrusal performans analizinin sonuçları açıklanmaktadır.

Son kısımda, sonuçlar sisteme ve elde edilen elemente dayanarak tamamlandı. TBDY-2018'e göre yapılan performans değerlendirme sonuçları sistem bazında ve eleman bazında iki başlık altında sunulmaktadır. Sistem bazlı sonuçlar yapının genel davranışını belirlemek için uygundur. Yapının genel davranışına hızlı bir bakış için yararlı olan bu sonuçların yapının performans seviyesini belirlemek için kullanımı zordur. Bu sonuçlar, taban kayma kuvvetlerini ve yer değiştirmeleri gösteren maksimum yer değiştirmeleri ve histerik eğrileri ve her deprem tarafından oluşturulan plastik menteşeleri içerir. Elementler bazındaki sonuçlar, taşıyıcı sistemi oluşturan her bir elementin analizinin bir sonucu olarak hasar bölgelerinin belirlenmesi sayesinde, sistem bazındaki sonuçlardan daha kapsamlıdır. Son bölümde, sonuçlar ve tavsiyeler açıklanmıştır. Izin verilen sınırlar ve kolon ve kiriş elemanları daha küçük yüklere maruz kalır ve bölümler, dolgu duvarları tarafından alınan taban kesme kuvvetinin çoğunluğu tarafından daha az zarar görür. Böylece, mevcut yapının performans seviyesinin arttığı ve dolgu duvar eklenmesi sonucu bölümlerin hasar seviyelerinin düştüğü görülmüştür. Elde edilen sonuçlara dayanarak, yapılara yeterli dolgu duvar elemanlarının eklenmesinin pratik ve etkili bir çözüm tekniği olacağı sonucuna varıldı.

1. INTRODUCTION

Earthquakes are considered as one of the biggest threats when it comes to natural disasters. The fact that their time and place cannot be predicted makes its effect even more dangerous. In the field of structural engineering it has enormous importance. The responsibility of the structural engineers has increased as a the result of lack of Technologies and inventions in predicting the earthquakes' intensity, time and place. Due to the facts mentioned the only solution is to design, project and build structures that will decrease the effect of the earthquake as much as possible.

The effect of the earthquakes on the structure changes depending on the type of structure, type of earthquake and the place where it will be constructed. Turkey is one of the countries where the earthquake has caused many losses of lives and material disasters. The most severe examples are the earthquakes that occurred in 1999, the first on August 17^{th} in Kocaeli and the second one in November 12^{th} in Düzce. The structures were not projected and constructed according to the seismic code and that is the most crucial reason for the hazard caused to the structures. On the other hand, the code itself was opened to questions.

In order to decrease the effects of the earthquakes to the structures, the relevant authorities decided to change the seismic code and as a result of the researches, they came up with DBYBHY 2007 (The seismic code of the structures built in earthquake zones). It made a positive effect because it started the topics of performance analysis and strengthening

However, due to several discusses of the experts in the field of structural, earthquake and soil mechanics engineering in 2018, they proposed the new code, which was approved by the relevant authorities. It is obviously a more extended version including topics that were not mentioned before. The new code (TBDY2018) has several updates in almost every chapter including the performance analysis. The part which regards the most this thesis is the chapter of time history analysis. The most important update of this chapter is the number of earthquake data that have to be used to obtain the performance.

The performance acquired from a structure depends on the level of the earthquake, earthquake design level, intended purpose, the height of the structure and some other factors. As a result of the mentioned above, there are four different categories of performance.

1.1 Purpose of the Thesis

The primary purpose of this thesis is to define the performance of an existing building using the nonlinear analysis according to two different design codes, EC8 and TBDY2018, and compare them. The things that make this thesis differ from others is the analysis made by two different codes and comparation of the codes.

The content of the thesis is a 24 Story, 84 meters tall reinforced-concrete structure build in Mersin according to the old earthquake design code (DBYBHY2007). The structure is made of a shear wall and frame combined system. The structure will be built in two other examples in two different countries, Turkey and Montenegro. The existing code in Turkey is TBDY2018 and in Montenegro EC8. We managed to find locations with the same PGA (0.34g), so we can expect similar behavior from the buildings. As required from the TBDY2018 11 different earthquake pairs (x and y direction) are used. The average values of structural reactions that came up from seismic loads were used to evaluate the performance of structural elements. After analyzing every structural element general result of the structure is given.

2. PERFORMANCE ANALYSIS

2.1 Introduction

In structural engineering, during the project of a building, the most commonly used method is the method based on force, the classical method of the Capacity design. Almost every design code including the TBDY2018 as the fundamental method for design uses the Capacity Design. The Capacity design is a method that regards to the inner forces' reaction acting in the cross-sections.

Recently the design codes beside the Capacity Design Method started to prioritize the Deformation-Based method.

The performance method is being used for the observation of the safety of existing buildings and to develop the capacity of the structure in new structural projects. The main goal of the performance-based design is to obtain the non-elastic behavior in order to find the bearing capacity of the structures before it collapses.

To obtain the performance of existing structures, two methods are used: linear elastic analysis method and nonlinear analysis method. The linear analysis method is not able to solve the non-elastic behavior of the structure. For this reason, TBDY2018 offers us the nonlinear analysis in order to observe the non-elastic behavior of the structural elements.

2.2 Definition of Performance in Non-linear Analysis

Chapter 15 of the TBDY 2018 has precisely defined the rules used for defining the behavior and performance of the structure, and it also defines the principles of structural reinforcement calculations. According to this chapter, for the evaluation of the performance of existing buildings, we need to collect information about the structure and to obtain damage limits and damage zones of structural elements in order to find the proper target performance level and the calculation method.

2.3 Reinforced Concrete Material Model

Since concrete and reinforcement steel have different properties, it is tough to obtain a single stress-strain plot for reinforced concrete. Therefore, TBDY 2018 and EC 8 have defined separate models for concrete and reinforcement steel.

2.3.1 Confined and unconfined concrete models

TBDY-2018 in chapter EK 5.A.1 has given necessary information about confined and unconfined concrete models that are used in the non-linear analysis. Figure 2.1 shows the relations between stress and strain for confined and unconfined concrete.

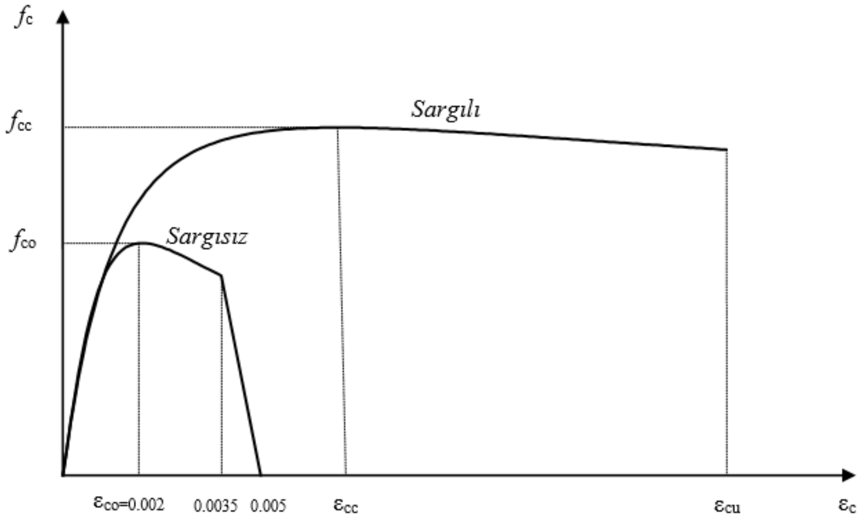


Figure 2. 1 Confined and unconfined concrete stress strain relations (TBDY 2018)

As it can be observed from the graphic above ε_{co}, ε_{cc}, ε_{cu}, f_{co}, f_{cc}. values are represented as follows

ε_{co} strain at maximum confined concrete stress f'cc;

ε_{cc} strain at maximum unconfined concrete stress fco;

ε_{cu} ultimate concrete compressive strain, defined as strain at first hoop fracture;

f_{co} compressive strength of unconfined concrete;

f_{cc} compressive strength (peak stress) of confined concrete;

From the graphic, we can declare that after yielding point, a section of confined concrete has the ability to make more deformation than the unconfined sections which means that confined concrete is more ductile.

The behavior of confined concrete depends on several factors such as longitudinal and confinement bars distribution through the section, the spacing between bars and the volume ratio of bars, compressive strength of concrete, axial loads level and similar parameters.

2.3.2 Reinforcement steel model

Steel reinforcement model used in the nonlinear analysis according to TBDY 2018 is presented in Figure 2.2

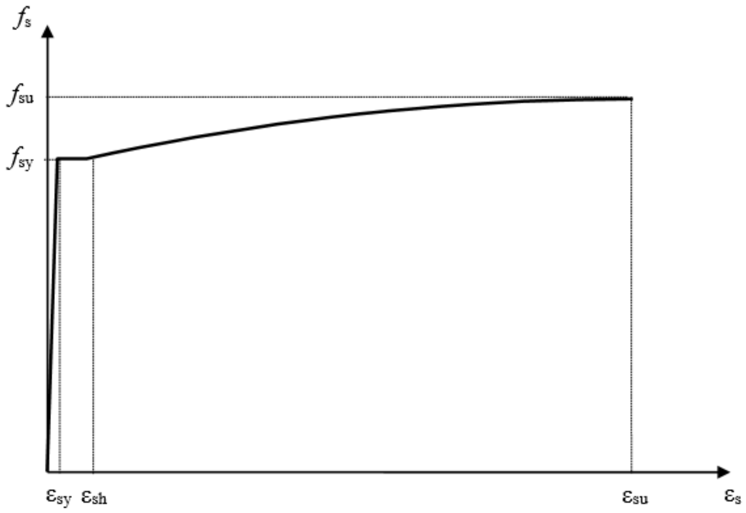


Figure 2. 2 Reinforcement steel stress-strain relations (TBDY 2018)

As shown in the figure below material has three different types of behavior: elastic, plastic area and train hardening region of the graphic.

Therefore,

ε_{sy} : strain value at the yield point of the steel

ε_{ch} : strain value at steel hardening start point

ε_{cu} : strain value of steel rupture

f_{sy} : yield strength

f_{sy} : rupture strain value.

If the relationship between stress and strain of rebar steel is analyzed, it is observed different behavior in elastic and plastic zone. Until the stress reaches the yield value f_y strain values change is very low, while in the elastic-plastic zone value of strain changes on a larger scale and that zone is called hardening zone. This zone continues until the ultimate stress occurs. The values for different reinforcement steel types are shown in Table 2.1 below.

Table 2. 1 Information about reinforcement steel (TBDY 2018)

Kalite	f_{sy} (Mpa)	ε_{sy}	ε_{sh}	ε_{su}	f_{su} / f_{sy}
S220	220	0.0011	0.011	0.12	1.20
S420	420	0.0021	0.008	0.08	1.15 – 1.35
B420C	420	0.0021	0.008	0.08	1.15 – 1.35
B500C	500	0.0025	0.008	0.08	1.15 – 1.35

2.4 Bending Effect of Reinforced Concrete Sections

Moment-curvature interaction of a section under simple bending is shown in figure 2.3. As seen in figure 2.3, it can be mentioned that the behavior of the section is divided into three several zones: OA, AB, BC. The first zone OA defines the elastic behavior of the reinforcement steel while concrete is exposed to compression and tension under low values of moments. Under low moments almost all the section of concrete is active, so the effect of reinforcement is limited. Second zone AB represents the increase of the bending moment resulting in cracking of concrete in tension area which is spread towards the neutral axis. In this level, we assume that in the tensile part every part of concrete cracks and all the tension is met by the reinforcement. So, after this, we consider that the change of behavior in moment-curvature results to the end of linear behavior. With moments increasement, the nonlinear behavior of concrete becomes significant and reinforcement reaches the yield point. The moment value on the yield point of reinforcement is considered as M_y. In the last part BC with moment increasement, we observe plastic deformation of rfeinforcement while the non-linear stress-strain change in the section becomes significant. The energy consume point generally happens when the concrete reaches the maximum strain capacity, and this occurs because the strain capacity of steel is larger than the strain capacity of concrete. This figure 2.3 also gives information about the energy needed to reach the C point. The energy amount depends on the area below the graphic.

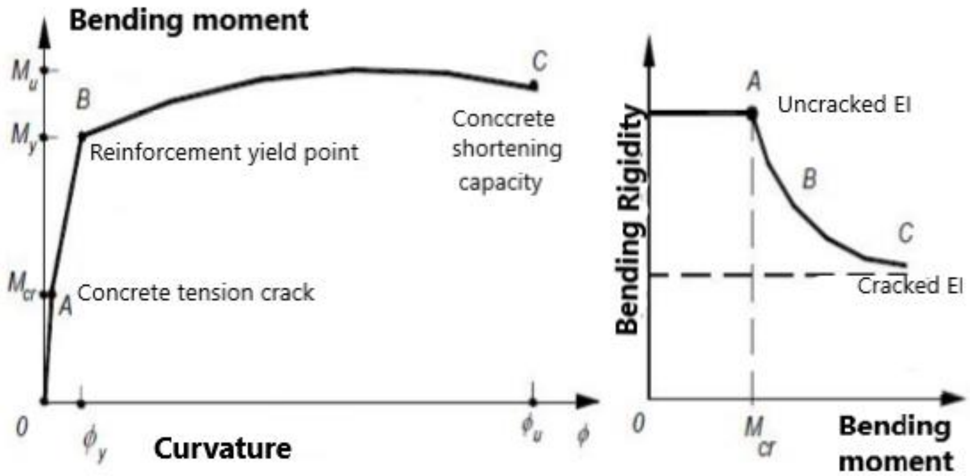


Figure 2. 3 Bending moment-curvature relations of a RC section (Celep)

2.5 Plastic Hinge

Bending moments effect on different structural elements such as columns and beams generates rotation to the sections. While the behavior of the elements affected by low values of moments is linear–elastic, sections exposed to larger values of moments, especially in zones near supports, show a non-linear behavior and as a result, plastic rotations outcome. The hypothesis which assumes that these non-linear deformations are collected in small areas and other zones show linear behavior is called Plastic Hinge Hypothesis. The place where the non-elastic deformation occurs is called a plastic hinge. The plastic hinge can receive moment until it reaches the value of bearing capacity of the ductile section and this fact differs the plastic hinge from the standard hinge. Figure 2.4 shows the process of the plastic hinge hypothesis.

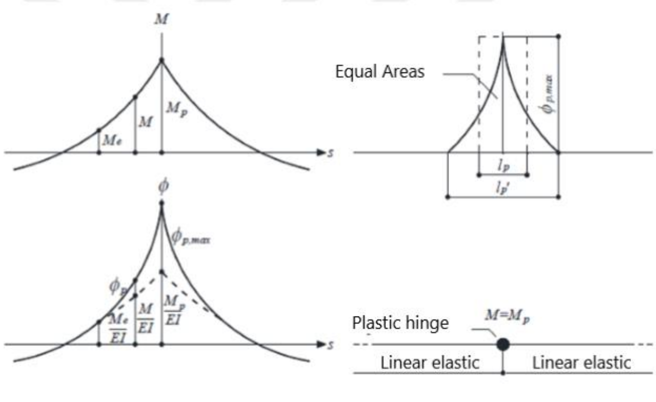


Figure 2. 4 Plastic Hinge Hypothesis

In practice the real moment-curvature relation, showed in Figure 2.5 is idealized in two different linear parts as shown in Figure 2.6

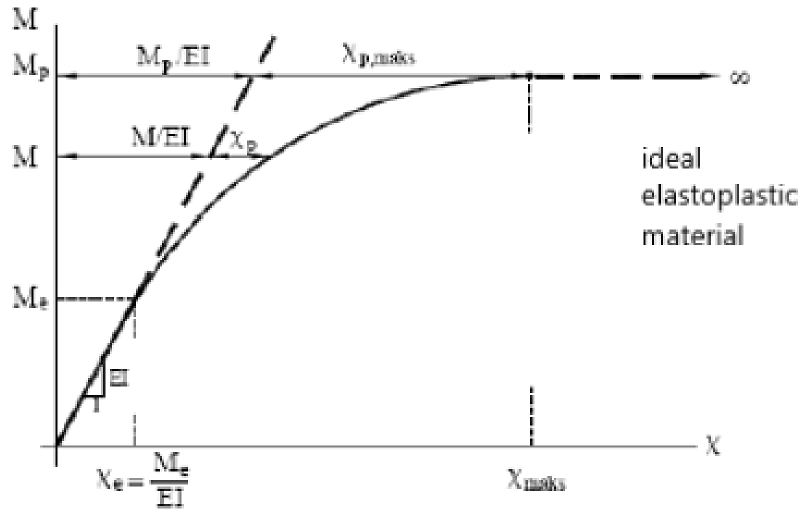
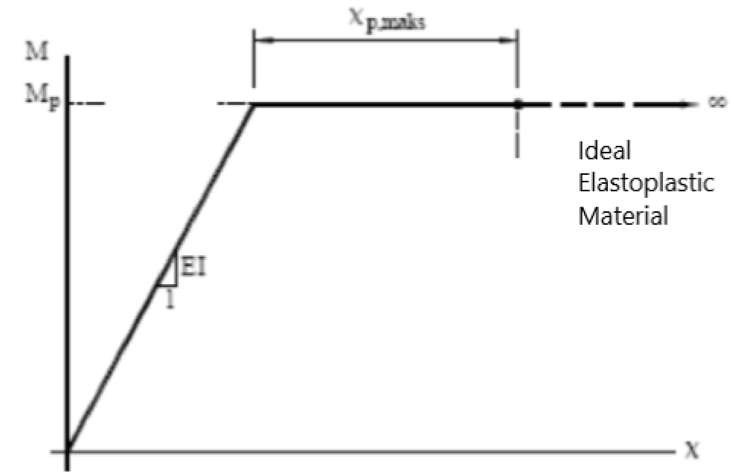


Figure 2. 5 Real bending moment-curvature relation



For $M \leq M_p$ $\quad \mathcal{X} = \frac{\mathbf{M}}{\mathbf{EI}}$

For $M = M_P$ $\quad \mathcal{X} \rightarrow \mathcal{X}_{P,max}$

Figure 2. 6 Idealized bending moment-curvature relation

According to plastic hinge hypothesis, non-linear deformations are collected in the plastic hinge length l_p. Maximum rotation capacity of the plastic hinge is defined in Equation 2.1 below.

$$maks\theta p \int_{lp} \mathcal{X}_p \, \mathbf{ds} \, (\mathcal{X}_p \rightarrow \mathcal{X}_{p\max}) \qquad (2.1)$$

Plastic hinges are generally formed near the support and middle part of frame elements. They appear when the moment value at that part reaches the plastic moment value. After that, moment remains constant and the section rotates freely. When the rotation value reaches maxθ_p section collapses. If this situation happens to other sections of the structure, it can lead to the collapse of structure which will make it useless.

TDBY 2018 has defined two different non-linear behavior models: lumped plasticity model and distributed plasticity model. The most commonly used system is lumped plasticity model.

- The length of plastic deformation zone which is called as plastic hinge length (L_p) is equal to the half of the height (h) of the section in active direction (L_P=0.5*h) and for elements working only by axial force, the length of plastic deformation zone is equal to the dimension of the sections free direction.
- In practical applications, it is considered that plastic hinges should be assigned near joints of beams and columns or othewise said in the edge points of columns and beams, but if beams are under the effect of vertical loads then plastic hinge is placed in the middle part of the beam too.
- For the shear walls, at every story, plastic hinges are placed at the down parts of the sections. If there is a basement floor surrounded by rigid shear walls, the walls that start from the basement and continue to upper floors will have the plastic hinges assing starting from the down part of the first basement floor.

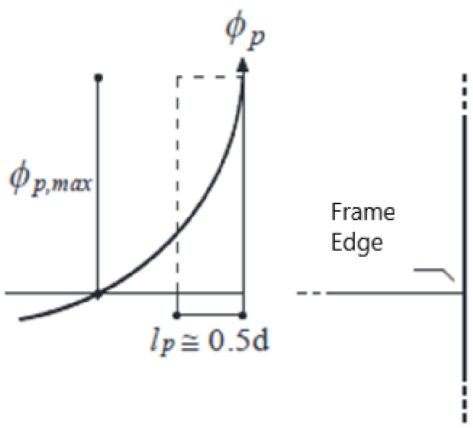


Figure 2. 7 Position of plastic hinges

2.5.1 Distributed plastic hinge model

Spreaded plastic behavior model, refeering to TBDY 2018 5.3.2.20 chapter is defined as the non-linear shape along the entire length or the plastic deformation zone used to take into account continuous changes. Especially in case of complicated RC shear walls sections, the sections is divided in fibers where each of the represents particular material. For every fiber element the value of stress and strain is calculated. Distributed plastic hinge definition of rectangular shear wall is given in figure 2.8 bellow.

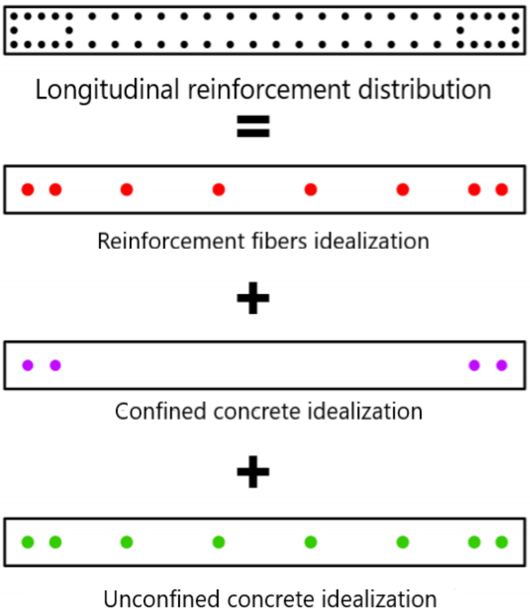


Figure 2. 8 Hinge model of a shear wall fiber section

2.6 Identification of Knowledge Level

2.6.1 Content of the collected information

In order to obtain the capacities of existing structural systems and therefore calculate their resistance during earthquakes, collecting information about the structure is necessary. Therefore, dimensions of the structural elements, their regulation details and materials used in structures must be read from reports and projects, measurements and observations performed in the structure and the strength test done to the material samples taken from the structure. If the project is available, it has to be controlled by a structural engineer to define its correctness. About the collection of information about the structure, TBDY 2018 has provided different knowledge levels. These are: limited knowledge level and comprehensive knowledge level.

2.6.1.1 Limited knowledge level

Limited knowledge level is considered acceptable only for structures belonging to BKS = 3 usage level and I = 3 importance level. The properties of the structure are obtained by measurements done in the structure. The measurement performed in-situ to find the properties such as object geometry, element details and material properties are done as explained below.

- Object geometry: If the architectural project is available the plan of the structural system must be obtained. In order to make a mathematical model we need to obtain all situations of every RC element of every story, materials used, the distance between elements, etc. Short columns or similar disorders that are found in the structure should be obtained and processed in the plan. In order to define the type and characteristics of the foundation sufficient number of excavations must be done. The relation with the neighbor objects should also be analyzed, if they are connected, separated or if there is a dilatation joint between them.

- Element details: In order to estimate the reinforced bar number, concrete clear cover is slipped from 5% of the total number of columns and shear walls with the condition of minimum one column and shear wall per story and one beam per story. The slip process of beams and columns starting from the center is done in three places and after the control the slipped places are repaired with proper material. The reinforce bars number of column and shear wall elements that were not slipped is estimated with a reinforcement detector device applied in 20% of all columns and walls. Detected rebar proportion is divided with the minimum possible proportion of rebars, if the result is less than one the rebar realization coefficient is estimated. Using this coefficient, we can predict the approximate value of rebars at other column and shear wall sections.

- Material properties: Three samples, with the same diameter and length which is not smaller than 100 mm, are taken from columns and shear walls at every story in order to test their strength. If the total number of taken samples is three, the smallest value is considered as concrete strength. If there are more than three samples, the difference between the average of every strength value and the standard deviation is compared to 0.85*average. The largest value is considered as the concrete strength of that building. As for the rebar steel when the concrete is slipped to check the number of rebars the material characteristics of steel will be checked. The yield value will be considered as a strength value of the rebar steel. Corrosions of rebars will also be considered in calculations.

2.6.1.2 Full knowledge level of reinforced concrete structures

The estimation of the object geometry, element details and material properties for the full knowledge level is defined in TBDY 2018. This knowledge level is more advanced that the limited knowledge level.

- Object geometry: If the architectural project is available, it must be compared with the in-situ measuring of the object. If the measurement results are different from the project the information from the project is neglected. If the structural project is not available, the structural system plan is obtained by measurements. After that, in order to create the mathematical model, the steps followed in the previous method will be repeated.

- Element details: If the static design project is available the method of controlling the structure is the same as in limited information method. The rebar number of column and shear wall elements that were not slipped is estimated with a reinforcement detector device applied in 20% of all columns and walls. Also, the same process is done for 10% of beams including the number of longitudinal and stirrup rebars. If the project is not available then, to estimate the number of the reinforced bar, concrete clear cover is slipped from 10% of the total number of columns and shear walls with the condition of minimum two columns and shear walls per story and one beam per story. After the control, the slipped places are repaired with proper high-resistance material. The rebar number of column and shear wall elements that were not slipped is estimated with a reinforcement detector device applied in 30% of all columns and walls and 15% of beams.

- Material properties**:** At least three samples from base floor and two samples from other floors, with the condition of minimum nine samples in total and for every 400 m^2 at least one sample are taken from columns and shear walls at every story in order to test their strength. The difference between the average of every strength value and the standard deviation is compared to 0.85*average. The largest value is considered to be the concrete strength of that building. As for the rebar steel, when the concrete is slipped to check the number of rebars the material characteristics of steel will also be checked. For every type of steel (S220, S420, etc.), one sample will be controlled by experiment. After finding the yield stress, ultimate stress and strain values, the availability of steel will be discussed. If it has proper characteristics, the value of the characteristic yield will be the yield strength of existing steel during the performance calculations. If the results are not proper, at least three more samples will be taken and analyzed. The smallest value of yield strength will be used as the yield strength of existing steel. Corrosions of rebars will also be considered in calculations.

2.6.1.3 Knowledge level characteristics

According to the information collected by in-situ inspection, the knowledge level is obtained and it corresponds to an knowledge level coefficient which is shown in table 2.2.

Table 2. 2 Knowledge level coefficients (TBDY 2018)

Knowledge level	Knowledge level Coefficient
Limited	0.75
Extensive	1.00

2.7 Damage Limits and Damage Zones of the Structures

2.7.1 Section damage situations

Ductile sections can face three different damage situations or damage limits, according to TBDY 2018. These situations are Limited damage (SH), Controlled damage (KH) and Near Collapse situation (GÖ) and their limit values.

1. Limited Damage (SH): Limited amount of non-linear behavior in sections
2. Controlled Damage (KH): Non-linear behavior in such amount that security can be provided

3. Near Collapse (GÖ): Non-linear behavior at an advanced level. Elements damaged in the fragile way are not included in this classification.

2.7.2 Section damage zones

If critical sections damage did not reach SH (limited damage) level, then structure remains in Limited damage zone. If the damage is between SH (limited damage) and KH (controlled damage), then it is Significant Damage Zone. When damage is between KH (controlled damage) and GÖ (Near collapse), then it is Advanced Damage Zone. When damage passes GÖ (Near collapse), then it is Collapse Zone. It is defined in figure 2.10 as required in TBDY 2018.

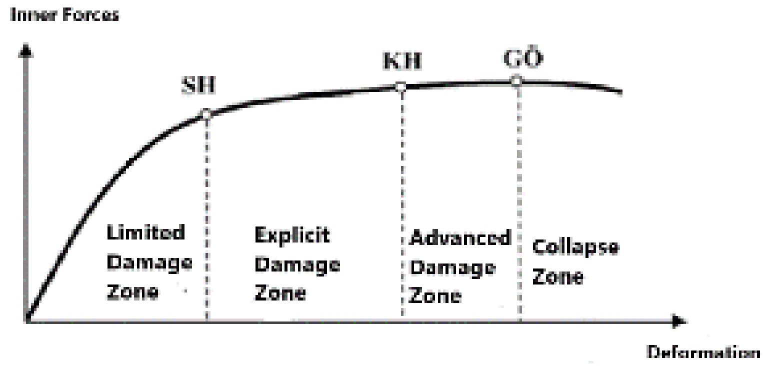


Figure 2. 9 Section damage zones (TBDY 2018)

2.7.3 Section and elements damage definition

The damage zone where the structure belongs is obtained by finding the inner forces and deformation using several linear and nonlinear methods as required in the design codes.

2.8. Structural Performance Levels

Performance levels of the structure under the effect of the earthquake are explained in TBDY 2018 as follows:

2.8.1 Serviceability performance level

This performance level is used for the situation when there is no damage in the building, or the damage can be neglected.

2.8.2 Limited damage performance level

This performance level deals with the situation when the damage is limited, or in other words, it is within the limits of the linear behavior limit.

2.8.3 Controlled damage performance level

This performance level is used for the situation when the safety of people is provided and the structure is not heavily damaged so that most of the structural elements can be repaired.

2.8.4 Collapse prevention performance level

This performance level is used for the situation when the level of damage of the structural elements is advanced and there are substantial damages. The full or partial collapse is prevented.

2.9 Structural Performance Target

Depending on the level of seismic ground motion and earthquake design level TBDY-2018 has given choices for performance target in table 2.3. As the owner of the structure wishes, advanced level or normal performance level can be chosen. For existing reinforced RC high-rise structures target performance is defined in table 2.3 below.

Table 2. 3 Existing High-Rise Structures (BYS=1) (TBDY 2018)

Seismic Motion Level	DTS=1, 2, 3, 3a, 4, 4a		DTS=1a, 2a	
	Normal Performce target	Design approach	Advanced Performce target	Design approach
DD-4	KK	DGT	-	-
DD-3	-	-	SH	ŞGDT
DD-2	KH	DGT	KH	DGT
DD-1	GÖ	ŞDGT	KH	ŞGDT

2.10 Defining the Performance of Existing Structures

2.10.1 Earthquake performance evaluation of existing structures

Considering the predicted damages caused to the structure as an effect of applied seismic load and by using a linear or nonlinear calculation method as provided in

TBDY 2018, damage zones of the structure can be estimated, therefore the performance of the structure is determined. TBDY 2018 defines the performance level based on four different damage situations.

2.10.1.1 Limited damage performance level of existing structures

Structural systems elements show a deficient level of damage and they have preserved their strength. In this performance level at any story of the reinforced concrete structure and for any direction that earthquake load is applied, a maximum of 20% of the total number of beams can belong to the significant damage zone, with the condition that every other element remains in the limited damage zone. If there is any element showing damage at a fragile level, it must be strengthened.

2.10.1.2 Controlled damage performance level of existing structures

One part of the structural system elements have been damaged, but the horizontal stiffness and strength are not lost. So, in the structure, there is no local or total collapse. Criteria used for this damage level, according to TBDY 2018, are mentioned below.

a) At any story of the structure as a result of earthquake calculation for any direction maximum 35% of the total number of beams, except the secondary beams, can belong to Advanced Damage Zone, as far as vertical elements (columns, shear walls) are concerned the rules are explained in the following paragraph.

b) The amount of shear force carried by vertical elements belonging to the advanced damage zone must be less than 20% of the total shear force carried by vertical elements of that floor. As for the last floor, amount of shear force carried by vertical elements belonging to advanced damage zone must be less than 40% of the total shear force carried by vertical elements of that floor.

c) Every other element remains in the Significant Damage Zone or significant damage zone. The amount of shear force carried by vertical elements that passed the significant damage limit, at their top and the bottom section at the same time, must be less than 30% of the total shear force carried by vertical elements of that floor.

2.10.1.3 Collapse prevention performance level of existing structures

In this performance level, an essential part of the elements of the structure have been damaged and the horizontal stiffness and strength are lost at high rates, too. Every element damaged in a fragile way should be considered as it belongs to Collapse damage zone. Criteria used for this damage level, according to TBDY 2018, is mentioned below.

a) At any story of the structure and for any direction that earthquake load is applied, maximum 20% of total number of beams except the secondary beams can belong to Collapse damage zone

b) Other elements belong to Limited damage zone, significant damage zone or advanced damage zone. If any vertical element has passed the significant damage limit in its upper and lower section the proportion of the shear force carried by that element in that floor must be maximum 30%

c) Usage of the object in this situation is dangerous and life threatening.

2.10.1.4 Collapse situation

It represents the performance level when the structure exposed to seismic load exceeds the collapse limit. If the structure does not supply the prevention of collapse level it means that it will collapse. Object usage in this situation is dangerous and life threatening.

2.11 General Principals and Rules About Earthquake Calculations

In order to determine the earthquake performance of a structure, seismic code has defined several linear or nonlinear methods. Since these methods use different theoretical approaches the results will be different. For both methods there are some common principals and rules.

- In defining the earthquake effect, for specified seismic ground motion levels, horizontal elastic design spectrum will be used. Structure Importance Coefficient will not be applied (I=1,0).
- Vertical loads of the structure and horizontal effects of earthquake will be used combined in process of estimating the structures' earthquake performance.

- Mass in earthquake calculations will be defined according to table 4.3 given in TBDY 2018.
- Both directions of seismic loads must be applied to the structure in two axes separately.
- The mathematical model of structure will be prepared in valid manner in order to calculate inner forces, deformation and displacements of the structural elements under seismic and vertical loads.
- Two lateral and rotation degree of freedom will be assigned to the structure by the diaphragm effect to the slab elements. Every stories degree of freedom center will be concentrated in the mass center and additional eccentricity will not be applied.
- Structural uncertainties of existing structures will be reflected to the structure by the coefficient of knowledge level.
- Columns defined as short columns will be defined to the structural system model with their real free length
- Conditions for defining the interaction diagram of reinforced concrete sections with one or two axial bending and axial forces are mentioned below:

a) In seismic calculations concrete and rebar steel existing strength will be used according to knowledge level

b) Maximum compressing strain of concrete can be taken as 0,0035 while maximum strain of steel is considered as 0,01

c) Interaction diagram can be modelled in many directions or many planes if it is properly linearized

- When defining the dimensions of reinforced concrete elements joint zones can be considered as rigid edge zones.
- Values of the crack stiffness of the bending elements will be used as stiffness values. Effective section stiffness is mentioned in table 2.4

Table 2. 4 Structural elements effective stiffness coefficients (TBDY 2018)

RC Elements	Effective Section Stiffness Coefficient	
Shear Wall-Slab System (In-plane)	Axial	Shear
Shear Wall	0.50	0.50
Basement shear Wall	0.80	0.50
Slab	0.25	0.25
Shear Wall-Slab (Out-of-plane)	Bending	Shear
Shear Wall	0.25	1.00
Basement shear Wall	0.50	1.00
Slab	0.25	1.00
Bar Elements	Bending	Shear
Transverse beam	0.15	1.00
Frame beam	0.35	1.00
Frame column	0.70	1.00
Shear Wall	0.50	0.50

- During the calculation of positive and negative moments of T-beams the flange part of concrete and rebar can be included in calculations
- In situations when the overlapping length is not enough the value of the moment capacity will be reduced in proportion with deficiency of overlapping length
- Soil deformations which affect the structure will be projected in soil properties analysis model.

2.12 Evaluation of Structure Performance Using Linear or Non-linear Calculation Analysis Methods

2.12.1 Linear analysis methods

Within the strength-based structural design (DGT), TBDY 2018 assumed that the linear calculation principle considers that the structure behaves in linear limits and displacement values are very low. The main aim of this method is to evaluate the capacity by comparing the inner forces that came as a result of load affecting the structural system with bearing capacity of structural elements calculated with strength-based DGT method. The seismic load gets reduced with a reducing coefficient. As reaction of reduced seismic load linear behavior is expected. This method is divided in three types: Equivalent Seismic load method, Mode Superposition Method and Mode Collection in Time History Method.

2.12.1.1 Equivalent seismic load method

This method is based on principle of loading the calculated earthquake to the structure by considering the first vibration period and distributing the load depending on mass or stiffness. It is appropriate method for regular structural systems and structures with low rate of non-regularity. According to this method two components of earthquakes in X and Y axis are loaded in separated calculations. Structure which are available for this method are shown in table 2.5

Table 2. 5 Structures available for Equivalent seismic load method (TBDY 2018)

Structure Type	Allowed Structural height level (BYS)	
	DTS= 1, 1a, 2, 2a	DTS= 3, 3a, 4, 4a
Structures with torsion coefficient $\mu_{bi} \leq 2.0$ and without B2 non-regullarity type	BYS ≥ 4	BYS ≥ 5
Other Structures	BYS ≥ 5	BYS ≥ 6

Total load effecting the structure (Base shear force)

$$V_{tE} = m_t \, S_{aR} \, (T_p) \geq 0.04 \, m_t \; I \, S_{DS} \, g \qquad (2.2)$$

is calculated as mentioned in Equation 2.2. S_{aR} is the reduced value of design spectral acceleration, S_{DS} is design spectral acceleration coefficient, m_t is the mass of the structure.

2.12.1.2 Modal superposition method

This elastic-dynamic analysis method calculation is done considering every vibration mode of structural behavior and finding statistical coefficients responding to them. Seismic loads are found by unifying these coefficients results.

2.12.1.3 Time history analysis method

Using original or adapted earthquake data time history analysis for every time increasement interval provides separated calculations which results to evaluation of seismic loads. By using this method inner forces and displacements which occur during every time interval of the earthquake can be obtained.

2.12.2 Nonlinear Analysis Methods

For the structures designed using Deformation Based Design calculation method (ŞDGT) TBDY 2018 in non-linear analysis calculations method includes the fact that materials can overpass the linear elastic behavior limit. The target of deformation-based methods is to obtain the deformations occurring in the structures main points which are considered as plastic hinges under seismic loads. Using several advanced software technologies, we can observe more realistic nonlinear behavior of structural elements under seismic loads. Nonlinear analysis methods are divided in three types: Single-mode pushover analysis, Multi-modal pushover analysis and time history non-linear analysis.

2.12.2.1 Single mode pushover analysis method

Single mode pushover analysis is the incremental equivalent of the linear modal superposition method. Seismic load in the precised earthquake direction and proportional to the vibration mode is applied step by step until the earthquake displacement value reaches the required value and as a result of this displacement, plastic deformations and inner force increments cumulative values are calculated. At the last step cumulative values corresponding to earthquake request are considered. Conditions required to use this method are as follows: without considering additional eccentricity, torsion irregularity coefficient is $\mu_{bi} \leq 1.40$, at predominant period, the rate between shear force of any story and effective mass is at least 0.70 (basement floor surrounded by rigid shear walls are not included).

2.12.2.2 Multi modal pushover analysis method

In order to be able to use the nonlinear Multi modal pushover analysis method calculation rsults must confirm that all inner forces and displacement values calculated according to design spectrum initial (elastic) stiffness using linear methods are equivalent to inner forces and displacement values calculated using Mode superposition method. Also, according to Multi modal pushover analysis method, the value of inner forces when constant Single mode vectors defined for different modes are applied to the structure separated will not be statistically combined and inner forces will be calculated compatible with modal combined elements limit displacements and yield rotations, according to 4B.2.4.

2.12.2.3 Nonlinear time history analysis method

Using recorded or adapted earthquake data and considering the nonlinear behavior of the structure, this method provides the step by step integration of the motion equations of the structural system under seismic loads. Due to nonlinear behavior of the structure, stiffness matrix changes with time and for every time increment step maximum value of inner forces and elastic or plastic deformations are calculated. By defining the plastic deformations, the sections damages can be found. The fact that makes this method more realistic is the step by step integration of analysis.

2.13 Defining Strain Deformations and Plastic Rotation Values

Using Equivalent earthquake load or mode superposition method, as a result of calculation, elements strain and plastic rotation request is found using the value of total displacement axis at any edge of the element θ_k. The total curvature request of edge sections of the elements F_k is defined in equation 2.3 as explained below:

$$\emptyset_t = \frac{\theta_k - \theta_y}{L_p} + \emptyset_y \tag{2.3}$$

θ_y : value of yield rotation of elements edge sections

$\emptyset_y$: value of yield curvature of elements edge sections

L_p : Length of the plastic hinge equal to the half of the length sections length in effective direction

Value of effective yield curvature $\emptyset_y$ and curvature moment M_y of RC elements will be calculated with moment-curvature method.

2.14 Allowed Deformation Limits for Structural Elements

Performance of the structural system is obtained by comparing the strain requests of concrete and rebar steel with the deformation capacities mentioned below. Allowed deformation limits (capacities) of different damage limits occurred as result of plastic deformation of ductile RC structures are defined by TBDY 2018. If the steel bars used in longitudinal bars are not ribbed, then the deformation and plastic deformation request values will be multiplied by 1,5.

a) Total strain and plastic rotation allowed for concrete and rebar steel for collapse prevention performance level:

$$\varepsilon_c^{GÖ} = 0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018 \quad ; \quad \varepsilon_S^{GÖ} = 0.4\varepsilon_{SU} \tag{2.4}$$

$$\theta_p^{GÖ} = \frac{2}{3}\left[\left[(\varphi_u - \varphi_y)L_p\left(1 - 0.5\frac{L_p}{L_s}\right)\right] + 4.5\varphi_u d_b\right] \tag{2.5}$$

b) Total strain and plastic rotation allowed for concrete and rebar steel for controlled damage performance level:

$$\varepsilon_c^{KH} = 0.75\varepsilon_c^{GÖ} \quad ; \quad \varepsilon_S^{KH} = 0.75\varepsilon_S^{GÖ} \tag{2.6}$$

$$\theta_p^{KH} = 0.75\theta_p^{GÖ} \tag{2.7}$$

c) Total strain and plastic rotation allowed for concrete and rebar steel for limited damage performance level:

$$\varepsilon_c^{SH} = 0.0025 \quad ; \quad \varepsilon_S^{SH} = 0.0075 \tag{2.8}$$

$$\theta_p^{SH} = 0 \tag{2.9}$$

can be calculated using equations mentioned above.

3. GROUND MOTIONS AND EARTHQUAKE DATA USED IN TIME HISTORY ANALYSIS

In order to perform the non-linear time history analysis and, therefore, to observe the performance of the structure, we first have to find earthquake data and then process them as the code requires. Requirements of the TBDY-2018 in the process of finding and proccesing earthquake data will be discussed in this chapter.

3.1 Seismic Ground Motion Levels

TBDY 2018 has defined four different earthquake levels, as mentioned below.

3.1.1 Seismic ground motion level -1 (DD-1)

DD-1 earthquake ground motion level contains earthquakes with a probability of 2% to exceed the magnitude in 50 years, which corresponds to a repeat period of 2475 years and it is considered a very rare earthquake. This seismic ground motion level is also characterized as the level with the heaviest earthquakes.

3.1.2 Seismic ground motion level -2 (DD-2)

DD-2 earthquake ground motion level contains earthquakes with a probability of 10% to exceed the magnitude in 50 years which corresponds to a repeat period of 475 years and as a result it is considered a very rare earthquake. This seismic ground motion level is also characterized as standard design seismic ground motion.

3.1.3 Seismic ground motion level -3 (DD-3)

DD-3 earthquake ground motion level contains earthquakes with a probability of 50% to exceed the magnitude in 50 years which corresponds to a repeat period of 72 years

3.1.4 Seismic ground motion level -4 (DD-4)

DD-4 earthquake ground motion level contains earthquakes with a probability of 68% to exceed the magnitude in 50 years which corresponds to a repeat period of 43 years and it is considered a widespread earthquake. This seismic ground motion level is also characterized as servis design seismic ground motion.

3.2 Seismic Ground Motion Spectrum

The design spectrum, according to TBDY 2018, is defined by considering the characteristics of the zone where the structure is located and the distance of the structure form active fault. In order to determine more realistic results, the values used in defining the earthquake spectra, such as map coefficients referring to the spectral short period and spectral one second period depend on fault distance coefficient and soil coefficient as well. Also, in order to consider the effect of the vertical earthquake there is vertical elastic spectrum defined.

3.2.1 Map spectral acceleration coefficient and design acceleration coefficient

In TDBY 2018 the map spectral acceleration coefficient of the short period Ss and map spectral acceleration coefficient for 1s period are defined as the geometrical average of the spectral acceleration coefficients which responds to the effects of an earthquake whose both perpendicular directions are considered. These non-dimensional coefficients are found by dividing the Map spectral acceleration of a specific earthquake considering V_s=760m/s as soil conditions reference and absorption rate 5% by the gravity acceleration value. The non-dimensional map spectral acceleration coefficient S_s and S_1 are transformed to S_{DS} and S_{D1} using the equation below with the help of local soil effect coefficients (F_s and F_1).

$$S_{DS} = S_S F_S \quad (3.1)$$

$$S_{D1} = S_1 F_1 \quad (3.2)$$

3.2.2 Local soil effect coefficient

Local soil effect coefficient F_s and F_1 in TBDY 2018 are defined in the Table 3.1 and Table 3.2 depending on the defined local soil category. For the intermediate values of the map spectral acceleration coefficient linear interpolation can be done. For the soil that belong to ZF local soil category special soil behavior analysis must be done.

Table 3. 1 Local soil effect coefficients for short period (TBDY 2018)

Local Soil Category	Local soil effect coefficients for short period F_s					
	$S_s \leq 0.25$	$S_s=0.50$	$S_s=0.75$	$S_s=1.00$	$S_s=1.25$	$S_s \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	special soil behavior analysis will be done					

Table 3. 2 Local soil effect coefficients 1.0 second period (TBDY 2018)

Local Soil Category	Local soil effect coefficients for 1.0 second period F_1					
	$S_1 \leq 0.10$	$S_1=0.20$	$S_1=0.30$	$S_1=0.40$	$S_1=0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	special soil behavior analysis will be done					

3.2.3 Horizontal elastic design spectrum

Value of the horizontal elastic design acceleration S_{ae} (T) in terms of gravity acceleration (g) which responds to any value of vibration period (T) , for any seismic ground motion level is defined in TBDY 2018 in Figure 3.1.

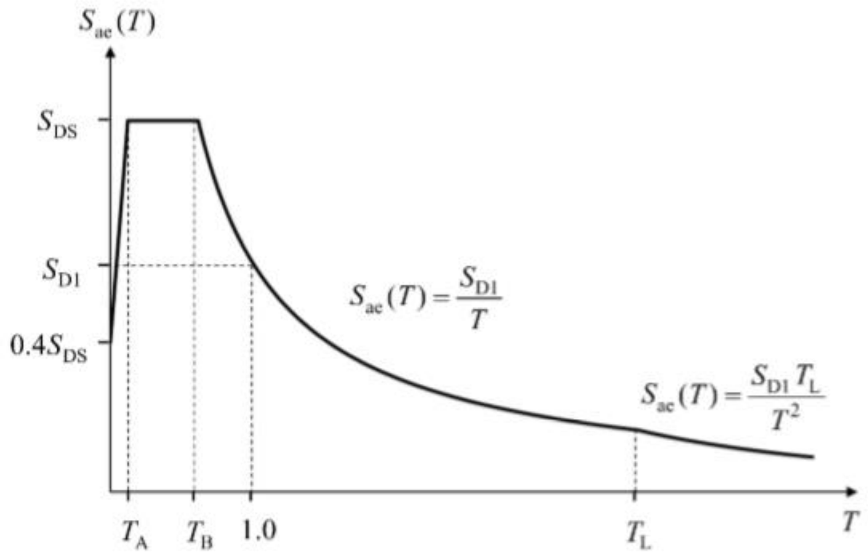


Figure 3. 1 Horizontal elastic design spectrum (TBDY 2018)

Values of horizontal elastic design acceleration in the spectrum are defined with different equations between the periods T_A, T_B and T_L. Horizontal elastic design acceleration S_{ae} value coincides vibration period of the structure (T) according on the interval where it belongs. It can be calculated with equations mentioned below:

$$S_{ae}(T) = \left(0.4 + 0.6\frac{T}{T_A}\right) S_{DS} \qquad 0 \leq T \leq T_A \tag{3.3}$$

$$S_{ae}(T) = S_{DS} \qquad T_A \leq T \leq T_B \tag{3.4}$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad T_B \leq T \leq T_L \tag{3.5}$$

$$S_{ae}(T) = \frac{S_{D1} x T_L}{T^2} \qquad T_L \leq T \tag{3.6}$$

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} \quad ; \quad T_B = \frac{S_{D1}}{S_{DS}} \tag{3.7}$$

S_{ae} (T) represents the elastic design acceleration value, S_{DS} and S_{D1} are the design spectral acceleration coefficients T_A and T_B edge values of period while T_L shows the transition period.

3.2.4 Vertical elastic design spectrum

TBDY2018 has defined a vertical elastic design spectrum in order to include the vertical effect of the seismic action. Vertical elastic design spectrum model is presented in figure 3.2.

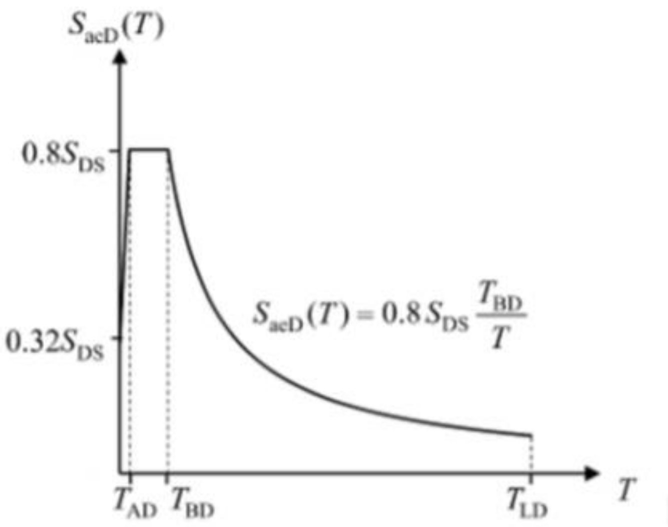


Figure 3. 2 Horizontal elastic design spectrum (TBDY 2018)

TBDY2018 Vertical elastic design acceleration values calculation depending to the period is divided in three zones. The boundary periods used to define Vertical elastic design acceleration are T_{AD}, T_{BD} and T_{LD}. Depending on the zone where the vibrations period belongs vertical elastic design accelerationis calculated utilizing following equations.

$$S_{ae}(T) = \left(0.32 + 0.48\frac{T}{T_{AD}}\right)S_{DS} \qquad 0 \le T \le T_{AD} \tag{3.8}$$

$$S_{ae}(T) = 0.8S_{DS} \qquad T_{AD} \le T \le T_{BD} \tag{3.9}$$

$$S_{ae}(T) = \left(0.8S_{DS}\,\frac{T_{BD}}{T}\right) \qquad T_{BD} \le T \le T_{LD} \tag{3.10}$$

$$T_{AD} = \frac{T_A}{3} \quad ; \quad T_{BD} = \frac{T_B}{3} \quad ; \quad T_{LD} = \frac{T_L}{3} \tag{3.11}$$

3.3 Seismic Ground Motion and Criteria Defined in TBDY-2018

Time History nonlinear analysis is the method which includes the calculation of the inner forces and displacements as the result of time increasement on the equation of motion which comes from the load reaction of the structural system exposed to several loads. Making this analysis for every time increasement provides the update of the stiffness matrix at every time step and it also determines the non-linear behavior of the sections. The fact that the data bases of the strong ground motions throughout the world are getting more enriched and access to them got very easy made this method much easier.

One of the most important problems which we face while performing the time history analysis is finding proper strong motion data and matching them by meeting the requirements of the design codes. Acceleration data which can be used during analysis is divided in three types: acceleration data compatible to design acceleration spectrum, simulated data and real data recorded during the earthquakes. Artificial earthquake data is created by finding the average value of many recorded earthquake data and by making it compatible to design spectrum. This process provides us an earthquake data which includes all the characteristics of design spectrum. Therefore, if the number of circuits at strong ground motion data is increased the result will be an unreal energy release from the earthquake. Huge values of inner forces will come up at analysis results and this will cause an uneconomical solution for the structure. Adapted or differently known as simulated earthquake data are produced by considering the properties of soil and the spread of earthquake waves. At the other side, when these information are not available design code does not allow simulated data to be used. As for the third type of earthquake data, real data, if the information of ground motion nature, amplitude, duration, frequency content, fault type and its properties, source effecting the data and similar information are available, it makes the earthquake analysis more realistic. Also, the increasement of the recorded data day by day, the easier access to them happens as the result of technological development. This fact makes the time history analysis easier to be performed and the complete earthquake data used in this thesis is real earthquake data.

3.3.1 Selection of real earthquake data

According to TBDY 2018 earthquake data that will be used for a specified region must comprehend to the reaction spectrum which is a result of seismic danger analysis and also must provide the geological and seismic conditions. As the result of the mentioned above, when choosing the data that will be used in time history analysis of structural systems, distance from the fault, fault mechanism and local soil properties must be considered. Priority is given to the data of the earthquakes whose properties are compatible with the zone where structure is found if they are available. If the sufficient number of earthquake data for mentioned properties cannot be found then simulated data can be used. When using this type of data, the seismic source of the area, wave spread and local soil properties must be considered. Model used for simulation will be compared with earthquake occurred in that zone throughout the history, thereby it will be controlled. It is observed that effect of different earthquakes on structure lead to different types of reactions, therefore non-linear analysis methods require many different data to be used and loaded to the structure separately. Due to this for the time history analysis the minimum data pair (two direction) to be used is 11. Maximum number of data from the same earthquake is 3. The data which consists two perpendicular directions will be loaded to the structures global axis (X and Y) together and simultaneously. After that, data will be rotated 90 degrees and analysis will be repeated. The values of deformations of elements showing ductile behavior and inner forces of elements showing fragile behavior will be obtained from all analysis (2x11). The result will be the average of largest absolute values taken from every analysis.

3.3.2 Matching and scaling of the earthquake data

Real earthquake data can be adapted by matching and scaling according to a specific design spectrum in time history or frequency history. Matching in time history is done by multiplying the ground motion data with a coefficient, depending on the target spectra, for the period interval required by the code. This process does not change the frequency of the earthquake, but it changes the amplitude. When it comes to frequency history, we get results that are almost fully compatible to design spectrum.

According to TBDY 2018 matched earthquake data must fulfill several conditions.

These conditions are:

- In one or two dimensional calculations earthquake data matching method, referring to structure vibration T_p, in the interval between 0.2 T_p and 1.5 T_p average value of the amplitude of every spectrum that belongs to chosen earthquake data spectrum must be equal or larger than the average of the design spectrum amplitude values in the same time interval.
- In three dimensional calculations, considering two horizontal components Square-Root-of-Sum-of-Squares (SRSS) of the spectrums for every earthquake data will give us product of horizontal spectrum. Referring to structure vibration as T_p, in the interval between 0.2 T_p and 1.5 T_p, the proportion between average value of the amplitude of every component spectrum that

belongs to chosen earthquake data spectrum and design spectrum amplitude values in the same time interval must be equal to 1.3 or larger. For both horizontal components matching must be done using same value coefficients.

3.4 Matching of Earthquake Data According to TBDY 2018

The process of finding the proper earthquake data in time history has a big importance. Earthquake data that will be used must have the characteristics provided in seismic code. Considering the seismic and geological properties of the soil where the structure is found, appropriate earthquake data is chosen. TDBY 2018 considers that the structure behavior and reaction changes according to the earthquake. Due to that fact TBDY 2018 required that in three-dimensional analysis 11 different earthquake data pairs must be used. Also, number of data used from the same earthquake is maximum three. In this thesis the earthquakes used in nonlinear analysis are shown in Table 3.3

Table 3. 3 Earthquake data

	Event	Year	Station	Mag	Mechanism	Rjb	Vs
1	Landers	1992	Amboj	7.28	Strike slip	69.21	382.93
2	Landers	1992	Bigbear Lake	7.28	Strike slip	45.48	430.36
3	Landers	1992	Barstow	7.28	Strike slip	34.86	370.08
4	Duzce	1999	Sakarya	7.14	Strike slip	45.16	411.91
5	Duzce	1999	Lamont 1059	7.14	Strike slip	4.17	551.3
6	Duzce	1999	Lamont 531	7.14	Strike slip	8.03	638.39
7	Kocaeli	1999	Iznik	7.51	Strike slip	30.73	476.62
8	Kocaeli	1999	Eregli	7.51	Strike slip	141.37	585.09
9	Bigbear	1992	Mt Baldy	6.46	Strike Slip	67.74	444.74
10	Chalfant	1986	LongValey Dam	6.19	Strike Slip	18.3	537.16
11	Morgan	1984	Gilroy Array 6	6.19	Strike Slip	9.85	663.31

Figure 3.3, Figure 3.4, Figure 3.5, Figure 3.6, Figure 3.7, Figure 3.8, Figure 3.9, Figure 3.10, Figure 3.11, Figure 3.12, Figure 3.13, below are the graphics of the original and unscaled data of every earthquake from the list above.

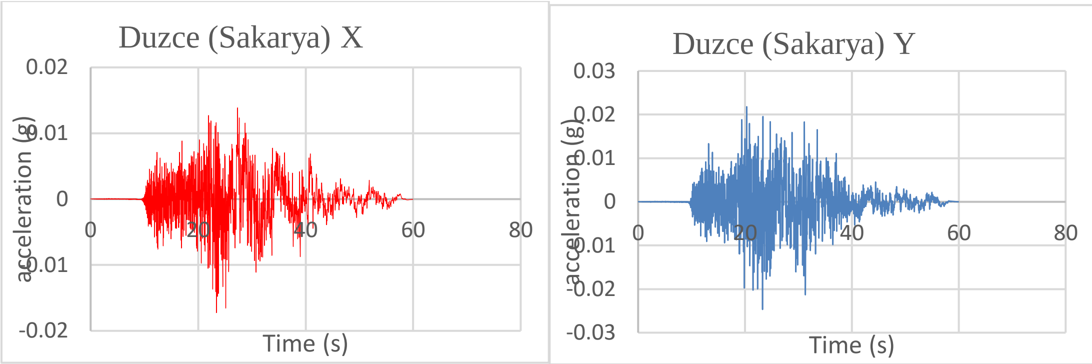


Figure 3. 3 Düzce (Sakarya) Original Earthquake data pair

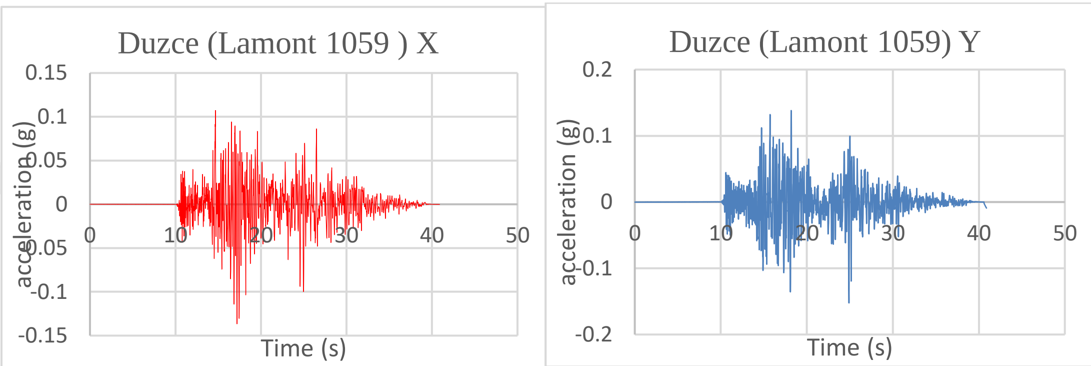


Figure 3. 4 Düzce (Lamont 1059) Original Earthquake data pair

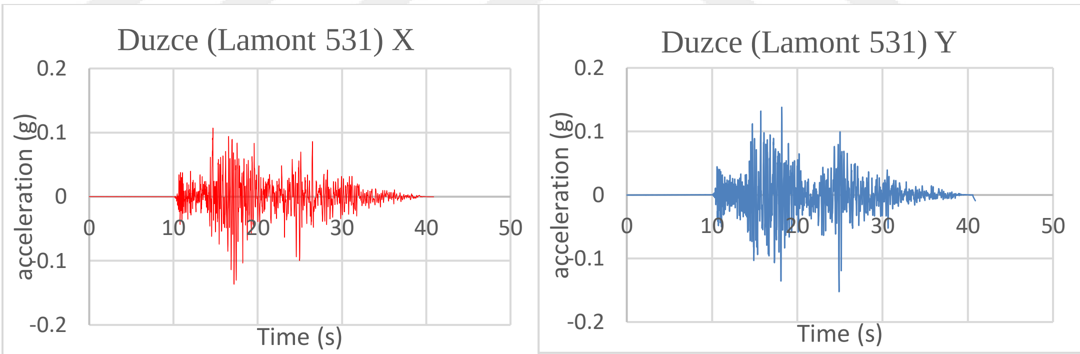


Figure 3. 5 Düzce (Lamont 531) Original Earthquake data pair

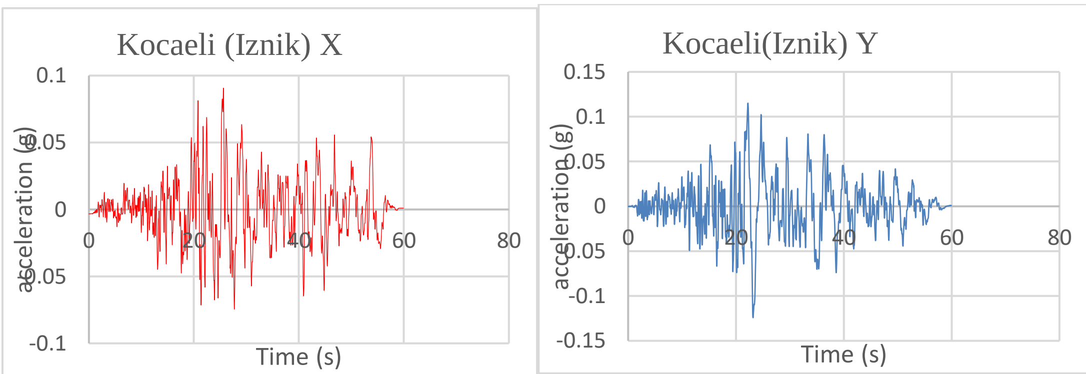


Figure 3. 6 Kocaeli (Iznik) Original Earthquake data pair

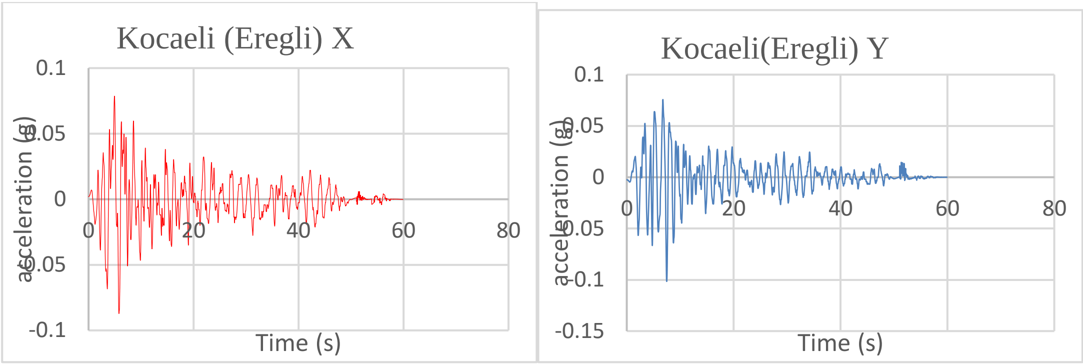


Figure 3. 7 Kocaeli (Eregli) Original Earthquake data pair

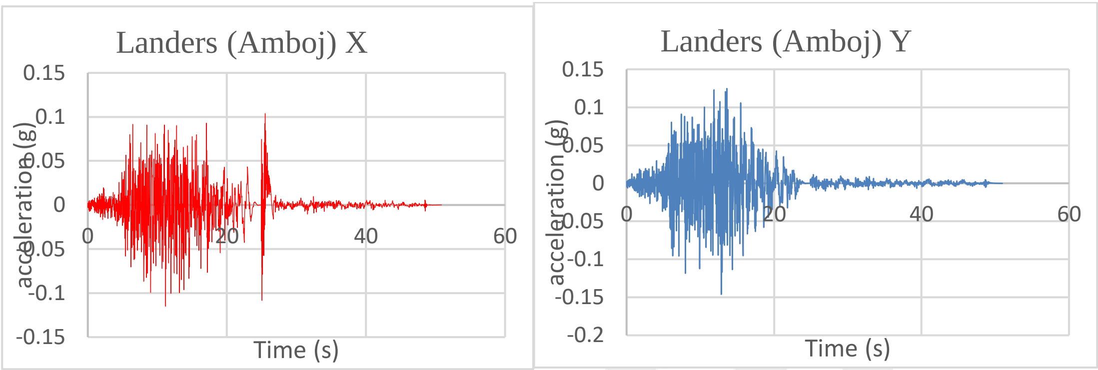


Figure 3. 8 Landers (Amboj) Original Earthquake data pair

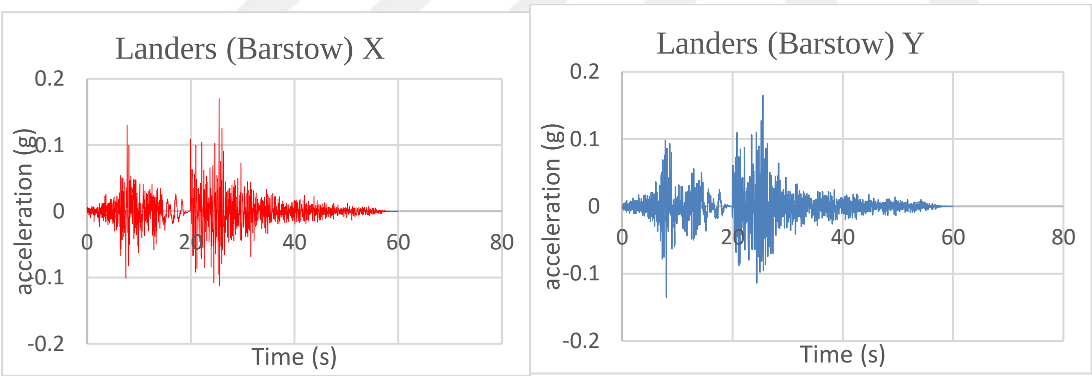


Figure 3. 9 Landers (Barstow) Original Earthquake data pair

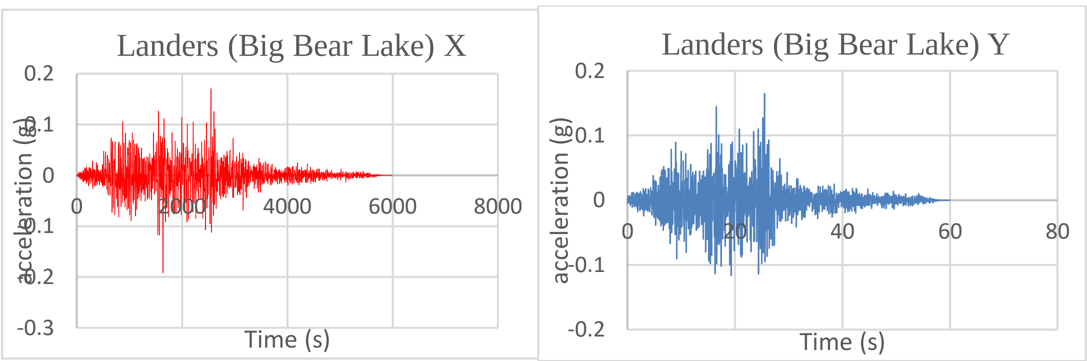


Figure 3. 10 Landers (Big Bear Lake) Original Earthquake data pair

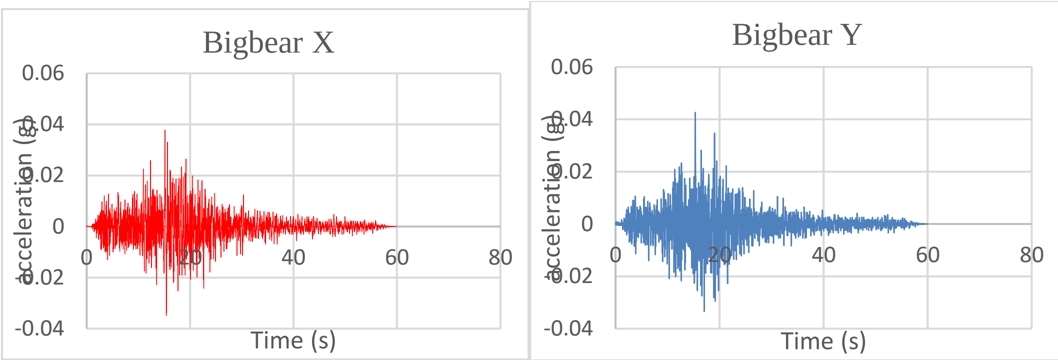


Figure 3. 11 Big Bear Lake Original Earthquake data pair

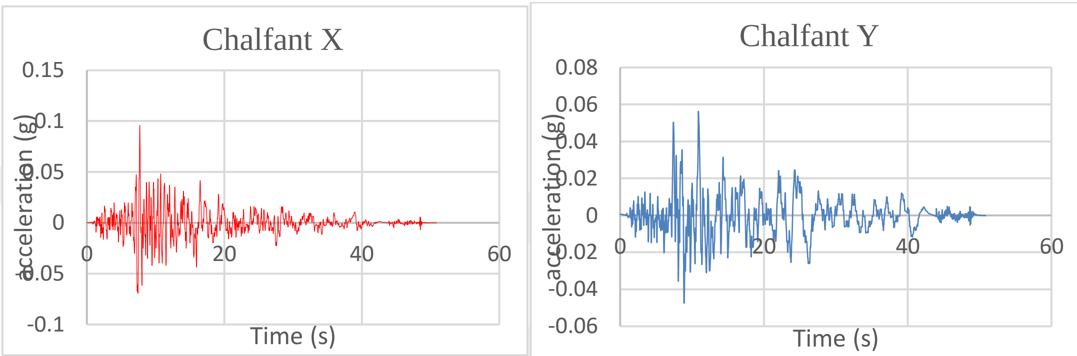


Figure 3. 12 Chalfant Original Earthquake data pair

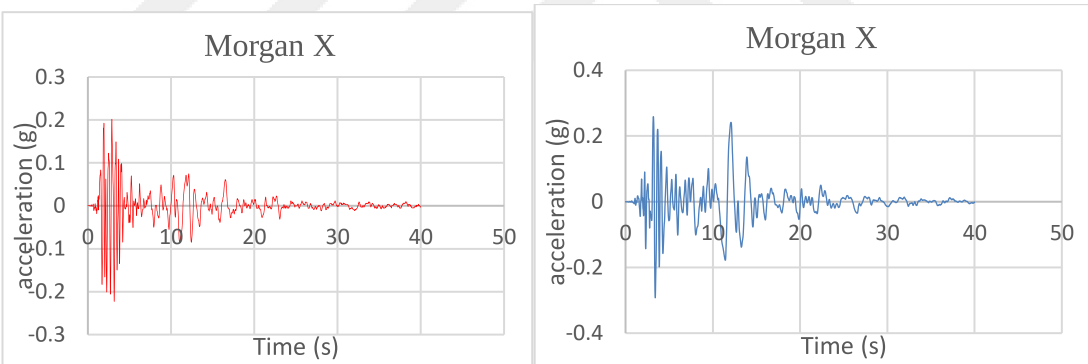


Figure 3. 13 Morgan Original Earthquake data pair

Every earthquake data chosen for this analysis is matched, as required in TBDY 2018, according to the proper design spectrum for DD-2 earthquake level. Matching is executed using Seismomatch software. After the matching two controls are done: average rate and SRSS rate. Both controls are considered for the interval from 0,2 Tp until 1,5 Tp. As shown in Table 3.4 below, every earthquake fulfils the rule that average of acceleration of earthquake and spectra ratio for every period must be at least one. Also, the second control, the ratio between SRSS of earthquake acceleration and spectra must be at least 1,3.

Figure 3.15, Figure 3.16, Figure 3.17, Figure 3.18, Figure 3.19, Figure 3.20, Figure 3.21, Figure 3.22, Figure 3.23, Figure 3.24, Figure 3.25, below are the graphics of the scaled data for all earthquakes.

Table 3. 4 Control of the matched earthquake data according to TBDY 2018

Event	Station	Average (S_{ax} / $S_{a\text{-}DD2}$)	Average (S_{ay} / $S_{a\text{-}DD2}$)	SRSS(S_{ax}, S_{ay})/ $S_{a\text{-}DD2}$	Check
Landers	Amboj	1.02	1.05	1.47 ≥ 1.30	✔
Landers	Big bear Lake	1.07	1.05	1.50 ≥ 1.30	✔
Landers	Barstow	1.07	1.05	1.51 ≥ 1.30	✔
Duzce	Sakarya	1.11	1.06	1.54 ≥ 1.30	✔
Duzce	Lamont 1059	4.17	4.17	551.3 ≥ 1.30	✔
Duzce	Lamont 531	1.05	1.07	1.50 ≥ 1.30	✔
Kocaeli	Iznik	1.07	1.05	1.50 ≥ 1.30	✔
Kocaeli	Eregli	1.03	1.04	1.46 ≥ 1.30	✔
Bigbear	Mt Baldy	1.00	1.05	1.44 ≥ 1.30	✔
Chalfant	Long Valey	1.06	1.06	1.50 ≥ 1.30	✔
Morgan	Gilroy Array 6	1.05	1.05	1.48 ≥ 1.30	✔

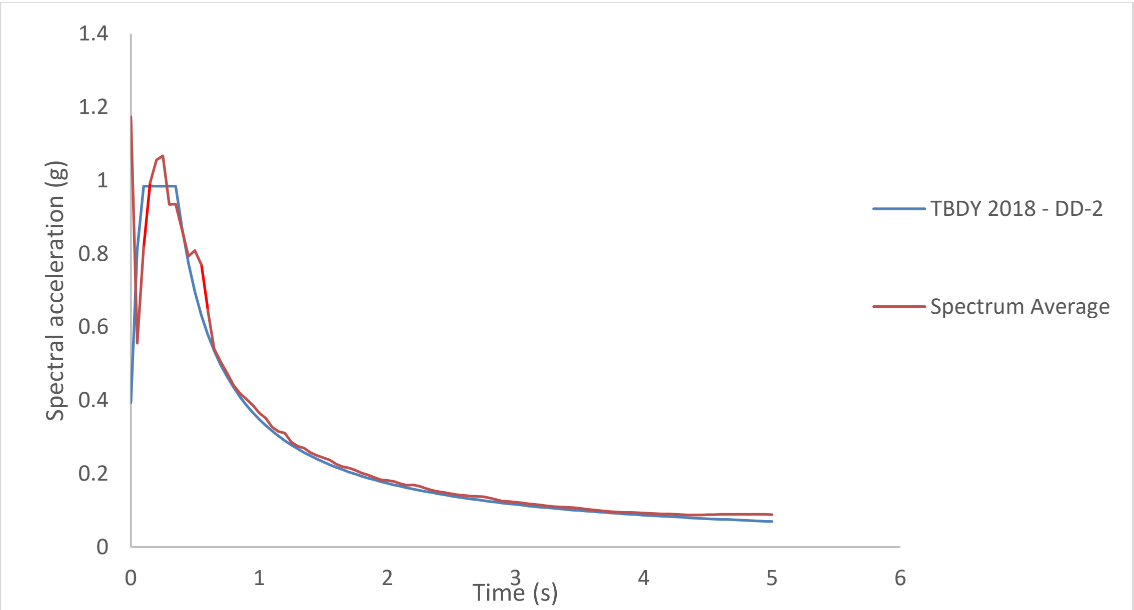


Figure 3. 14 Matching control

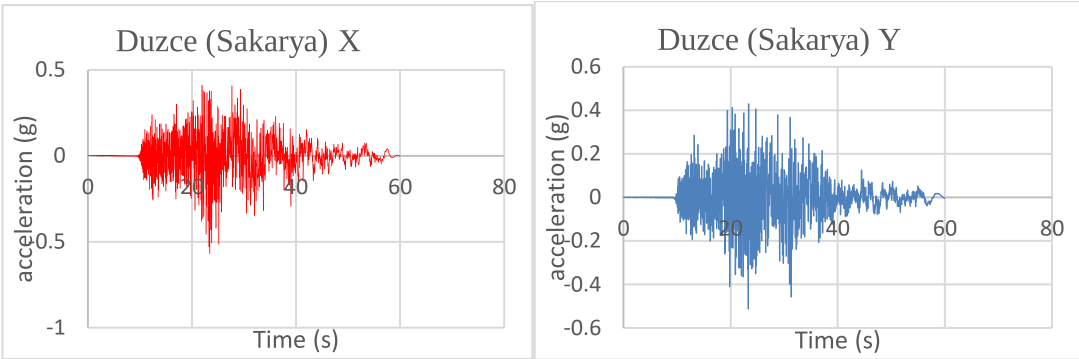


Figure 3. 15 Düzce (Sakarya) Matched Earthquake data pair

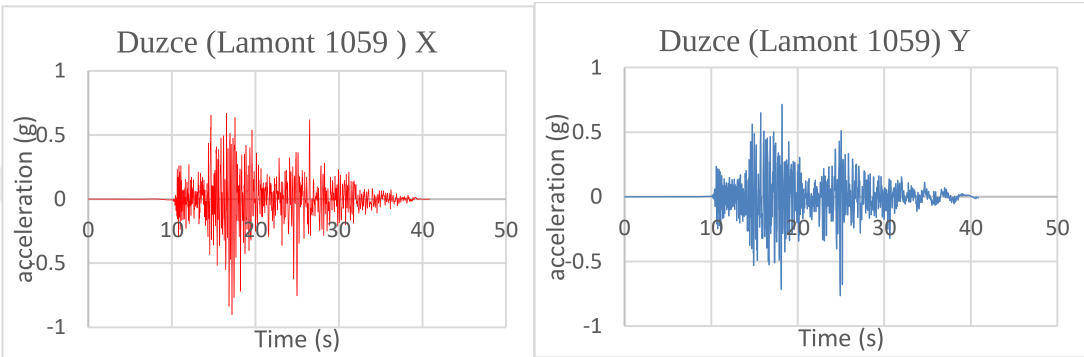


Figure 3. 16 Düzce (Lamont 1059) Matched Earthquake data pair

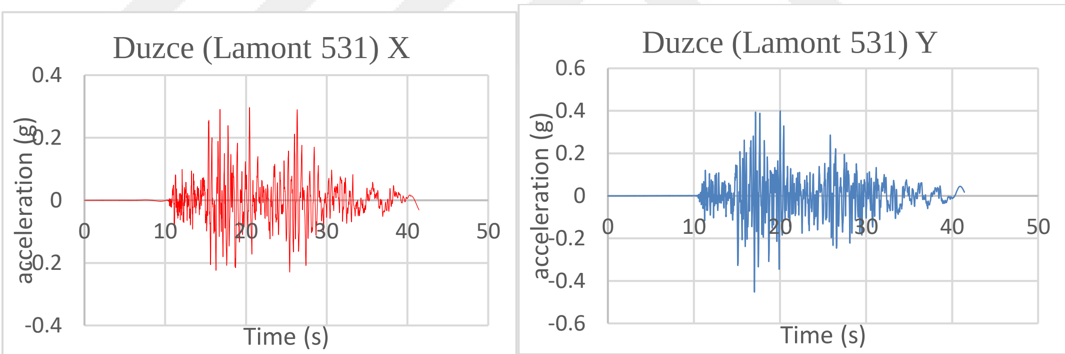


Figure 3. 17 Düzce (Lamont 531) Matched Earthquake data pair

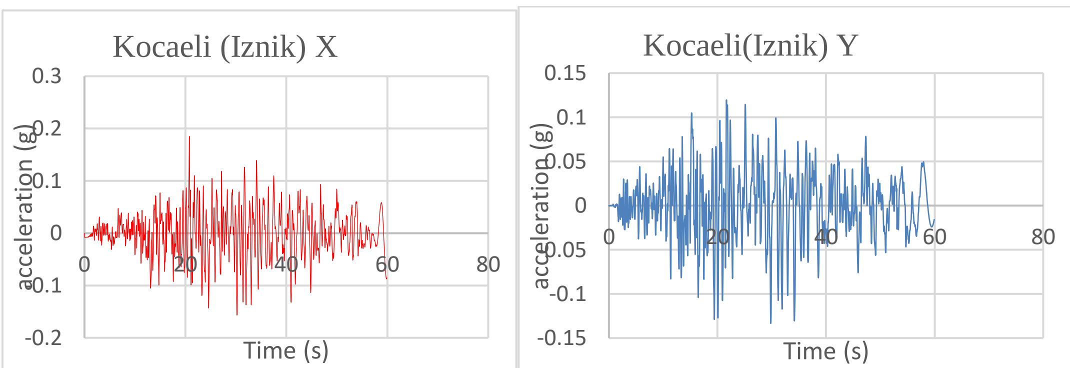


Figure 3. 18 Kocaeli (Iznik) Matched Earthquake data pair

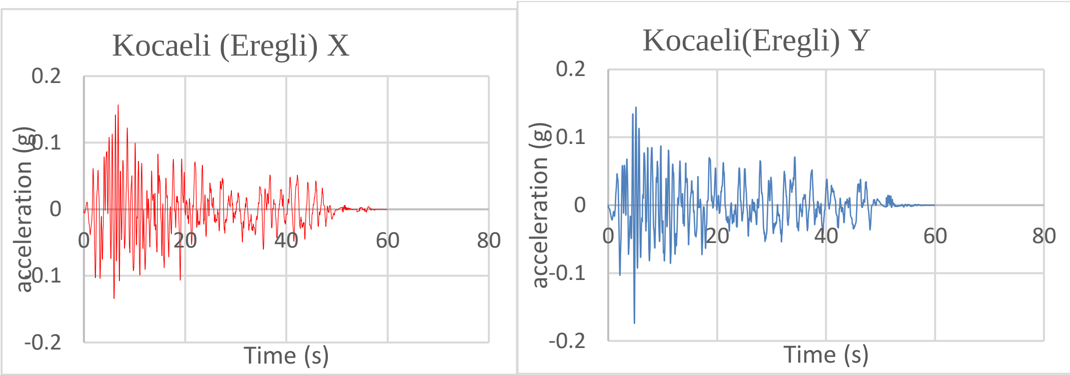


Figure 3. 19 Kocaeli (Eregli) Matched Earthquake data pair

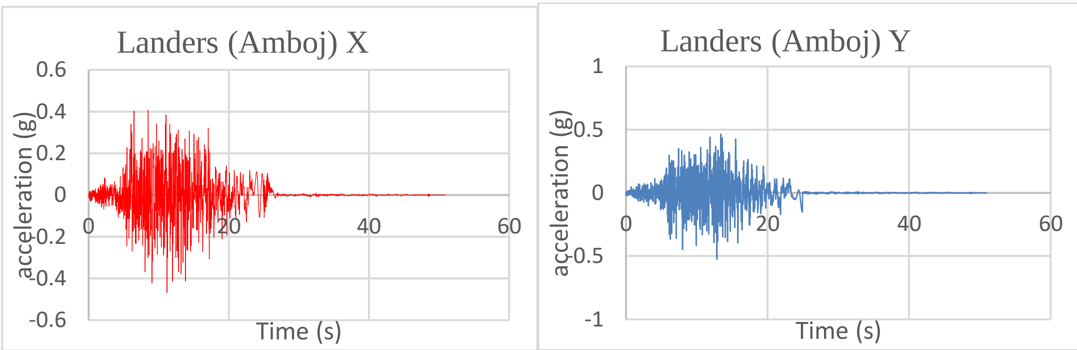


Figure 3. 20 Landers (Amboj) Matched Earthquake data pair

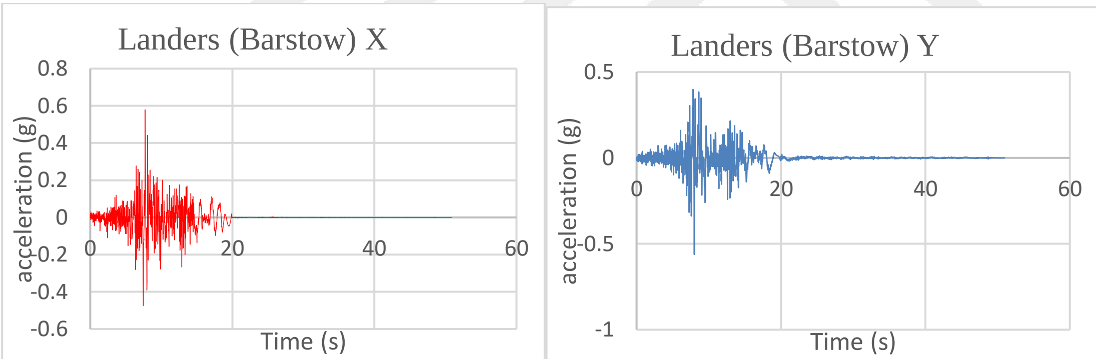


Figure 3. 21 Landers (Barstow) Matched Earthquake data pair

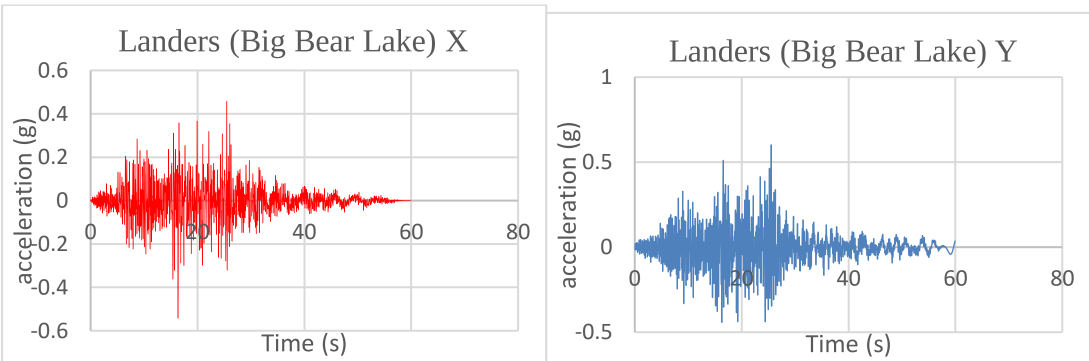


Figure 3. 22 Landers (Big Bear Lake) Matched Earthquake data pair

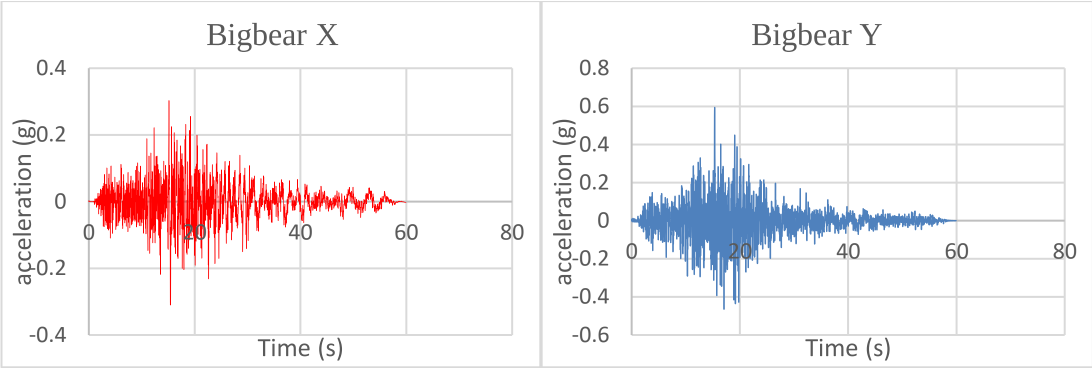


Figure 3. 23 Landers (Big Bear) Matched Earthquake data pair

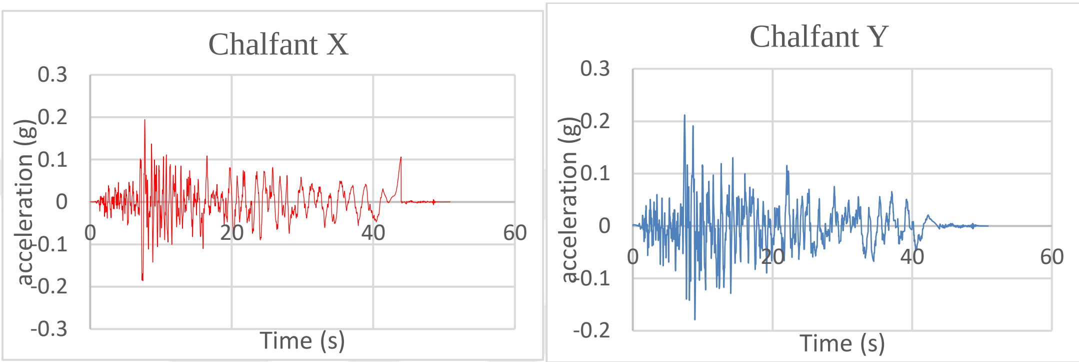


Figure 3. 24 Chalfant Matched Earthquake data pair

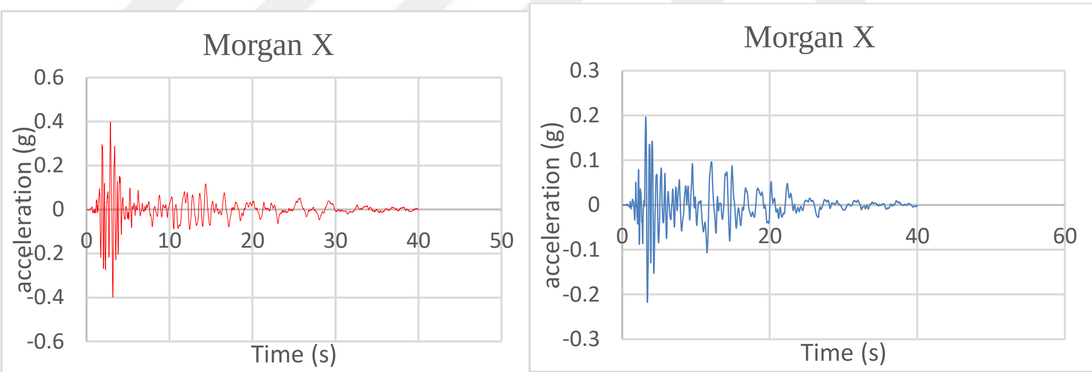


Figure 3. 25 Morgan Matched Earthquake data pair

4. BRIEF EXPLANATION OF CRITERIA ACCORDING TO EC8

4.1 Performance Requirements

4.1.1 Fundamental requirements

Fundamental requirements discuss the damage state of the structure that is divided in three Limit States (LS), Near Collapse(NC), Significant Damage(SD) and Damage Limitation (DL). Some of these Limit States Characteristics are as follows:

- Near Collapse (NC) defines the limite state where the structure is heavily damaged, has lost the lateral stiffness but it manages to sustain the vertical load. Most of non-nostructural elements collapse and large values of drifts outcome. Structure is near collapse and not able to survive another seismic action.
- Significant Damage (SD) defines the limit state where the structure is significantly damaged and an amount of lateral stiffness is left but vertical elements manage to sustain the vertical loads. Moderate permanent drifts occurred. The structure can survive the after-shocks with moderate intensity and its repairing is not an economic solution.
- Damage Limitation (DL) defines the limit state where the structure is lightly damaged and there is no loss in lateral stiffness. Yield deformations are considered negligible. There are some crackings in non-structural elements but they are easily repairable.

According to the national authorities requirements one, two ore three limit states should be checked.

4.2 Knowledge Level

Eurocode8 has defined three knowledge level in order to chose the appropriate confidence factor value and acceptable type of analysis. These levels are:

KL1: Limited knowledge

KL2: Normal knowledge

KL3: Full knowledge

There are three factors defined to determine the correct knowledge level:

- Geometry: generally about the architectural plan of the structure, defines the position of non-structural elements (e.g. masonry infill panels)
- Details: this part includes the details of the structural elements. Every reinforced concrete sections reinforcement amount and detailing is part of this classification.
- Material: the mechanical properties of the materials used in the structure.

Respectivelly to the knowledge level gained by in-situ analysis the proper analysis method is chosen. The relationship between knowledge levels and method of analysis used to determine the structural performance are given in Table 4.1 and in following part some brief explanations are given.

Table 4. 1 Knowledge level characteristics (EC8)

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample **visual** survey ***or*** from **full** survey	Simulated design in accordance with relevant practice ***and*** from **limited** *in-situ* inspection	Default values in accordance with standards of the time of construction ***and*** from **limited** *in-situ* testing	LF- MRS	CF_{KL1}
KL2		From incomplete original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **extended** *in-situ* inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	CF_{KL2}
KL3		From original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	CF_{KL3}

NOTE The values ascribed to the confidence factors to be used in a country may be found in its National Annex. The recommended values are CF_{KL1} = 1,35, CF_{KL2} = 1,20 and CF_{KL3} = 1,00.

Table 4. 2 Knowledge level criteria (EC8)

	Inspection (of details)	Testing (of materials)
	For each type of primary element (beam, column, wall):	
Level of inspection and testing	Percentage of elements that are checked for details	Material samples per floor
Limited	20	1
Extended	50	2
Comprehensive	80	3

4.3 Assesment

4.3.1 General

Assesement is a calculation procedure to check if an existing structure will provide the limit states considering their criteria. The main goal of the assessment is to evaluate whether the structure requires intervention and thereby to design the outfitting project is necessary. The method of assessment is carried out by means of the general analysis methods specified in EN 1998-1 : 2004. If it is possible it is preferred to include the assessment analysis of similar structures during previous earthquakes.

4.3.2 Seismic action and load combinations

Earthquake motion, according to EC8, is presented by scaling the seismic data refering to the elastic ground acceleration response spectrum. There are two alternatives allowed: artificial and recorded data.

Seismic loads must be combined with other permanent and variable load in accordance with EN 1998-1 2004 chapter 3.2.4.

The inner effects of the seismic action should be evaluated considering the presence of masses which associates every gravity load.

4.3.3 Structural modelling

Informations required for structural modeling:

- Structural regularities criterias shall be checked. The informations about the structure should be collected from on site measurements and from project drawings. If any structural change was done after the constructions it should be included.

- Building foundations type is identified.
- Soil conditions are defined as chategorised in EN 1998-1: 2004
- Every structural elements cross-sectional dimension and the mechanical properties of used materials
- Inadequate detailing and material defects
- Information about the seismic loads used for initial design and force reduction factor (q-factor)
- Importance class of the structure
- Information about any damage and outfit action in the paste
- Based on collected informations a model should be set up.

4.3.4 Methods of analysis

The effect of the seismic action combined with permanent and variable loads, according to the seismic code, can be evaluated using one of the following methods:

- Lateral force analysis (linear)
- Modal response analysis (linear)
- Non-linear static (pushover) analysis
- Non-linear time-history analysis

In every method used, seismic action should correspond to the elastic (un-reduced by the baviour factor q) response spectrum.

Analysis method used in this thesis is non-linear time-history analysis thereby only this method will be discussed.

4.3.4.1 Non-linear time history analysis

As an important alternative, that is commonly used in performance evaluation is the full non-linear time history analysis. The non-linear model of the structure is analyzed under a time-history whose data matches the acceleration spectrum. EC8 requires minimum 3 seismic actions data are used to perform at least seven different anlysis. The average of analysis is used to evaluate the elements performance.

4.3.5 Damage limitations

Damage Limitation is considered satisfied in situation when the structure under a seismic action with a larger occurance probability than the design seismic action corresponding to Near Collapse requirement, according to seismic zone, intersotrey drift are within limited values as mentioned below.

4.3.6 Plastic hinge length

Depending on the method used to obtain the value of ultimate curvature (Φ_u), Eurocode 8 has defined two equations to calculate the plastic hinge length. The equations are as follows:

$$l_p = 0.1L_V + 0.17h + 0.24\frac{d_{bL}f_y}{\sqrt{f_c}}$$
$$l_p = \frac{L_V}{30} + 0.2h + 0.11\frac{d_{bL}f_y}{\sqrt{f_c}} \quad (4.1)$$

L_V is the shear span, d_{bL} is the longitudinal bar diameter.

4.3.7 Limit values of axis rotations

Different from TBDY2018 that defines the sections damage using the strain values, EC8 calculates the elements damage using the chord rotation. Chord rotation is defined as follows:

$$\theta_i = \frac{\delta_i}{L_{vi}} = \left|\frac{\Delta}{L} - \phi_i\right| \quad (4.2)$$

θ_i is the value of axis rotation

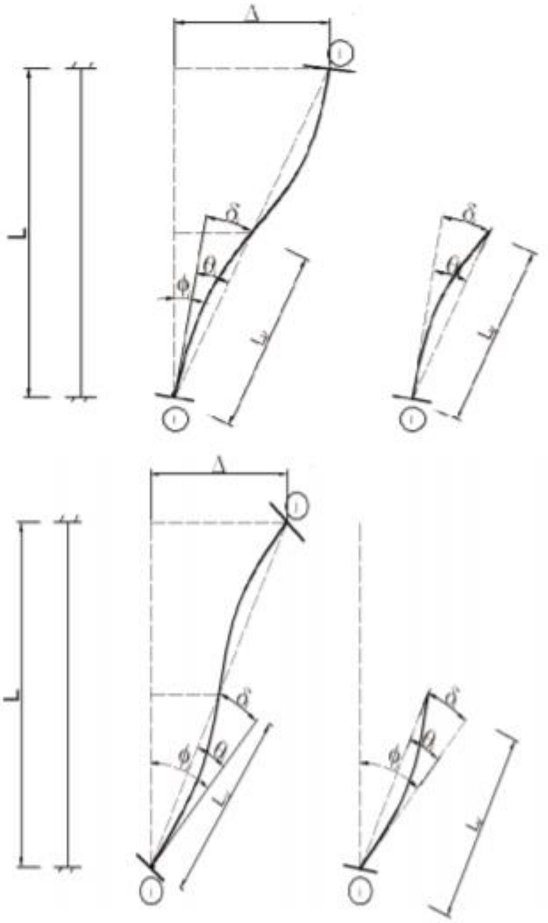


Figure 4. 1 Axis Rotation (Özal, 2005)

4.3.7.1 Axis rotations limit values for DL damage zone

The value of the calculated axis rotation for the Damage Limit must be lower than the value of axis rotation at sections yield point. Axis rotation value at section yield point is calculated using the formula bellow.

$$\theta_y = \phi_y \frac{L_V + \alpha_V z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_V}\right) + 0.13\,\phi_y\,\frac{d_b f_y}{\sqrt{f_c}} \tag{4.4}$$

Where, d is the tensile rebar depth, d’’ is the pressure rebar depth, d_b is the longitudinal rebar diameter, L_v is the shear span (M/V), a_v for ductile sections is 0. The unit of the result is MPa.

4.3.7.2 Axis rotations limit values for SD damage zone

The value of the calculated axis rotation for the SD must be lower than ¾ of the value of axis rotation calculated in 4.3.7.3. ($\theta_{SD} \leq$ ¾ θ_{um})

4.3.7.2 Axis rotations limit values for NC damage zone

The value of the calculated axis rotation for the NC must be lower than the value of ultimate axis rotation. For ductile section ultimate axis rotation is calculated using the following formula.

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016\,(0.3^{\nu}) \left[\frac{max(0.01,\ \omega_2)}{max(0.01, \omega_1)} f_c\right]^{0.225} \left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.25^{100\,\rho_d}) \tag{4.5}$$

4.4 Soil Conditions

Soil Conditions and Seismic Ground Motion

Depending on the layers and their parameters soils are divided in five different types as presented in Table 4.3. The soil conditions are obtained by researches, compatible to these five categories. In seismic design importance level and specific properties of structure should also be considered.

Table 4. 3 Soil types and parameters (EC8)

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

If there are information about particular soil properties the categorization should be done firstly by using the Average Shear Wave Velocity, $V_{S,30}$, if it not available the we use the N_{SPT} value calculated by SPT geotechnical experiment.

4.5 Horizontal Elastic Acceleration Spectrum

Horizontal Elastic Acceleration Spectrum which represents horizontal components of the seismic motion is shown in Figure 4.2 and depends on parameters as follows

$$S_e(T) = a_g * S * \left(1 + \frac{T}{T_B}(\mu * 2{,}5 - 1)\right) \qquad 0 \leq T \leq T_B \qquad (4.6)$$

$$S_e(T) = a_g * S * \mu * 2{,}5 \qquad T_B \leq T \leq T_C \qquad (4.7)$$

$$S_e(T) = a_g * S * \mu * 2{,}5 * \left(\frac{T_C}{T}\right) \qquad T_C \leq T \leq T_D \qquad (4.8)$$

$$S_e(T) = a_g * S * \mu * 2{,}5 * \left(\frac{T_C}{T^2} * T_D\right) \qquad T_D \leq T \leq 4s \qquad (4.9)$$

$S_e(T)$: Elastic acceleration spectrum

T : Vibration period of a linear single degree of freedom system

a_g : Design ground acceleration for type A soil

T_B : Down limit of constant spectral acceleration zone

T_C : Up limit of constant spectral acceleration zone

T_D :Starting point of constant deformation spectrum

S :Soil Factor

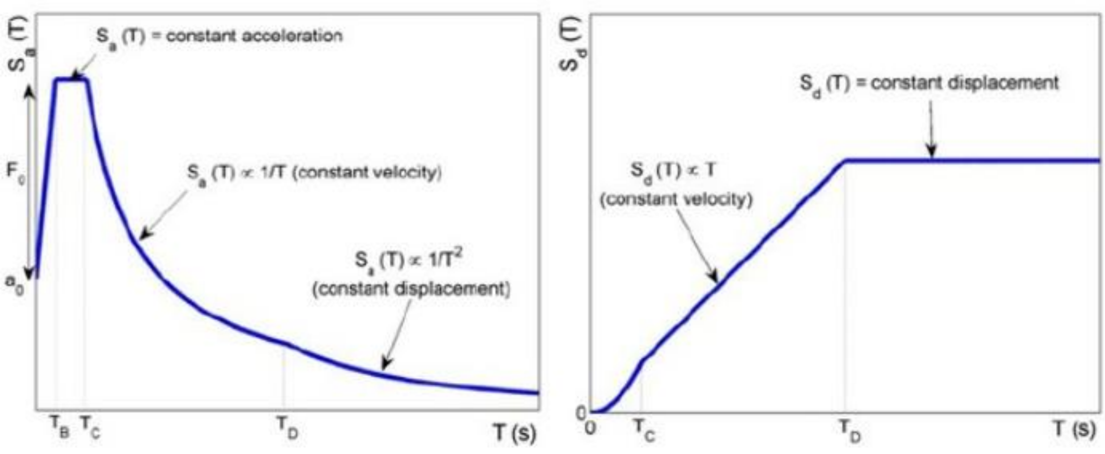


Figure 4. 2 Horizontal elastic acceleration and displacement spectrum (EC8)

Required parameters to obtain the response spectrum type 1 and type 2 are shown in Table 4.4

Table 4. 4 Horizontal Elastic Acceleration Spectrum parameters (EC8)

	Seismic action Type 1				Seismic action Type 2			
Ground Type	S	T_B (s)	T_C (s)	T_D (s)	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0	1,0	0,05	0,25	1,2
B	1,2	0,15	0,5	2,0	1,35	0,05	0,25	1,2
C	1,15	0,2	0,6	2,0	1,5	0,1	0,25	1,2
D	1,35	0,2	0,8	2,0	1,8	0,1	0,3	1,2
E	1,4	0,15	0,5	2,0	1,6	0,05	0,25	1,2

EC8 recommended to use one of these two types of spectrums. If the surface wave magnitude , M_s of the earthquake that affects the location of the structure is larger than 5,5 type 1 is used, otherwise type 2 is chosen.

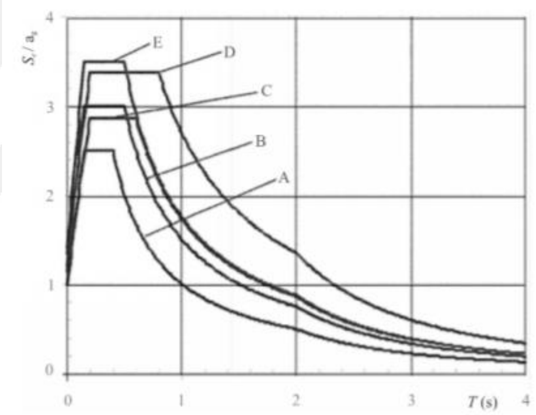


Figure 4. 3 Type 1 elastic acceleration spectrum recommended for every soil type (5% damping) (EC8)

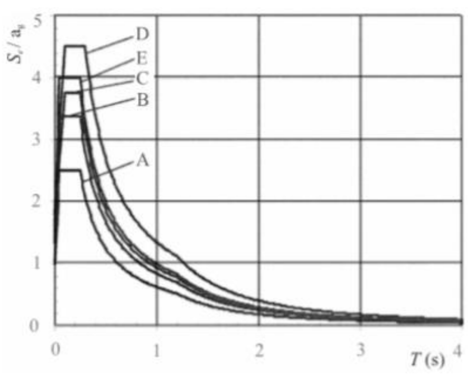


Figure 4. 4 Type 2 elastic acceleration spectrum recommended for every soil type (5% damping) (EC8)

5. SEISMIC PERFORMANCE EVALUATION OF 24 STORY RC BUILDING BY NONLINEAR TIME HISTORY ANALYSIS

5.1 Entrance

In this chapter time history analysis will performed using the principals of the new earthquake design code TBDY 2018 of an existing, 24 story, Shear Wall and Frame combined structure designed according to Turkish old seismic code DBYBHY 2007 (The code for the structures build in earthquake zones). After the TBDY 2018 the performance will also be checked by using EC 8 and the results will be compared. As required in the TBDY 2018 structure after being exposed to the earthquake with probability of 10% to occur in next 50 year will show KH or controlled damage. Within the thesis, respecting the requirements of TBDY 2018, 11 different earthquake data pair in two directions are loaded to the structure and results are discussed. The first part of the chapter includes the explanation of the structure and the time history nonlinear analysis according to TBDY 2018 while the second part explains the same analysis made utilizing EC 8. Last part is a conclusion and a comparation of both parts mentioned above.

5.2 General Information About the Structure

The structure is found in district Kağıthane in Istanbul with coordinates 41.060864° and 28.959906° respectively. The soil type is ZC (very dense sand, gravel or very stiff clay) according to TBDY 2018 which responds to the B type according to EC8. Structure was designed using the old design code DBYBHY 2007 (Code for the structures built in earthquake zones).

Structures base area changes with the floors. It varies from 570m^2 in first three floors following with 414m^2 in other floors. The height of the stories is 3,2m in most floors. Figure 5.1 and figure 5.2 describe the formwork plan of the two characteristic floors.

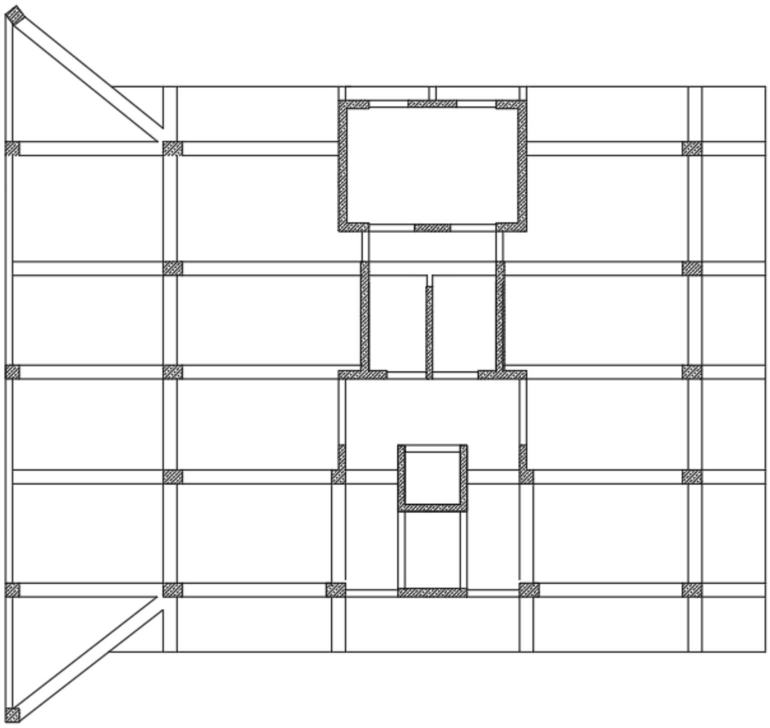


Figure 5. 1 Basement floor formwork plan

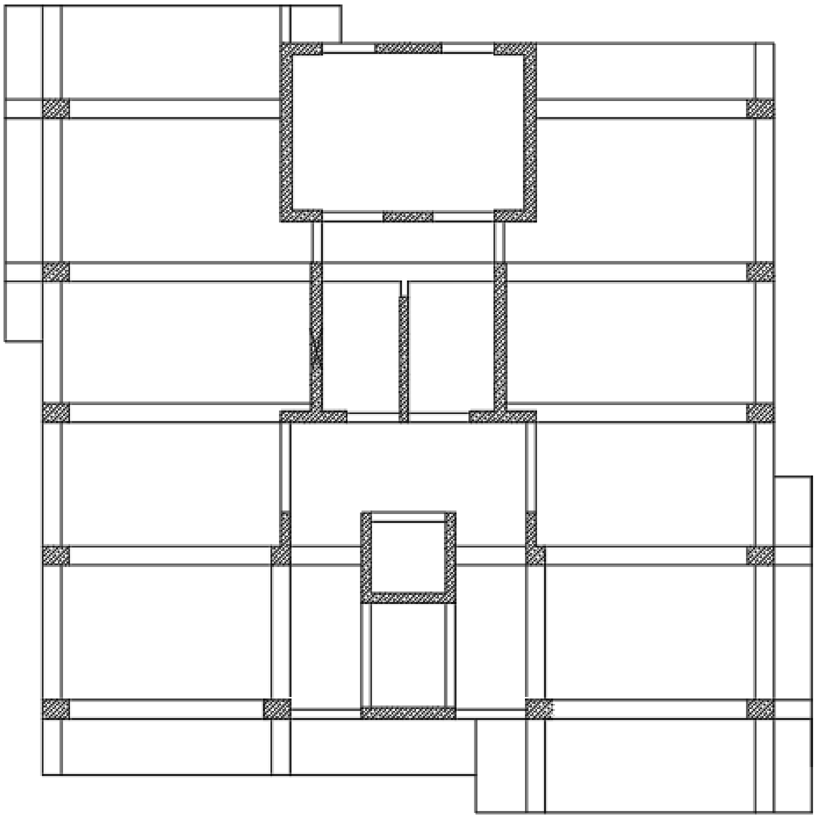


Figure 5. 2 5th floor formwork plan

As a result of control of the structure it is declared that the static project is applied properly and the data founded in the static project can be used to evaluate the performance of the structure. This conclusion means that the knowledge level that we have about the structure is full knowledge level and the coefficient provided for this knowledge level can be used. According to the information collected from the existing project properties of the structural systems are as follows:

- Floor number: Basement floor + 23 Normal floors
- Structure type: RC frame and shear-walls combined system
- Slab type: Beam supported slab
- Foundation type: Mat foundation
- Soil category: ZC
- Usage purpose: Office

As it is found in the project the thickness of the slab is 13cm in normal slab and 16cm in cantilever slabs. There are two types of columns: 50/50(cm) and 70/50(cm). The first column is found only in first four floors in the side part of the structure. Also, there are three types of beams: 50/40(cm) and 25/50(cm) used in all floors and 70/40(cm) beam which is used only in second floor to surround the opening in the slab.

5.2.1 Loads effecting the structure

The dead load that comes from the weight of the elements is calculated automatically from the software by considering the specific gravity as 25kN/m^3 which is assigned when the material is defined. At the other side the loads which come from the elements different from RC whether they are live or dead are described in table 5.1

Table 5. 1 Live and Dead loads

Slab Type	Dead load	Live Load
Normal Slabs	g=2kN/m^2	q= 3kN/m^2
Cantilever Slabs	g= 2kN/m^2	q= 5kN/m^2

5.2.2 Reinforcement details of structural elements

Table 5. 2 Column Sections

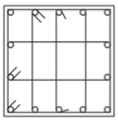
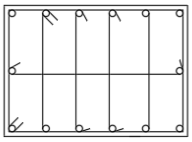
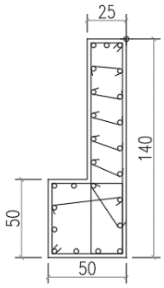
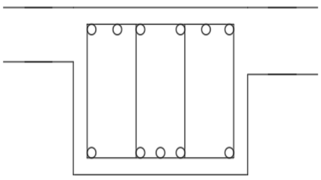
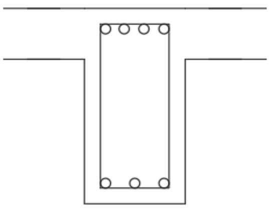
Column Sections	Dimensions (b/h) (cm)	Longitudinal bar	Cover bars
	50/50	14Φ16	Φ10/10
	50/70	14Φ20	Φ10/10
	L50/140/25	24Φ16	Φ10/10

Table 5. 3 Beam Sections

Column Sections	Dimensions (b/h) (cm)	Longitudinal bar Bottom	Longitudinal bar Bottom	Cover bars
	50/40	5Φ14	6Φ14	Φ8/10
	25/50	3Φ14	4Φ14	Φ8/10

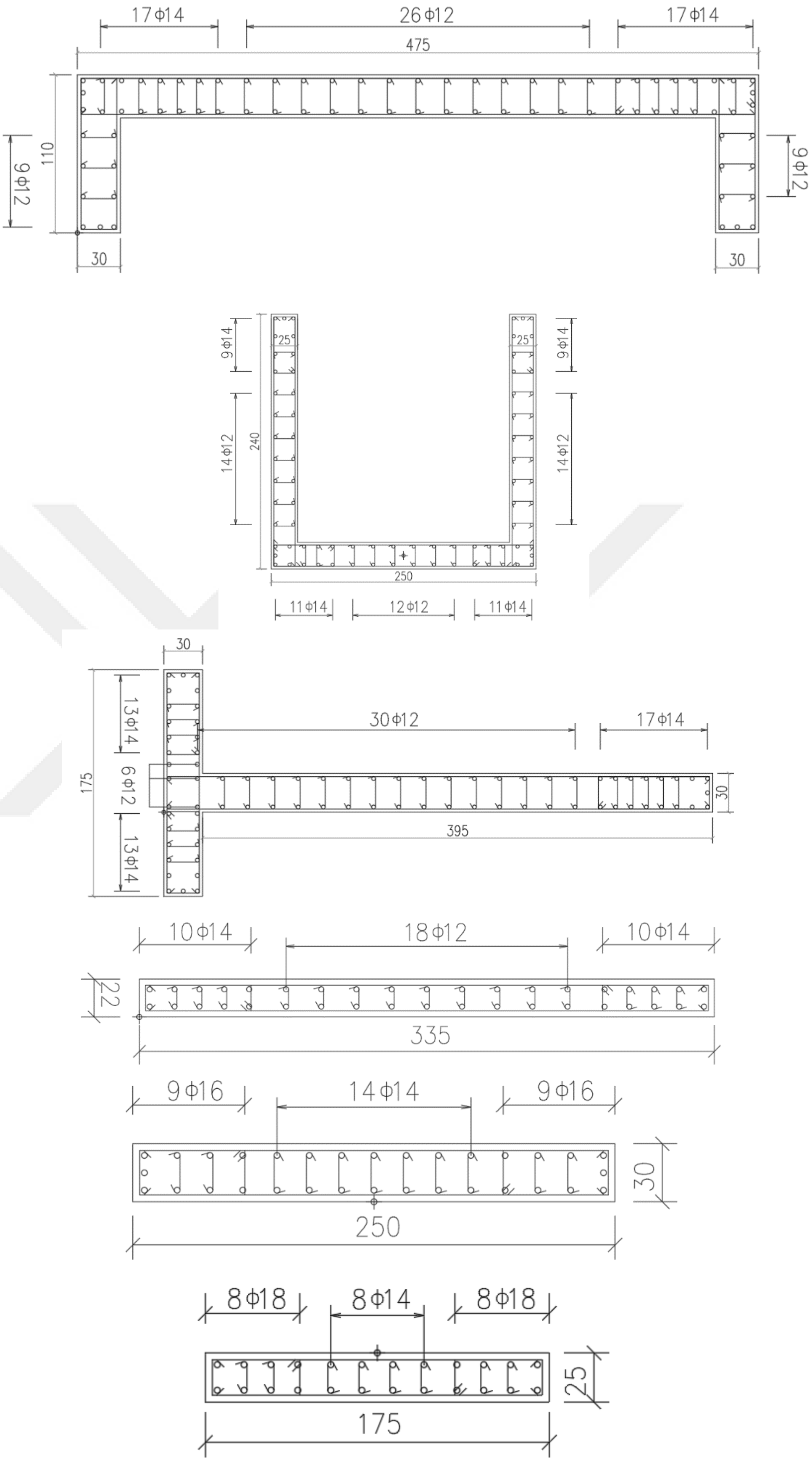


Figure 5. 3 Shear Wall sections

5.2.3 Material properties of existing structure

Properties of the materials used in the structure are taken from the project and no analysis and experiments of the materials were required because the tests of the materials were done during construction. Materials used are C35 concrete and S420 reinforcement steel. Properties of the reinforcement steel are defined in TBDY 2018. As for the properties of concrete, it is divided in two groups where each of them has different properties. Stress-Strain relation graphics are calculated using ETABS and XTRACT as shown in figure 5.4 below

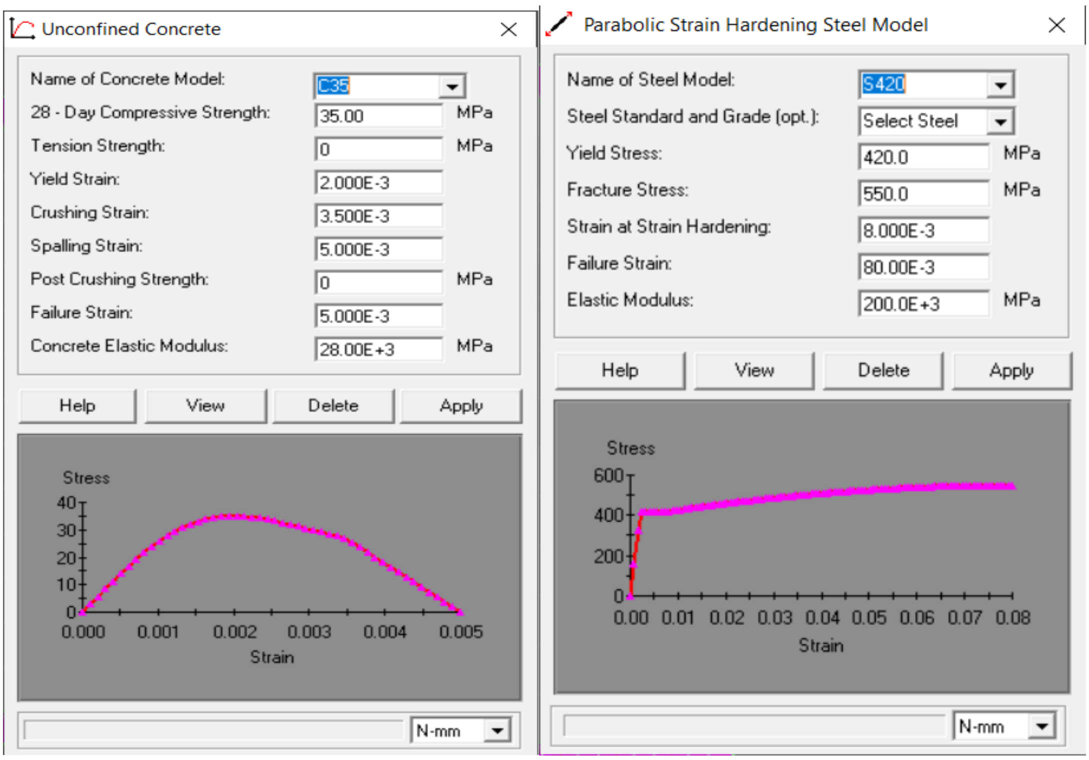


Figure 5. 4 Stress-Strain Relations of unconfined concrete and steel calculated by XTRACT

For the calculation of confined concrete there are two options: to enter the data of the longitudinal and cover reinforcement directly to the Xtract or to calculate the data and enter the results to Xtract. In order to provide more realistic solutions in this thesis the second option is chosen. The calculation is done using the rules in TBDY2018 in chapter 5A.1 a)

Table 5. 4 Calculation of stress and strain of confined concrete

INCOME		OUTCOME			
f_{co}	35	A_c	195364		
f_{ywd}	420	ρ_{cc}	0.014408		
ε_{co}	0.002	A_{cc}	192549.1		
ε_{su}	0.08	ρ_x	0.003412		
b	500	ρ_y	0.006823		
h	500	ρ_c	0.010235		
b_c	442	A_e	146323.6		
d_c	442	k_e	0.759929		
d'	25	f_{ex}	1.088908		Xtract
Φ	10	f_{ey}	2.177815	f_{cc}	47.74
s	100	f_e	1.633361	ε_{cc}	0.00564
s'	90	λ_c	1.291424	ε_{cu}	0.01983
W_i	77.42857	f_{cc}	45.19984		
		ε_{cc}	0.004914		
		E_{sec}	9197.727		

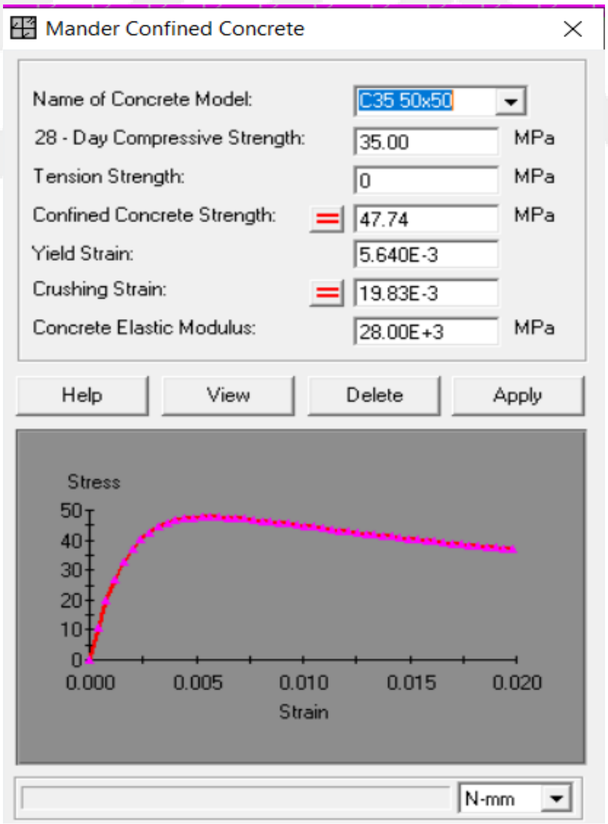


Figure 5. 5 Stress-Strain Relations of confined concrete by XTRACT

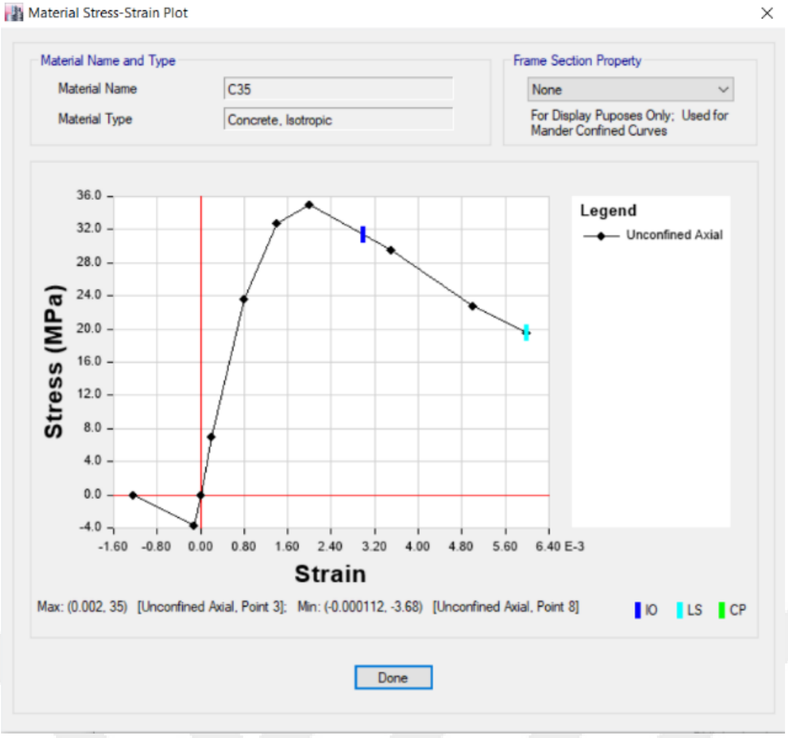


Figure 5. 6 Defining C35 in ETABS

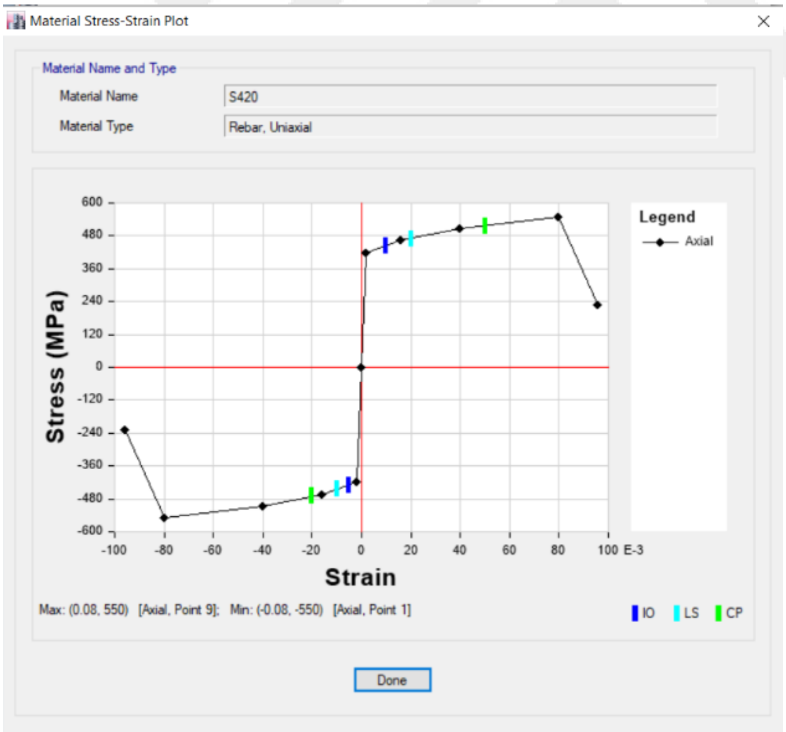


Figure 5. 7 Defining S420 in ETABS

5.3 Evaluation of the Performance of an Existing RC Structure with Non-linear Time History Analysis

5.3.1 Defining Three dimensional ETABS model

In order to evaluate the performance, the mathematical model is done using ETABS v16.2.0. Plan views of some characteristic stories are shown in figure 5.8.

Figure 5. 8 Plan views of different stories

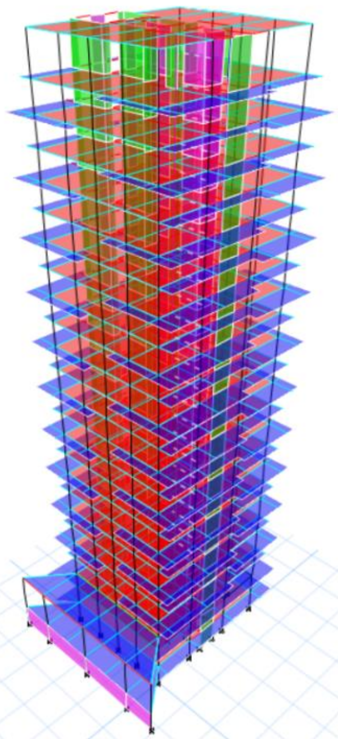


Figure 5. 9 3D view of ETABS model

5.3.2 Earthquake parameters and design spectrum

5.3.2.1 Earthquake parameters and design spectrum according to TBDY-2018

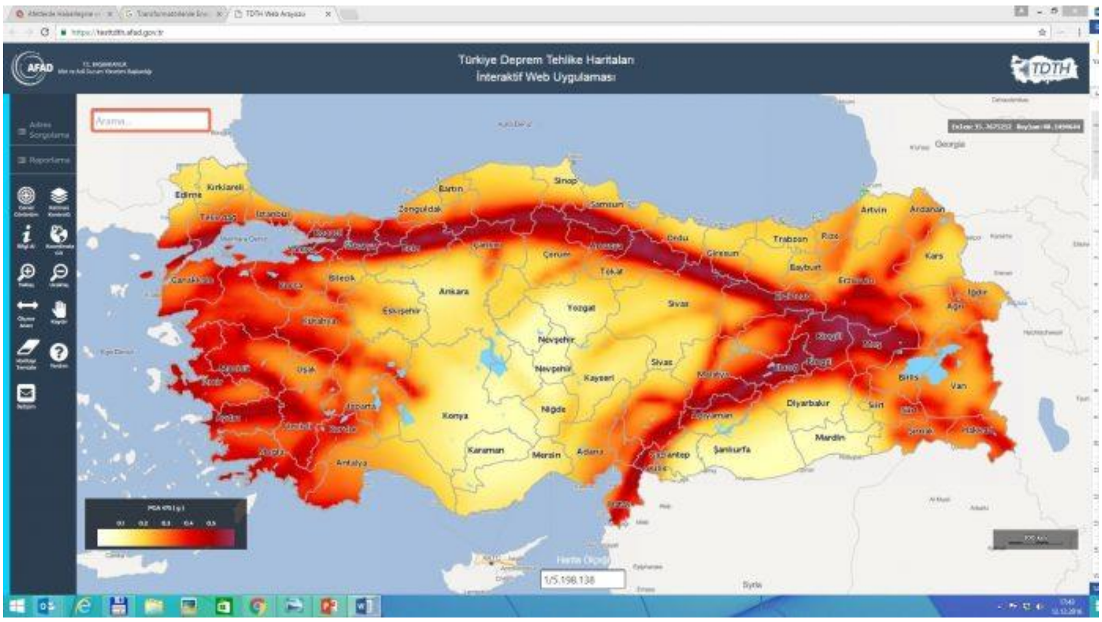


Figure 5. 10 TBDY2018 Earthquake map

Defining the design spectrum and earthquake parameters, according to TBDY 2018 has changed when compared to the old seismic code. One of the most fundamental changes is the evaluation of PGA. The old design code has defined it depending on earthquake zone while the new code, depending on coordinates location, gives parameters which help calculation of PGA and consequently leads to design spectrum. The spectrum parameters are defined in Chapter 3 of TBDY – 2018. Calculations is done by entering the coordinates, the soil type and the earthquake level in https://tdth.afad.gov.tr website.

The structure is found in district Kağıthane in Istanbul with coordinates 41.060864° and 28.959906° respectively. The soil type is ZC (very dense sand, gravel or very stiff clay) and earthquake category is DD-2 (possibility to occur in next 50 years is 10%). Data given by official website of afad.tdth.afad.gov.tr is as follows:

S_S=0,82 S_1=0,232   PGA=0,34g

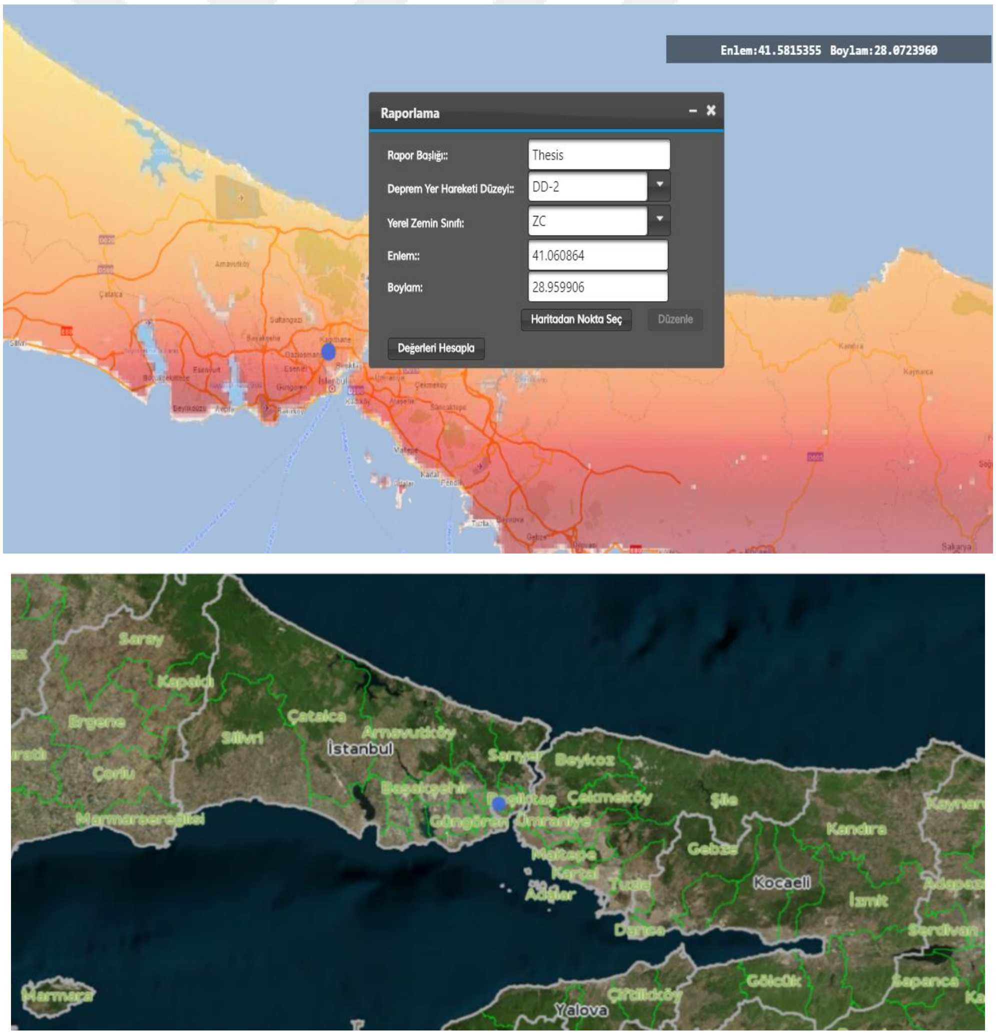


Figure 5. 11 Location of the structure

Using the tables 2.1 and 2.2 from TBDY2018 F_s (Soil reaction factor for long periods) and F_1 (Soil reaction factor for period of 1s). F_S=1,3 and F_1=1,5 is found. Using the values mentioned above we manage to plot the response spectra grafic as follows:

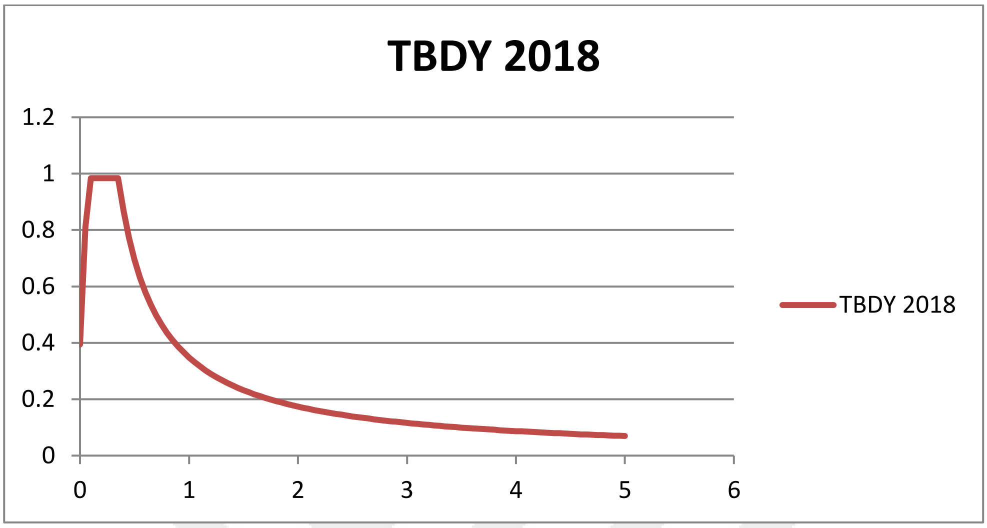


Figure 5. 12 Horizontal elastic design spectrum

5.3.2.2 Earthquake parameters and design spectrum according to EC8

As it is explained in chapter 4 depending on the value of magnitude there are two types of response spectrum. The level of seismic motion used in this thesis is DD-2 which corresponds to the Type 1 spectrum according to EC8. The mutual property of both earthquakes is the reoccurence period equal to 475 years. The a_{gR} value needed for the spectra is read from the seismic hazard region map shown in figure 5.13 below.

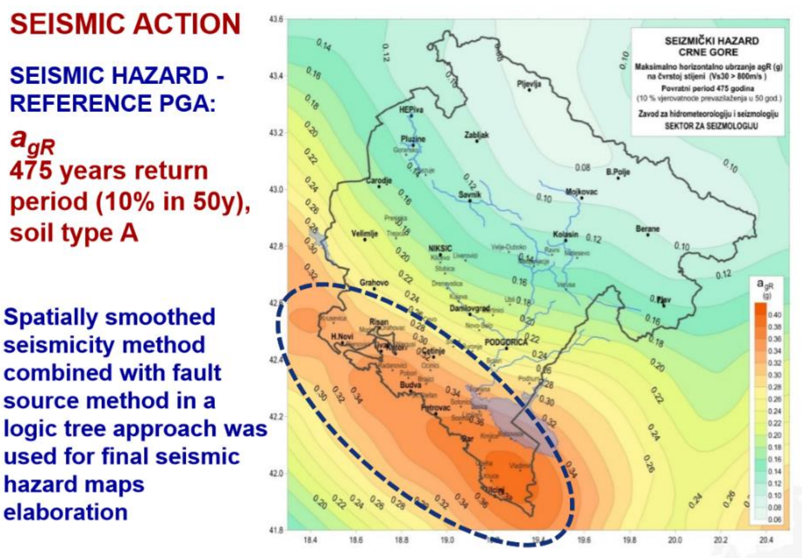


Figure 5. 13 EC Earthquake zone map

The a_{gR} is estimated as 0,34 g

The next value required for response spectrum is the importance class value. As our object is an ordinary building it belongs to importance class II.

Table 5. 5 Importance class of buildings EC 8

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

After the soil research is done in the field at the case of TBDY-2018 it was acknowledged that the type of soil is ZC, with Vs value 360-800m/s. By controlling the table of soil categories in chapter 4 we find out that the soil which corresponds to type ZC is type B, according to EC8

Considering all the factors mentioned above the graphic is plotted in figure 5.14 below.

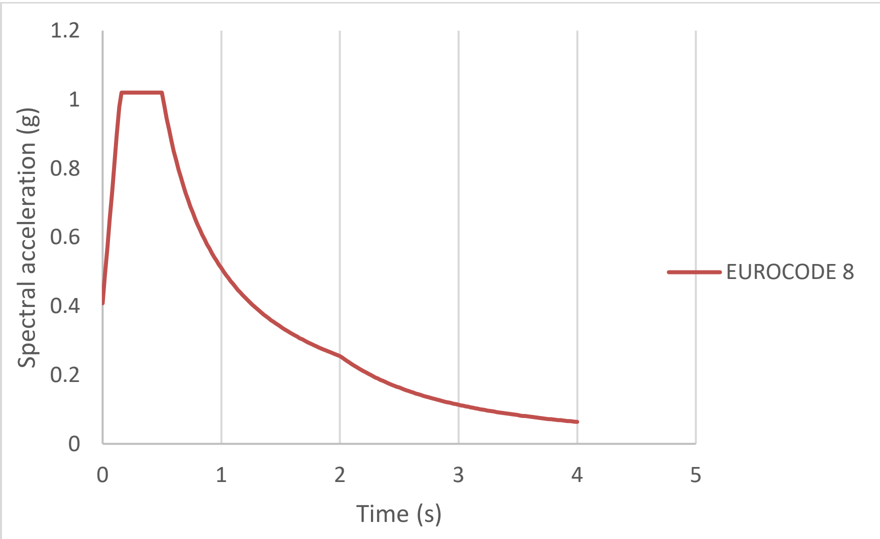


Figure 5. 14 Horizontal Elastic Spectrum

5.3.3 Defining the effective shear and bending stiffness of cracked sections

Chapter 15 of TBDY 2018 defines the general principals and rules of earthquake calculations. According to those principals, in earthquake calculations there are effective stiffness coefficient values used for cracked sections. Table 2.3, in TBDY2018, defines the values of those coefficients. In Figure 5.15 there is an example of column section and in figure 5.16 an example of beam section.

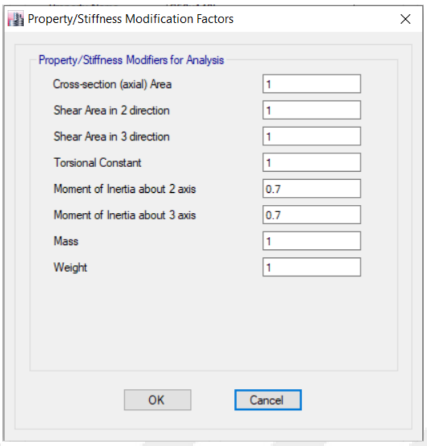


Figure 5. 15 Effective shear and bending stiffness of cracked column

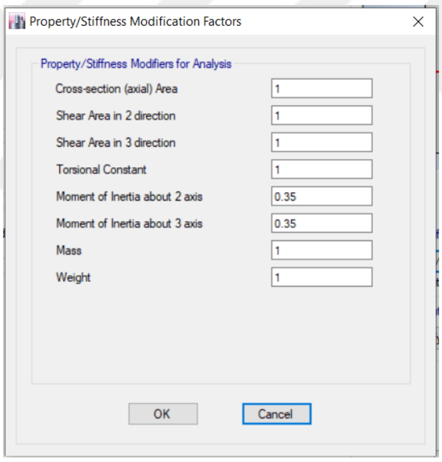


Figure 5. 16 Effective shear and bending stiffness of cracked column

5.3.4 Assignment of plastic hinges

Definition of plastic hinges is based in the rule that all plastic deformation is collected in those sections and all other parts of the elements show an elastic behavior. In order to define the plastic hinges of the elements ETABS software gives us two options. The first option is to automatically obtain the characteristics of the sections from the program and the second one is to calculate them separately and assign the results to the elements. In order to obtain a more realistic solution in this thesis the section properties are calculated manually. The outcome of the calculation is compared to the

ETABS results of automatic hinge assignments. The characteristics of defined plastic hinges are mentioned in the paragraph below.

5.3.4.1 Plastic hinges of beams (M3)

The most important factor which helps us to define the plastic hinge of the beams is the bending reaction around its surface. We obtain this due to the fact that beams have no axial load reaction and therefore we use the hinge type M3. By considering that plastic hinge occurs in areas near the supports and the reinforce bars distributions are different at the supports, there must be done separate calculations for beams with different cross-sections and reinforcement bar distribution. The sections with mutual properties are collected as groups and are assigned with same values. As the result of asymmetry of beam sections around the neutral axis, for every beam section two-direction (positive and negative) calculations are made. In order to obtain the load-strain diagrams of the beams Xtract software is used. In part below it is explained the calculation of hinge properties for B500/400.

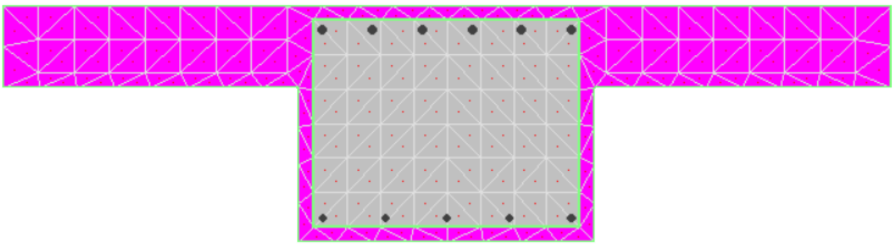


Figure 5. 17 Edge Section of Beam B500/400

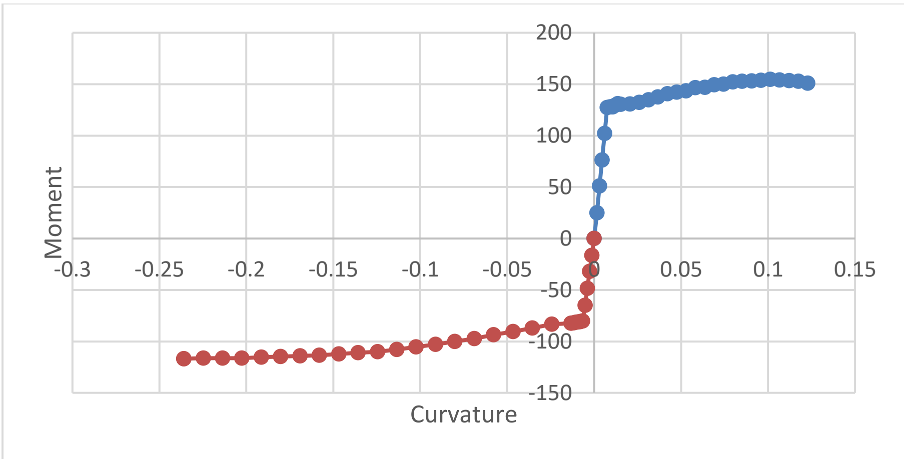


Figure 5. 18 Moment-curvature relationship of beam B500/400 for positive and negative direction

Calculation made using XTRACT provides the values of yield and ultimate moments and values of yield and ultimate curvature. Using the value of yield curvature and plastic hinge length of the beam we get the yield rotation value. The same process is repeated for ultimate curvature. Using the rule that sum of plastic and yield rotation gives the ultimate rotation we manage to calculate the plastic rotation of beams. The calculation and formulas used for beam B50/40 are showed below.

$$\Phi_y = \frac{\theta_y}{L_p} \tag{5.1}$$

$$\Phi_U = \frac{\theta_U}{L_p} \tag{5.2}$$

$$\theta_U = \theta_y + \theta_{Up} \tag{5.3}$$

Table 5. 6 Plastic hinge properties

Beam	Zone	Lp	My	φy	θy	Mu	φu	θu	θp
B50/40	Negative	200	93.48	0.0058	0.00116	116.8	0.236	0.0472	0.0414
	Positive	200	136.1	0.0091	0.00183	150.9	0.1227	0.02454	0.0154
B70/40	Negative	200	127.7	0.0123	0.00247	147.8	0.2444	0.04888	0.0366
	Positive	200	143.2	0.0053	0.00106	205.4	0.1103	0.02206	0.0168
B25/50	Negative	250	96.06	0.0077	0.00154	120.8	0.1827	0.03654	0.0288
	Positive	250	116.4	0.0052	0.00103	137.6	0.1199	0.02398	0.0188

Calculated values are compared to the automatic assigned hinges on ETABS

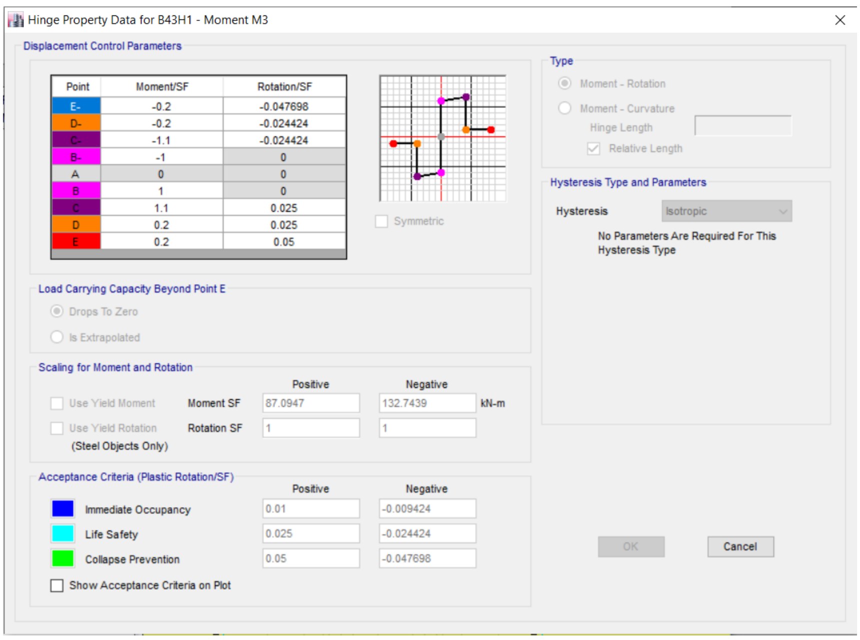


Figure 5. 19 Automatic values obtained by ETABS for section B500/400

The values of positive and negative yield moment of beam B40/50, assigned by ETABS, are My=132 and My=87kNm respectively. When moment and plastic rotations are compared to the values calculated by XTRACT their similarity is concluded and the values are not changed, they are left as they are assigned automatically by ETABS.

5.3.4.2 Plastic hinges of columns (P-M3-M2)

The fact that columns are affected from two-directional bending and axial load it is required that the plastic hinge is of type P-M2-M3. In order to define the properties of column plastic hinges, firstly, the yield surface of column and the axial load-moment diagrams is obtained. The yield surface and moment-curvature relations are calculated using Xtract. As an example, the results obtained from Xtract for the section C50x50 are compared to the results obtained from ETABS.

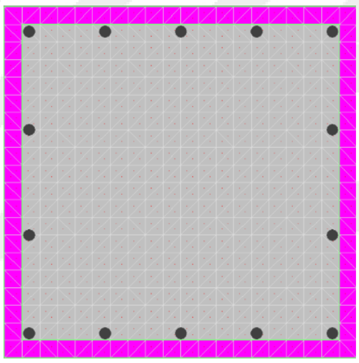


Figure 5. 20 Extract model of column C50x50

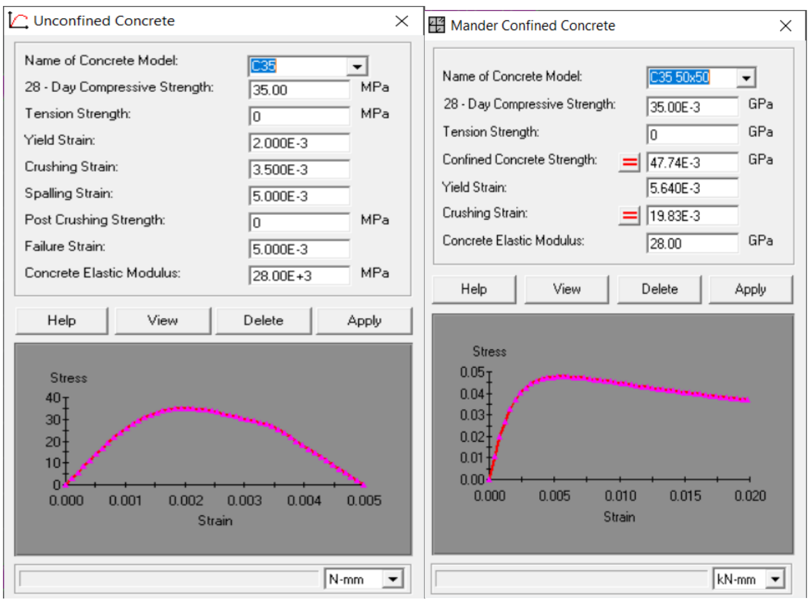


Figure 5. 21 Confined and unconfined concrete model of C50x50 Beam

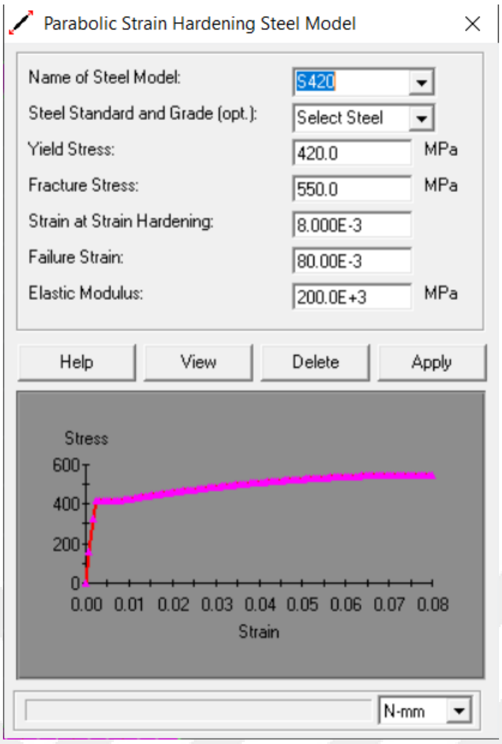
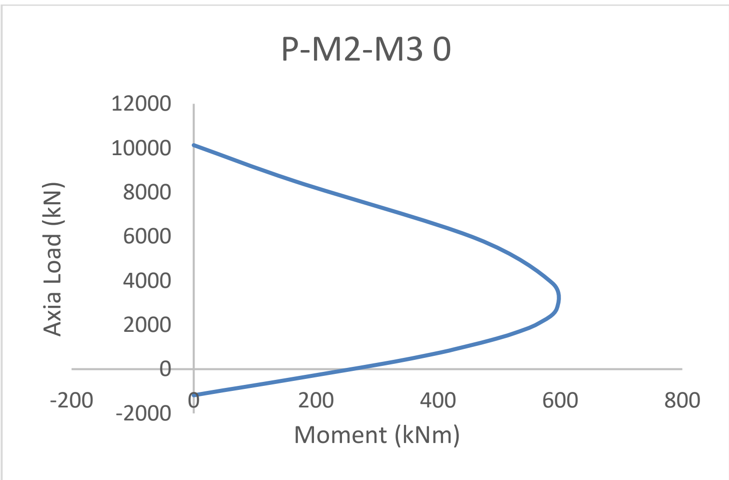


Figure 5. 22 Reinforcement steel model of C50x50 beam

In order to evaluate a three-dimensional yield surface of the column, firstly it is required to obtain the two-dimensional interaction diagrams. Between the M2 and M3 surface it is enough to calculate three different diagrams at 0, 45 and 90 degrees. In figure 5.23 there is presented the interaction diagram for 50x50 column.



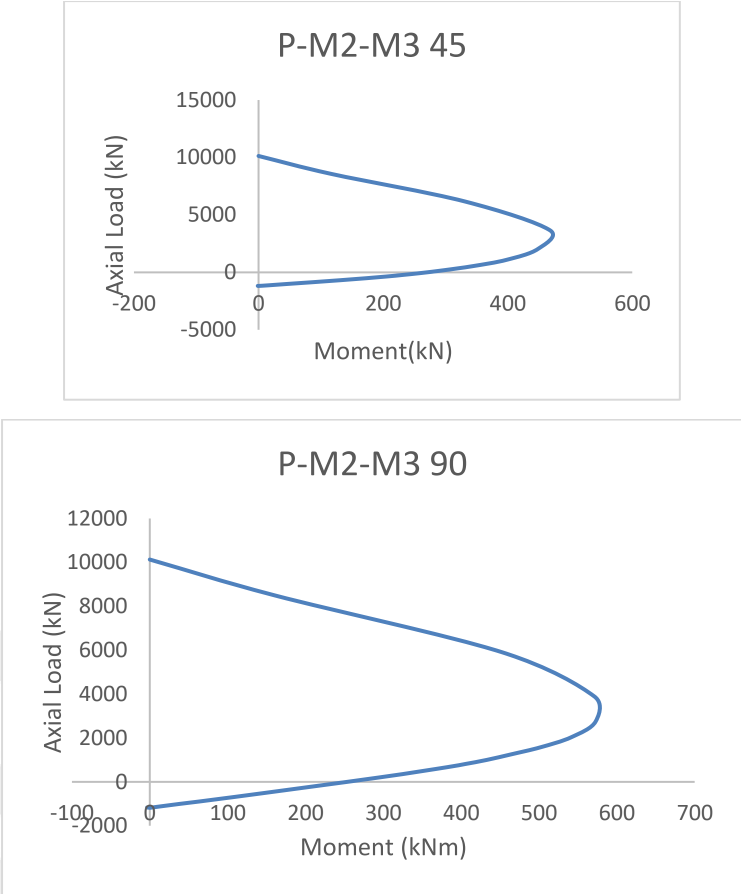


Figure 5. 23 P-M2-M3 relationship of beam B50/40

The same calculation is done for every column.

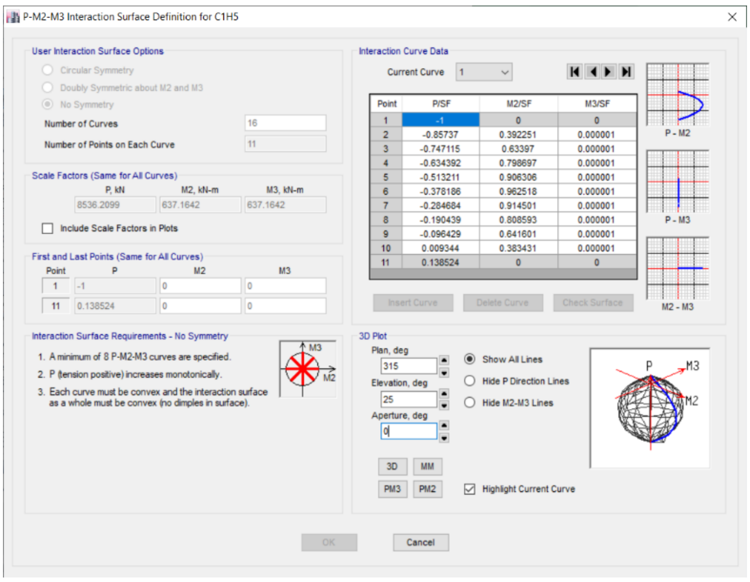


Figure 5. 24 Axial Load-Moment interaction graphics for 0

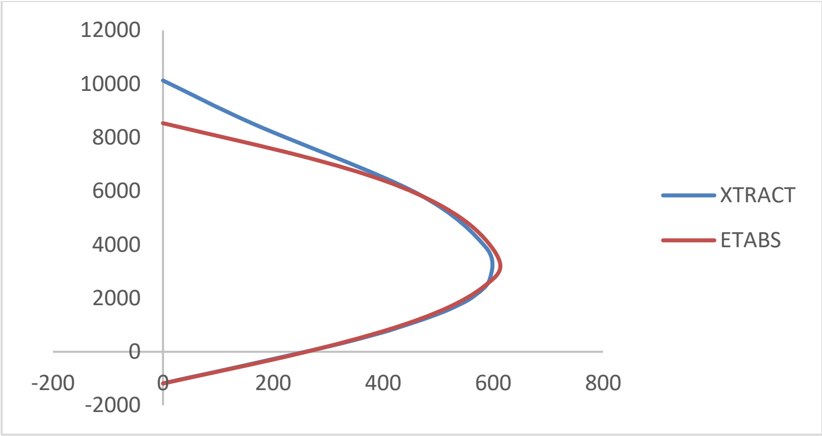


Figure 5. 25 Comparation of Axial Load-Moment interaction graphics

From the calculations and graphics shown in tables and figure 5.25 above it is concluded that the axial load-moment curvature calculation that were automatically assigned by ETABS are acceptable.

After the calculation it is required to obtain the moment-curvature diagram of the columns by loading P1 and P2. P1 and P2 loads are obtained by calculating the P load. P1 represents 45% of P, while P2 is 15% of P. P is the axial bearing capacity of the column. In this case it is calculated as follows:

$P = f_{cd} \times A = 35N/mm^2 * 500mm * 500mm = 8750$ kN

$P_1 = 8750kN \times 0{,}45 = 3900$ kN

$P_2 = 8750kN \times 0{,}15 = 1312$ kN

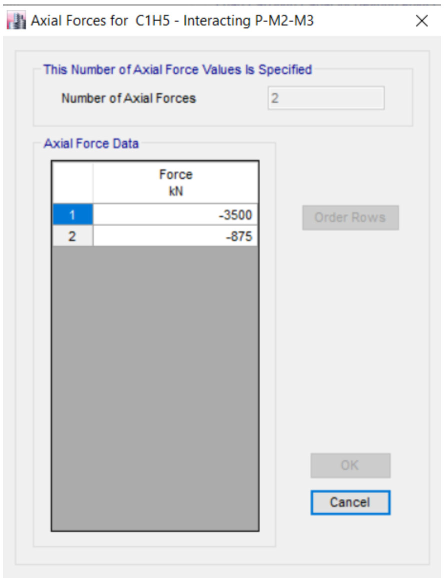


Figure 5. 26 Axial Loads assumed by ETABS

When the calculations are compared it can be concluded that ETABS used the 10% rule for P_2. The assignment of ETABS are accepted.

Table 5. 7 Moment-curvature relations

Column	Load	Dir.	My kNm	Mu kNm	Mu/My	Eff. Yield Curv Rad/m	Ultim. Curv Rad/m	Eff. Yield Rot Rad	Ultim. Rot Rad
50x50	P1=1312	0	515	476	0.923	0.00931	0.0411	0.0019	0.008
	P1=1312	45	506	492	0.972	0.00792	0.0208	0.0016	0.004
	P1=1312	90	524	493	0.940	0.00904	0.0419	0.0018	0.008
	P2=3900	0	750	683	0.911	0.00814	0.0198	0.0016	0.004
	P2=3900	45	608	654	1.075	0.00563	0.0138	0.0011	0.003
	P2=3900	90	774	701	0.906	0.00816	0.0199	0.0016	0.004

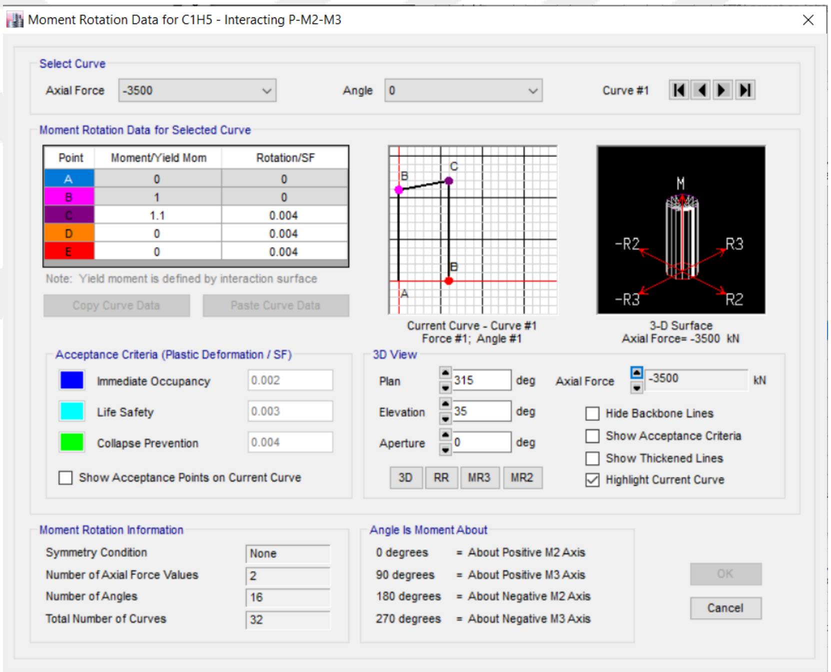


Figure 5. 27 Moment Curvature values according to ETABS

The value of the ultimate rotation for C50/50 at P_2 load in the calculations shown in table 5.7 are the same values as assigned by ETABS. We can read the 0,004 rad value of rotation in ETABS.

To sum up, after three consecutive controls, P_1 and P_2 loads, Axial Load Moment interactions and Moment Curvature relations, it is stated that assignments made by ETABS are correct.

5.3.5 Defining the nonlinear shear walls sections

In order to define nonlinear behavior of shear walls that are placed in both directions in the structure it is necessary to define their materials as non-linear. Firstly, the properties of the data were defined. In every shear wall section unconfined concrete and reinforcement steel show the same behavior, while confined concrete properties depend on the distribution of reinforcement on the section. Therefore, in figure 5.28 C35 unconfined concrete model and reinforcement steel model and figure 5.29 confined concrete calculated for shear wall 260/30 are shown as examples.

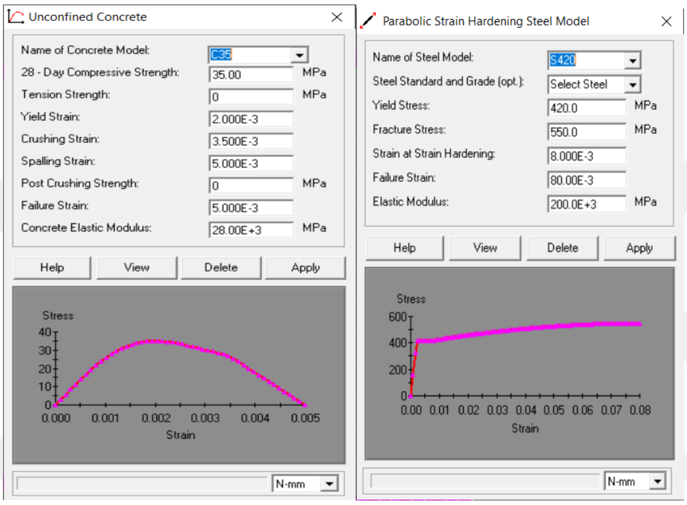


Figure 5. 28 Shear Walls unconfined concrete and steel model

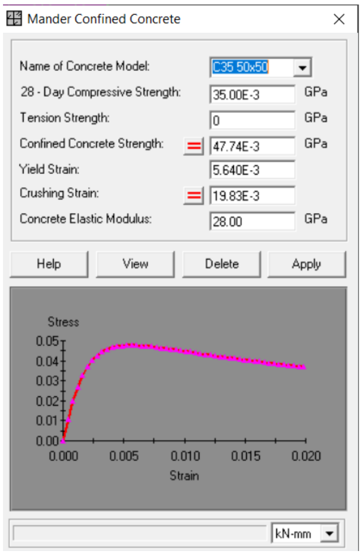


Figure 5. 29 W250/30 Confined Concrete model

In order to assign the plastic hinge as a distributed hinge it is necesserally to divide the section in fibers and using coordinates to define the reinforcement, confined and unconfined concrete fibers. In figure 5.30, W2 sections fiber elements are defined.

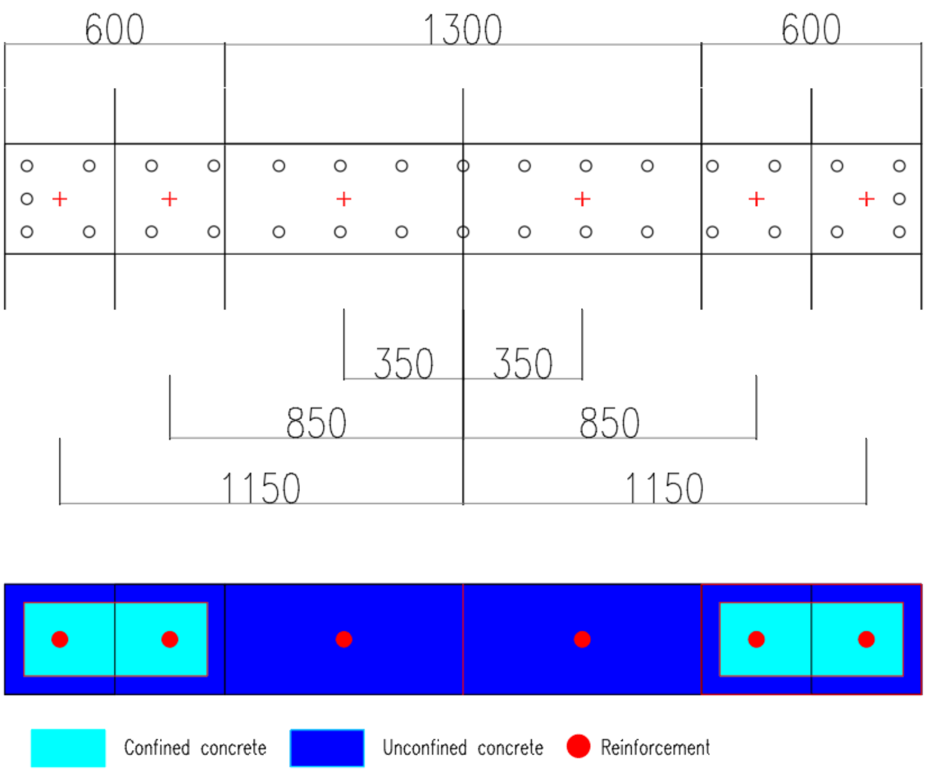


Figure 5. 30 Fiber model of shear wall section

The same procedure is repeated for other shear-wall sections. Material properties and fiber coordinates that are defined above are used to create the distributed plastic hinge in ETABS. The hinge type selection is presented in figure 5.31

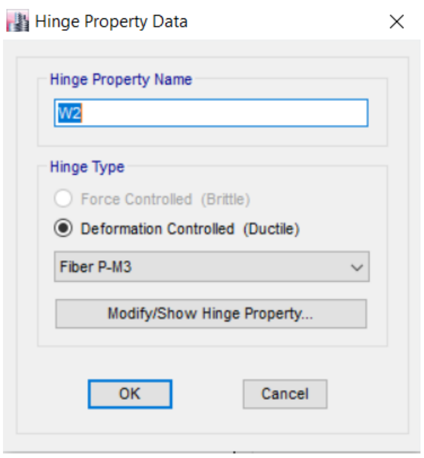


Figure 5. 31 Hinge Type selection for W2 (P-M3)

After defining the hinge type, fiber model characteristics are imputed in ETABS as shown in figure 5.32.

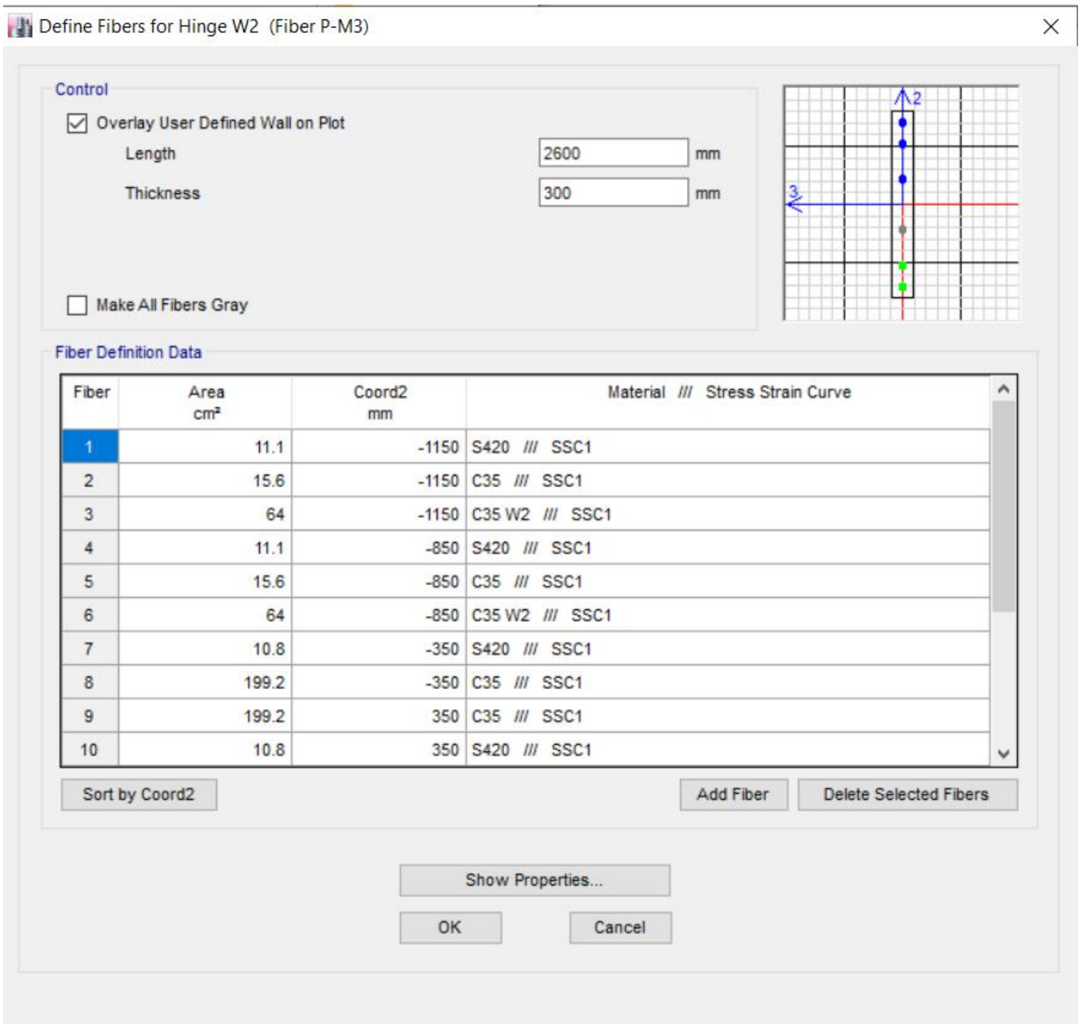


Figure 5. 32 Defining the fiber prpğerties of hinge section

5.3.6 Etabs data input required for time-history analysis

Firstly, the earthquake data which were scaled using the Seismomatch program are needed. The scaled data are input in Etabs. Before the input we have to check the units used in Seismomatch and units used in Etabs, in order to provide correct results. Especialy we have to check the acceleration unit and change it to m/s2. As an example, the graphic and process of defining of the Duzce_Sakarya_EW and Duzce_Sakarya_NS data is explained below. By following the Define>Functions>Time-History commands and after that From File>Add New

Function we manage to browse the scaled data and therefore we define it on ETABS. The mentioned data are shown in Figure 5.33

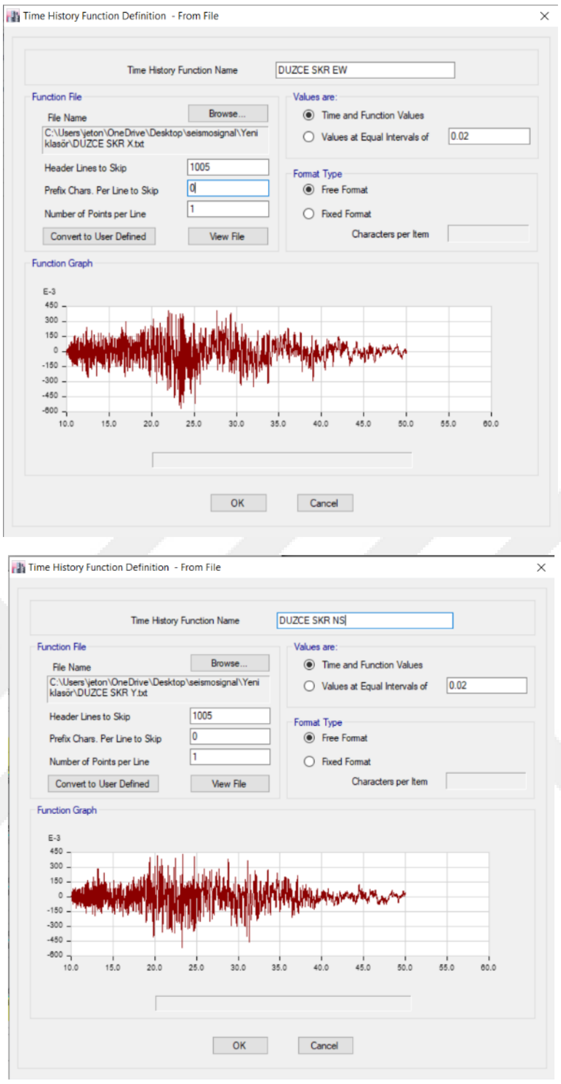


Figure 5. 33 Defining the earthquake data as functions in ETABS for both directions

As it is seen from the figures above, in order to read the text data in proper manner it is defined that first five lines of the text files do not contain acceleration data and there is only one acceleration value per line. Every text file belonging to 11 earthquake data is defined by following this way. After that, the load cases are defined. In order to get correct results, it is important how the seismic loads are defined when it comes to relations with vertical, live and dead loads. So, firstly we must define the vertical load by following Define>Load Case as shown in figure 5.34. It is required to define a constant function that will multiply the dead and live load for every time interval of time history. In this assignment it is also required to add the vertical seismic load to the initial load as required in TBDY2018 .

Seismic loads are defined after defining the initial load. It is assumed that seismic load continues at the end point of initial load in order to include the mass load of the structure. TBDY2018 in chapter 5.2.2.1 combination of seismic and vertical loads is defined as figure 5.34

The assignment of vertical load is shown in figure 5.34 while the seismic load for Duzce-Sakarya earthquake example is shown in figure 5.35.

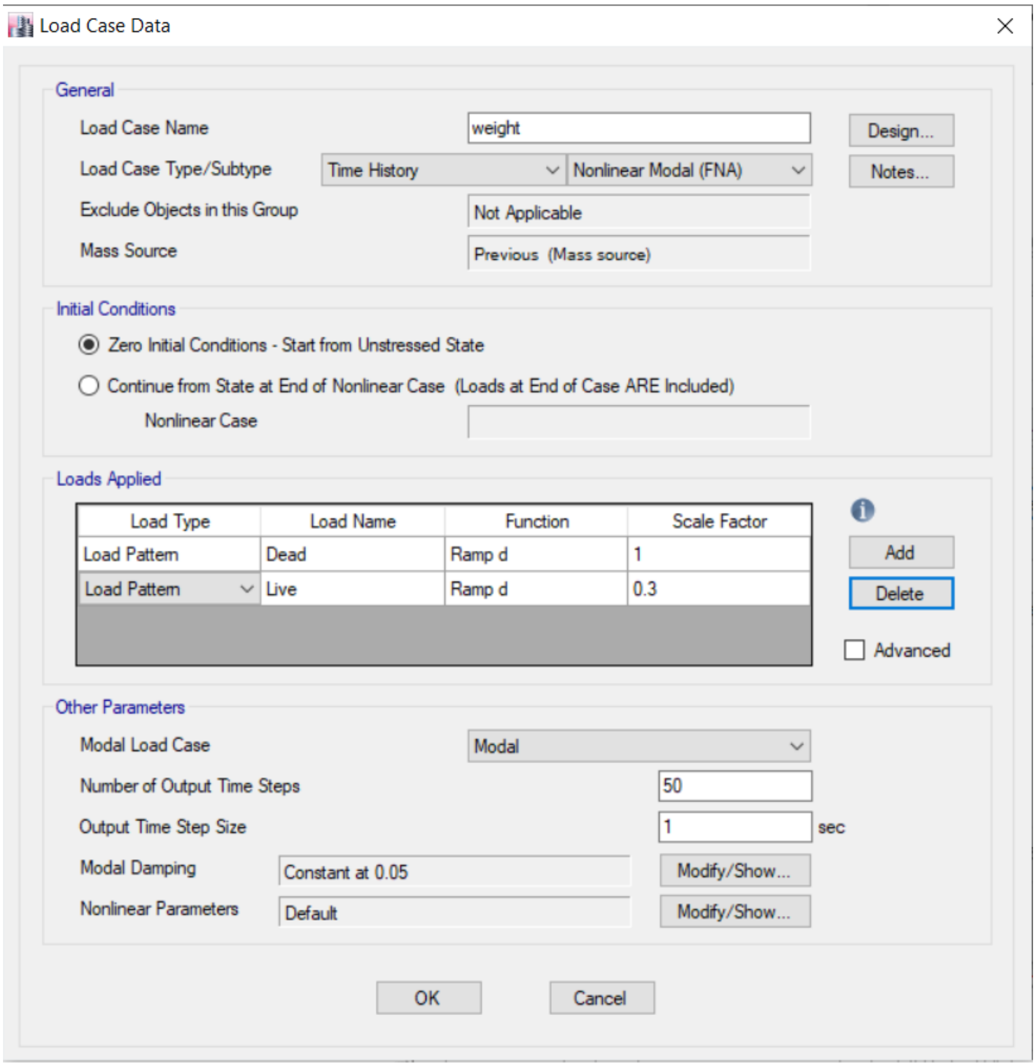


Figure 5. 34 Non-linear Vertical Load Case

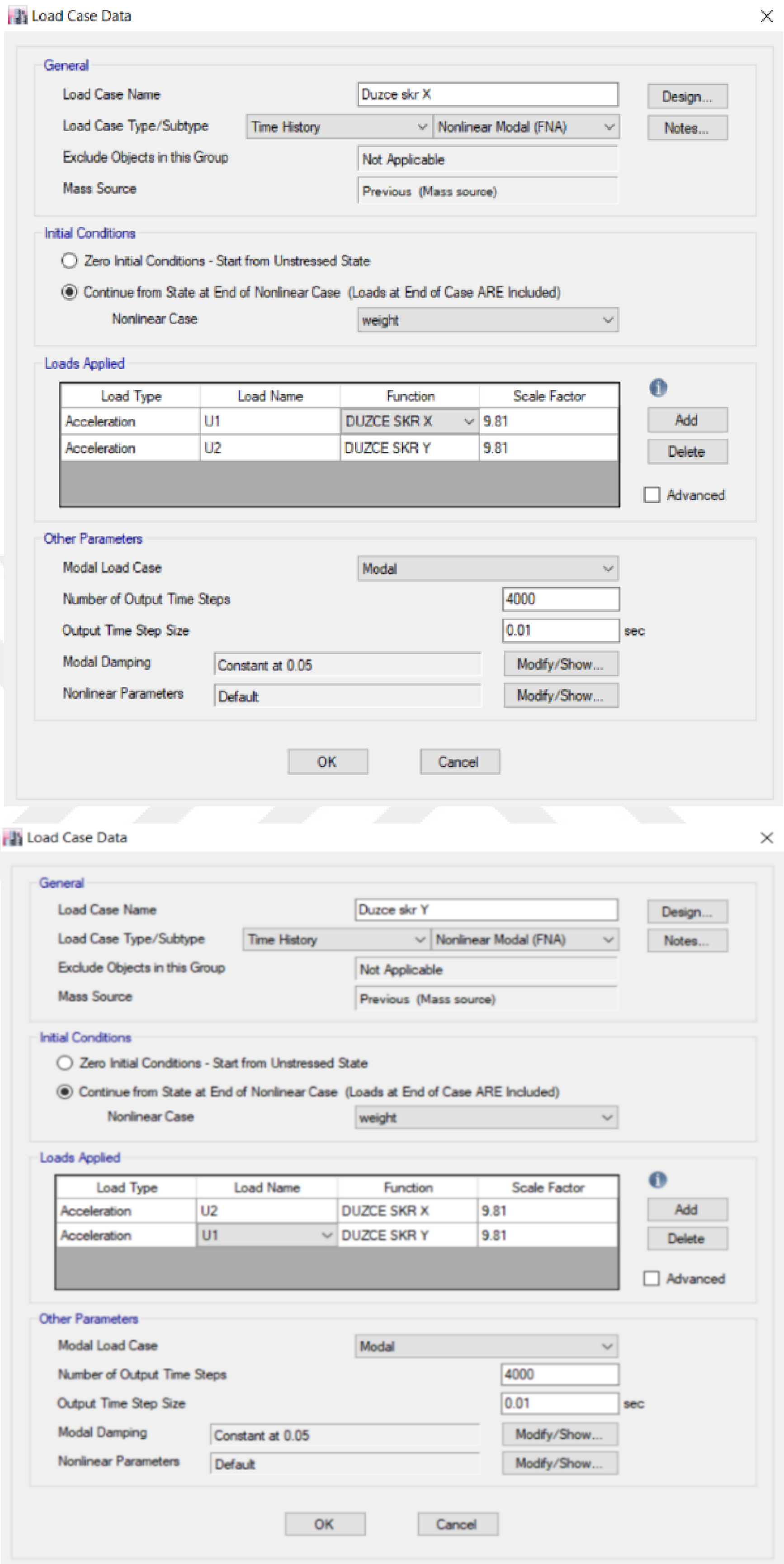


Figure 5. 35 Time-History Loads for both directions

5.4 Evaluation of the Results of Time History Non-linear Analysis of the High-rise Structure

The following chapter deals with the results and reaction of high-rise structure to 11 different earthquakes, which effecting in two directions make 22 different loads to calculate. The results are divided in two types: based on elements and structure.

5.4.1 Results based on structure

In non-linear analysis, to have an idea about the behavior of the structure before checking the elements separately it is preferred to have a look at the behavior of the systems, although it is very hard to come up with performance results. The following graphics show the story displacements and hysteric curves which contain shear loads and top storey displacements for every seismic load. In order to decrease the effect of P-Delta effect on the structure seismic codes require a limit value of storey drift allowed for each seismic load direction.

$$\boldsymbol{\lambda \frac{\delta_{i\,max}}{h_i} \leq 0.008}\ \kappa \qquad \textbf{(5.5)}$$

The ratio between the calculated elastic design spectrum acceleration according to DD3 and calculated elastic design spectrum acceleration according to DD2 is defined as λ. The K value is assumed as 0.5 for steel structures and 1 for RC structures. Using this equation for this structure the limit values for X direction is 0.008 while for Y direction is 0,008. From the graphics bellow we conclude that for every earthquake they remain under limit values.

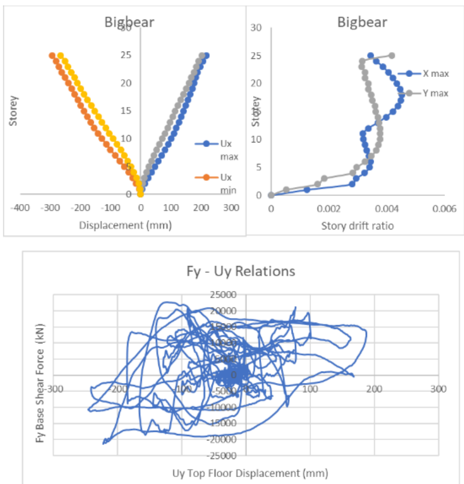


Figure 5. 36 General results of Bigbear Earthquake Loads

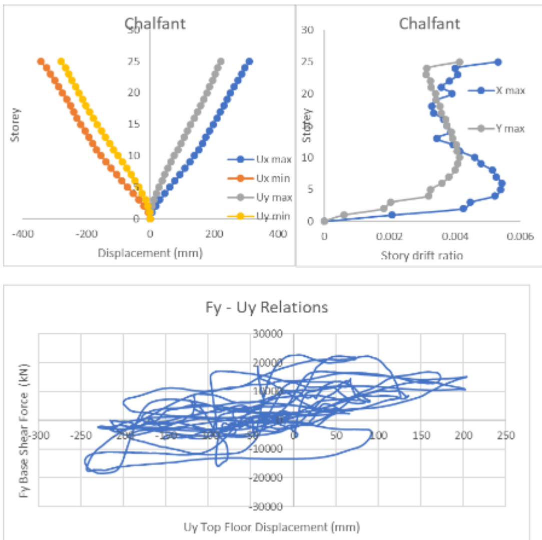


Figure 5. 37 General results of Chalfant Earthquake Loads

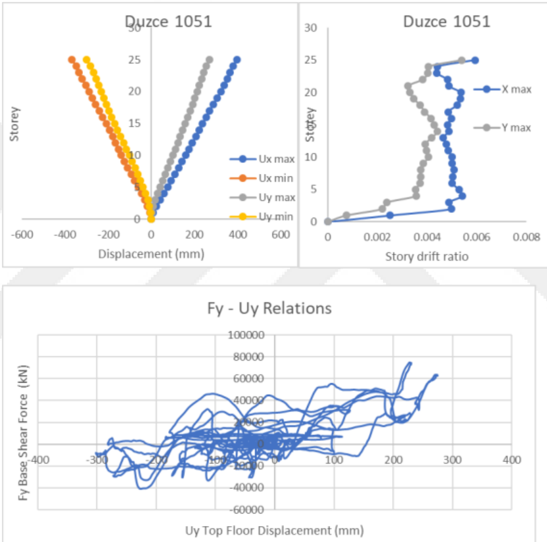


Figure 5. 38 General results of Duzce 1051 Earthquake Loads

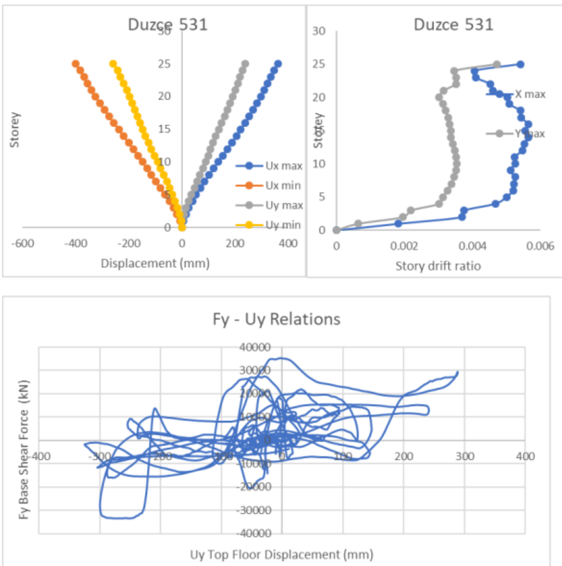


Figure 5. 39 General results of Duzce 531 Earthquake Loads

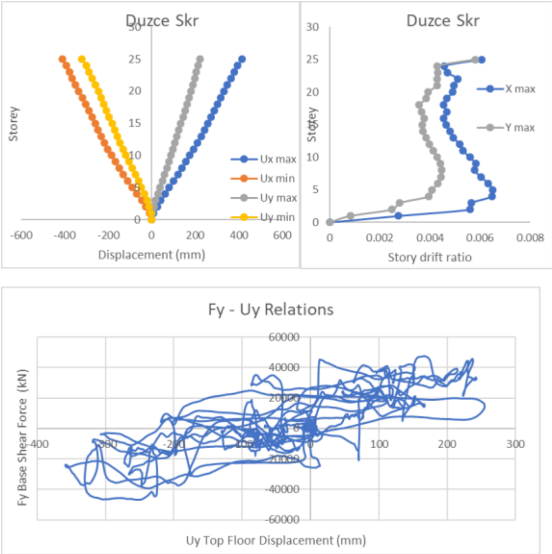


Figure 5. 40 General results of Duzce Skr Earthquake Loads

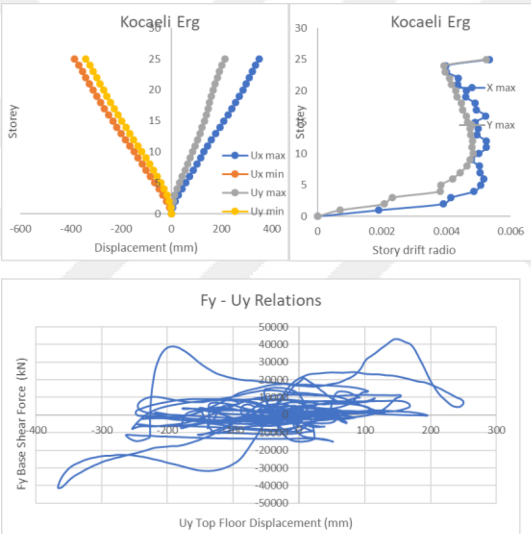


Figure 5. 41 General results of Kocaeli Erg Earthquake Loads

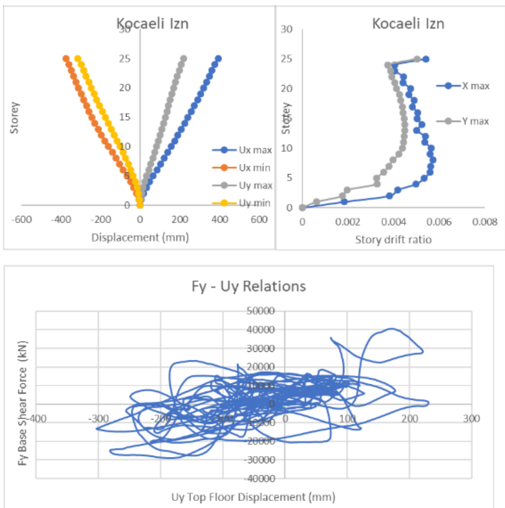


Figure 5. 42 General results of Kocaeli Izn Earthquake Loads

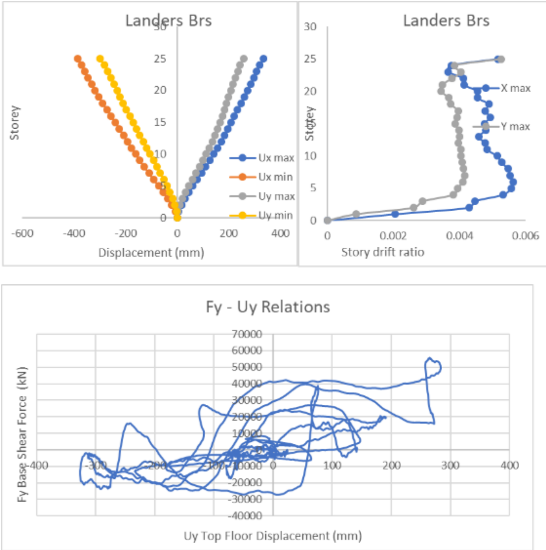


Figure 5. 43 General results of Landers Brs Earthquake Loads

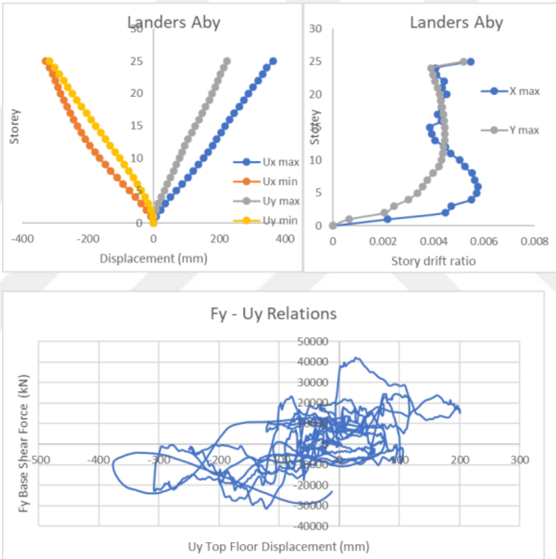


Figure 5. 44 General results of Landers Aby Earthquake Loads

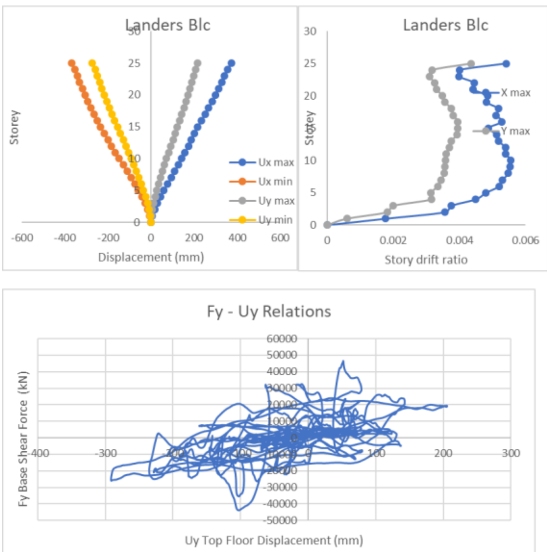


Figure 5. 45 General results of Landers Blc Earthquake Loads

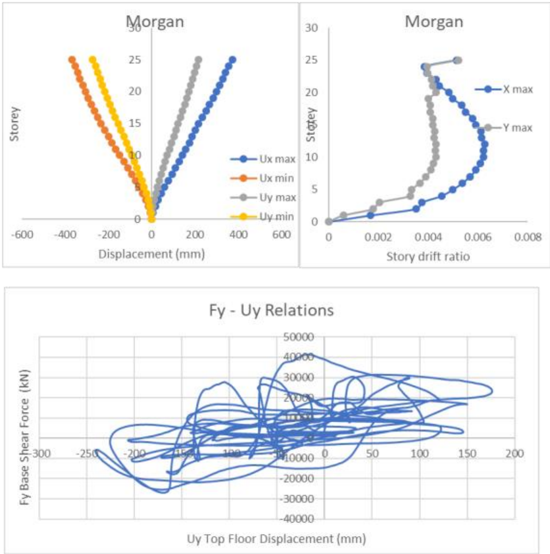


Figure 5. 46 General results of Morgan Earthquake Loads

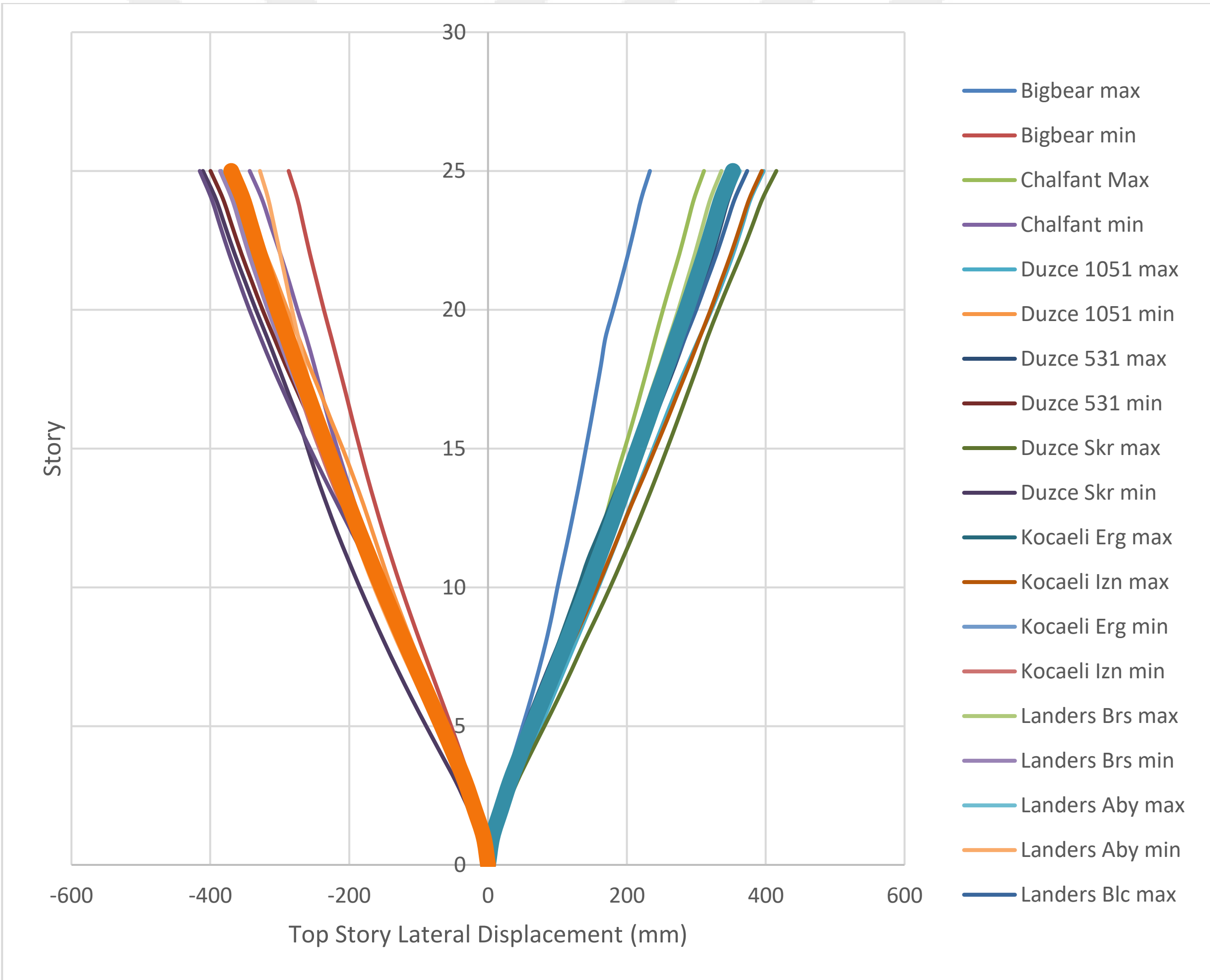


Figure 5. 47 Top Story Lateral Displacement for every earthquake (minimum and maximum)

Table 5. 8 Largest absolute values of displacements and shear forces in both directions

	Ux	Uy	Vx	Vy
Bigbear	293.405	265.3355	45144.85	56022.22
Chalfant	343.2595	279.7435	40148.65	62972.79
Duzce1051	397.0955	299.3125	77701.44	74493.28
Duzce531	399.7195	257.971	42538.73	48552.68
DuzceSkr	415.343	320.2975	84526.13	73741.75
KocaeliErg	384.3425	339.706	43200.18	68952.53
KocaeliErg	394.3825	314.3135	43491.6	57248.17
LandersBrs	385.94	299.156	39038.08	55091.88
LandersAby	362.921	317.35	58194.23	55998.65
LandersBlc	373.3075	273.2925	77705.36	60016.24
Morgan	415.189	311.85	33269.7	41793.01
Max	415.343	339.706	84526.13	74493.28

Hysteric graphics that contains shear force and displacement give us a brief idea about the structure, in this case elastic behavior of the structure. Structures largest shear force values in both directions come from Duzce earthquake, X direction 84526kN while Y direction 74493kN. As for top floor lateral displacement largest value of X direction 415mm while Y direction 339mm. The fact that reactions of every data are similar values shows that earthquake data was properly chosen.

5.4.2 Results based on elements

If we compare two types of results, due to the fact that every element is separately calculated therefore every damage level of elements is obtained, we can easily conclude that results on the basis of elements are more comprehensive results. Although, for high rise structure the obtaining of performance of every single element is a hard and very long process, TBDY 2018 has layed it down as a condition. Therefore, the following chapter has three parts for three different elements: beams, columns and shear walls. Calculation examples are also shown for every element mentioned below

5.4.2.1 Beams nonlinear behavior results

The plastic hinges defined in both sides of the beams provide us the results of non-linear behavior of the beams. The results are obtained and as an outcome we gain the values of plastic rotations. When the value of plastic rotation is divided by the length of plastic hinge, we gain the value of plastic curvature. Effective yield curvature values are taken from XTRACT calculation results. The sum of plastic curvature and effective yield curvature value gives the total curvature value of curvature for that beam. This value can be found within the values of curvature of that beam. There are also the values of concrete and steel strain which responds to the values of curvature. With the interpolation we obtain the value of concrete and steel strain which responses to total curvature value of the beam. At the end by comparing the strain values with the damage limits given by TBDY2018 we can obtain the damage zone for every beam. The most unfavorable damage zone of concrete or steel is considered as the beams damage zone.

The structure consists total 4436 beams and 351 columns. Estimation of damage zone for every element separately is very long process. Due to that fact by using Excel for interpolation and Visual Basic for filtering every section to the section where it belongs and obtain the damage zone the calculation is shortened.

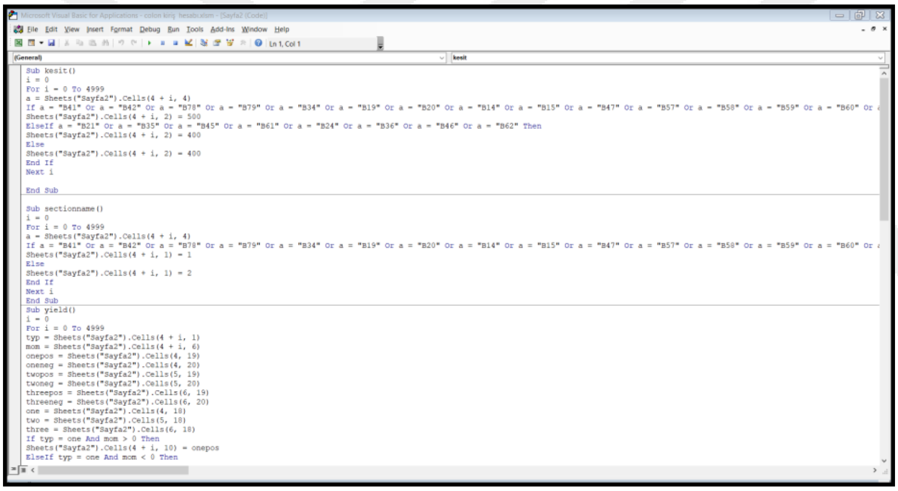


Figure 5. 48 The code used for beam calculations

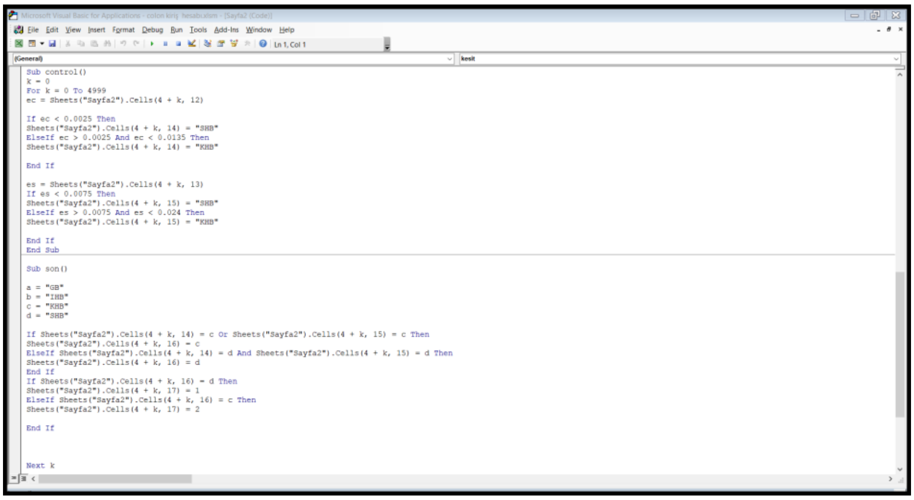
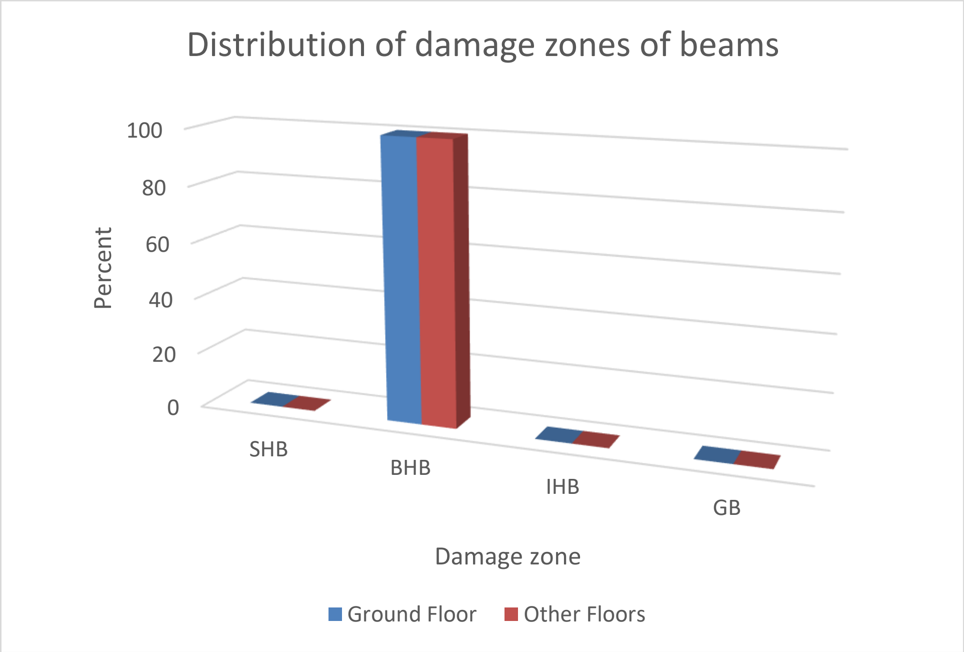


Figure 5. 49 The code used for beam calculations

The percentage of the damage zones where columns of first floor and of the entire structure belong is shown in figure 5.50 below.

Every column belongs to Significant Damage Zone (BHB). The rule in TBDY 2018 says that the structure belongs to limited hazard damage zone if not more than 20% of the beams reach the significant damage zone. In our situation the beams fulfill the condition for Limited Damage zone



	BHB	BHB	IHB	GB
Ground Floor	0	100	0	0
Other floors	0	100	0	0

Figure 5. 50 Distribution of damage zones of beams

5.4.2.2 An example of damage zone evaluation of beam

A detailed explanation of the calculation of damage zone of beam B42, found in the first floor, with a section 250/500 is shown below. Other sections results are presented in table APPENDIX A in appendix chapter.

$\theta_{p,left}$=0.00003909 rad

$\theta_{p,right}$=0.0000179 rad

In order to obtain the value of curvature the value of rotation is divided to the value of length of plastic hinge

$$\Phi_{p.left} = \frac{\theta_{p.left}}{L_p} = \frac{0,00003909}{0.25m} = 0.0002$$

$$\Phi_{p.right} = \frac{\theta_{p.right}}{L_p} = \frac{0,000179}{0.25m} = 0.00071$$

The values of effective yield curvature are calculated by XTRACT. The sum of plastic curvature and effective yield curvature gives the value of total curvature for that section.

$\Phi_t = \Phi_p + \Phi_y = 0.0002 + 0.0052 = 0.0054$

$\Phi_t = \Phi_p + \Phi_y = 0.00071 + 0.0077 = 0.0084$

Value of total curvature is proportional with the strain value of concrete and steel. As the result of section analysis made by XTRACT the values which respond to total curvature Φt = 0.0084 rad/m are calculated with interpolation as follows:

$\varepsilon c = 0.00205$

$\varepsilon s = 0.001889$

If we compare the results with damage limits

$\varepsilon_c = 0.00205 < \varepsilon_c^{(SH)} = 0.0025$

$\varepsilon_s = 0.001889 < \varepsilon_s^{(SH)} = 0.0075$

If we repeat the same process for another edge of the section

$\Phi t = 0.0084$

$\varepsilon_c = 0.002959 > \varepsilon_c^{(SH)} = 0.0025$

$\varepsilon_s = 0.002693 < \varepsilon_s^{(SH)} = 0.0075$

As it can be observed from the calculation both concrete strain values are larger than the strain values belonging to Controlled damage level. By considering the unfavorable value as a result, it means that the whole sections of beam belongs to limited damage zone.

5.4.2.3 Columns nonlinear behavior results

Similar to beams, plastic hinges defined in both sides of the column provide us the results of non-linear behavior of the column. The difference in behavior between the columns and the beams leads to two different methods of calculation performance. Due to the simultaneous effect of axial load and bending moment in two directions (P-M2-M3) column behaves differently. There are two different methods to obtain the performance of the columns: The first one is to calculate the total curvature by considering only axial load. The curvature value is proportional to strain of concrete and steel. Strain value which corresponds to total curvature will be compared with damage limit values. As the result of comparison, damage zone will be obtained. The second method uses the limit values of strain for three different levels of damage. Using these levels three different axial-load curvature graphics are defined. By calculating the total yield of the element and the axial load, the point with these two coordinates is drawn in graphic. The area where the point remains between three graphics is considered the damage zone of that section.

In order to shorten the calculation time due to the huge number of column section, calculation is done using MS Excel software.

5.4.2.4 An example of damage zone evaluation of column

A detailed explanation of the calculation of damage zone of column C4, placed in first floor, with a section 500/700 is shown below. As for the other section in EK-b there are solutions presented in a table. The principal provided in TBDY 2018 to calculate the reactions and rotation of the sections under 11 seismic loads is to take the average of the largest absolute value of every seismic load. Using this principal, bottom and top sections plastic rotation and axial loads are provided.

$\theta_{p,bottom}$=0.0000034 rad N=10451 kN

$\theta_{p,top}$=0.0000015 rad N=10369kN

The value of curvature which is also defined as unit rotation is calculated by dividing the plastic rotation to the plastic hinge length.

$$\Phi_{p.bottom} = \frac{\theta_{p.bottom}}{L_p} = \frac{0,0000034}{0.35m} = 0.000009326$$

$$\Phi_{p.top} = \frac{\theta_{p.top}}{L_p} = \frac{0,0000015}{0.35m} = 0.000006839$$

The sum of plastic curvature and effective yield curvature, which is calculated considering the axial load, gives the total curvature. Effective yield curvature is calculated using Xtract

$\Phi_{ybottom}$ = 0,0057

Φ_{ytop} = 0,0057

$\Phi_t = \Phi_p + \Phi_y$ = 0,00009326 + 0,0057 = 0,005709326

$\Phi_t = \Phi_p + \Phi_y$ = 0,000006839 + 0,0057 = 0,005706839

$\Phi_{ybottom}$ = 0,0057

Φ_{ytop} = 0,0057

The point with axial load and total curvature coordinates is compared to the Axial Load-Curvature graphics of every damage limit.

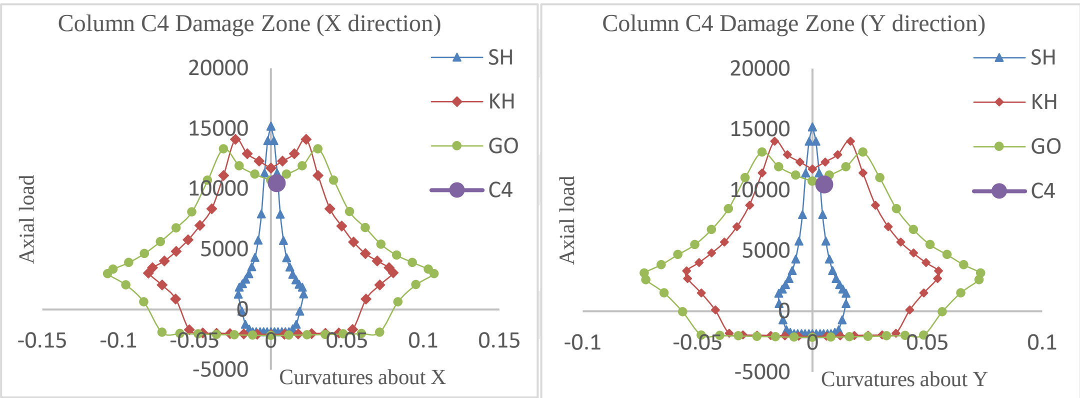


Figure 5. 51 Axial Load Curvature Diagram according to damage limits of C4

Diagram shows that both sections of the column C4 belongs to Significant Damage Zone. In situation when different directions of the column show different performance, we can declare that the common result is the most unfavourable one, in this situation BHB.

5.4.2.5 An example of damage zone evaluation of column according to EC8

$\theta_{p,bottom}$=0.0000034 rad N=10451 kN

$\theta_{p,top}$=0.0000015 rad N=10369kN

$$\Phi_{p.bottom} = \frac{\theta_{p.bottom}}{L_p} = \frac{0,0000034}{0.35m} = 0.000009326$$

$$\Phi_{p.top} = \frac{\theta_{p.top}}{L_p} = \frac{0,0000015}{0.35m} = 0.000006839$$

$$\underbrace{\theta_y = \varphi_y \frac{L_v + \alpha_v z}{3}}_{\text{Flexural}} + \underbrace{0.0013\left(1 + 1.5\frac{h}{L_s}\right)}_{\text{Shear deformation}} + \underbrace{\left(\frac{0.13\varphi_y d_{bl} f_y}{\sqrt{f_c}}\right)}_{\text{Anchorage slip of bars}}$$

θ_y=0,0005 rad

θ_e=0,001995

Table 5. 9 Compliance Criteria for assessment or retrofitting of concrete members (Total Chord rotation limitations) (EC8)

Member	Limited Damage (LD)	Significant Damage (SD)	Near Collapse (NC)	
			Linear analysis	Non-linear analysis
Ductile primary	$\theta_E^{(1)} \leq \theta_y^{(2)}$	$\theta_E^{(1)} \leq 0.75\theta_{u,m-\sigma}^{(3)}$	$\theta_E^{(1)} \leq \theta_{u,m-\sigma}^{(3)}$	
Ductile secondary		$\theta_E^{(1)} \leq 0.75\theta_{u,m}^{(4)}$	$\theta_E^{(1)} \leq \theta_{um}^{(4)}$	

According to the calculations and the criteria mentioned above it is concluded that according to the EC8 column belongs to the limited damage zone.

5.4.2.6 An example of damage zone evaluation of a shear wall

Performance evaluation of shear walls sections, done using the effect of 11 earthquake data pairs, is done by assigning the plastic hinge to the walls within the critical height which is defined in TBDY2018. Unlikely from column and beams, shear walls are defined as layered sections and by defining the nonlinear property of material non linear behavior of structure is assumed. Wall sections**,** same as columns**,** are affected by axial and bending moment loads. The estimation of walls damage zone is done by comparing strain value of every fiber element to the limit strain values of every damage zone. In the first story most of the wall sections belong to the Controled damage zone while in other floors every section belongs to Limited damage zone. In order to show a calculation example, the result of W2 shear wall are shown in table 5.9.

Table 5. 10 W2 Shear Walls fiber elements shear and stress values

Story	Wall	Hinge	No	Coordinate	Material	Stress	Strain	Damage Zone	
				mm		MPa		FIBER	Section
Story1	W2	W2	1	-1150	S420	422.13	0.002784	KHB	KHB
Story1	W2	W2	2	-1150	C35	35	0.002784	KHB	
Story1	W2	W2	3	-1150	C35 W2	34.99	0.002784	KHB	
Story1	W2	W2	4	-850	S420	420.69	0.002323	SHB	
Story1	W2	W2	5	-850	C35	35	0.002323	SHB	
Story1	W2	W2	6	-850	C35 W2	34.97	0.002323	SHB	
Story1	W2	W2	7	-350	S420	315.67	0.001578	SHB	
Story1	W2	W2	8	-350	C35	33.37	0.001578	SHB	
Story1	W2	W2	9	350	C35	34.49	0.001869	SHB	
Story1	W2	W2	10	350	S420	373.87	0.001869	SHB	
Story1	W2	W2	11	850	C35 W2	34.91	0.002673	KHB	
Story1	W2	W2	12	850	C35	34.99	0.002673	KHB	
Story1	W2	W2	13	850	S420	421.79	0.002673	KHB	
Story1	W2	W2	14	1150	C35 W2	35	0.003155	KHB	
Story1	W2	W2	15	1150	C35	35	0.003155	KHB	
Story1	W2	W2	16	1150	S420	423.29	0.003155	KHB	

When the fiber elements strain values are compared to the limit values of damage zones it is considered that the W2 Shear wall belongs to the Significant Damage Zone. Although some of the fiber elements belong to Limited Damage Zone the common result of the whole section is the most unfavourable values, in this case Significant Damage Zone. Other sections damage zone distribution is shown in figure 5.52

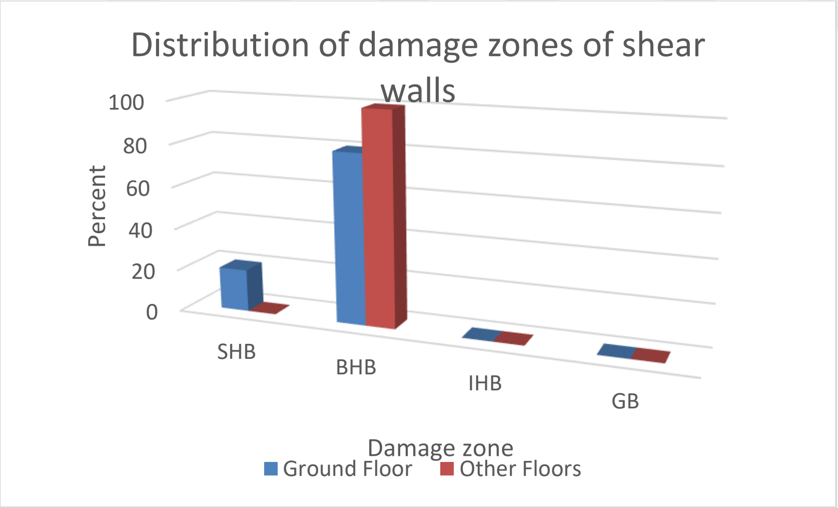


Figure 5. 52 Distribution of damage zones of Shear Walls

6. RESULTS, CONCLUSIONS AND PROPOSAL

6.1 Results

The purpose of this thesis research was to evaluate the performance analysis of 24 story, 84m high-rise structure, projected according to the DBYBHY2007 seismic code, using the non-linear time-history analysis according to TBDY2018 and EC8. First part of the thesis as an introduction gives a brief information about the research. Second chapter explains in details the performance analysis with every step needed to evaluate the performance analysis. As for the third chapter, it discusses the seismic loads required to perform the analysis starting from response spectrum definition, to continue with required strong ground data and the matching process. Forth part is about the EC8 rules needed for time history analysis. In the last chapter of the thesis there is studied the example structure and its performance and as a result the performance of the structural elements is obtained.

Hence the project of the structure is available and it was properly constructed, it is considered that knowledge level is advanced knowledge level and the information coefficient level is assumed as one. Therefore, properties of the materials remained unchanged.

The nonlinear behavior of structure was defined by assigning plastic hinges to beams, columns and shear walls. The results of plastic hinge reactions are used to determine the performance of the whole structure by using the unfavorable result.

It is important to mention the fact that the structures mathematical model was created by neglecting the relations between the structure and the soil. Rigit joints were assigned to the column bottom edges.

Time history analysis was executed using 11 different earthquake data that were properly chosen considering earthquake properties of structure zone. The result of the analysis is the average of the largest absolute value of every result. If more than 11 earthquake data are used it contributes to more realistic results.

When the analysis is performed the results in structure basis are compared to each other. If the results do not show an explicit difference to each other and their values are similar it means that proper data was chosen and chosen data were properly matched. In this analysis the largest absolute values are observed from Morgan earthquake, for X direction load we get the highest value Ux=415mm while for Y direction lateral displacement is calculated as Uy=311mm.Also, relative floor drift values are calculated and checked.

After summing up the results basing on elements the damage zones are distributed as follows:

- Beams: As shown in graphic in chapter 5, it is concluded that every beam of every story belongs to the Significant Damage Zone. When the conditions of damage zones are checked it is concluded that, in terms of beams, structure belongs to the Limited Damage Zone (BHB).
- Columns: When every Axial Load- Curvature graphic is summed up it is seen that most of the columns remains in Limited Damage Zone. When the conditions of damage zones are checked it is concluded that when it comes to columns structure belongs to the Limited Damage Zone (BHB).
- Shear-Walls: Fiber States results of every wall gives the strain and values of both concrete and reinforced steel. Result showed that in the first floor 80% of the shear walls belong to the Controled damage zone, while 20% of the remain in Limited damage zone. Therefore, when the conditions of damage zones are checked it is concluded that, in terms of shear walls, structure belongs to the Controled Damage Zone (BHB).

When three elements results are collected it is declared that the structure belongs to the Significant Damage Zone and it fulfills the conditions of DD-2 seismic load level.

As for the EC8 the results were compared for one column section. The damage observed in that column when compared to the result according to TBDY2018 has shown that EC8 and TBDY2018 give very similar damage results.

6.2 Proposal

As it is commonly known time history analysis calculates the equation of motions for every time increment, thereby stiffness matrix changes for every time step and this requires enormous drive memory and reduces the possibility to include the P-Delta effect therefore the additional moments are not included.

Similar situation is for Wall elements meshing. Size of finite elements were assumed as 1m, if it was lower value the result could be more realistic.

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APPENDICES

APPENDIX A: Ground Floor Beams Damage Zones

APPENDIX B: Ground Floor Columns Damage Zones

APPENDIX C: Ground Floor Shear Walls Damage Zones

Appendix A

Table A. 1 Damage zones of beams X-Direction

Beam	Beam Section	Position	M3 (kNm)	R3 plastic hinge (rad)	Lp (m)	φp	φy (m)	φt (m)	εc	εs	Concrete Damage	Steel Damage		
B2	500/400	0	7.799	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B2	500/400	1	39.325	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B2	500/400	0	-51.195	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B2	500/400	1	-79.623	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B7	500/400	0	0.773	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B7	500/400	1	-0.802	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B7	500/400	0	-0.083	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B7	500/400	1	-33.468	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B8	500/400	0	1.012	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B8	500/400	1	5.603	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B8	500/400	0	-0.598	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B8	500/400	1	-31.047	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B9	500/400	0	1.049	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B9	500/400	1	7.796	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B9	500/400	0	-0.688	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B9	500/400	1	-32.876	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B10	500/400	0	1.476	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B10	500/400	1	-2.404	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B10	500/400	0	-0.750	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B10	500/400	1	-21.693	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B11	500/400	0	33.558	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B11	500/400	1	47.617	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B11	500/400	0	-105.911	-9.09091E-07	0.2	-4.5E-06	0.00914	0.009145	0.003225	0.002936	KHB	SHB	KHB	
B11	500/400	1	-91.417	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams X-Direction

B13	500/400	0	34.830	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B13	500/400	1	44.330	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B13	500/400	0	-102.211	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B13	500/400	1	-100.867	2.45455E-06	0.2	1.23E-05	0.00914	0.009152	0.004087	0.003806	KHB	SHB	KHB	
B14	250/500	0	89.671	9.86818E-05	0.25	0.000395	0.00516	0.005555	0.002144	0.001974	SHB	SHB	SHB	**KHB**
B14	250/500	1	89.731	0.000248955	0.25	0.000996	0.00516	0.006156	0.002388	0.0022	SHB	SHB	SHB	
B14	250/500	0	-117.133	-7.82273E-05	0.25	-0.00031	0.0077	0.008013	0.003551	0.003305	KHB	SHB	KHB	
B14	250/500	1	-117.217	-0.000213409	0.25	-0.00085	0.0077	0.008554	0.003805	0.003542	KHB	SHB	KHB	
B15	250/500	0	89.724	0.000233318	0.25	0.000933	0.00516	0.006093	0.00236	0.002174	SHB	SHB	SHB	**KHB**
B15	250/500	1	89.674	9.13182E-05	0.25	0.000365	0.00516	0.005525	0.002132	0.001963	SHB	SHB	SHB	
B15	250/500	0	-117.217	-0.000241	0.25	-0.00096	0.0077	0.008664	0.003857	0.00359	KHB	SHB	KHB	
B15	250/500	1	-117.133	-5.89091E-05	0.25	-0.00024	0.0077	0.007936	0.002785	0.002533	KHB	SHB	KHB	
B16	500/400	0	42.176	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B16	500/400	1	45.629	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B16	500/400	0	-108.443	-9.54545E-07	0.2	-4.8E-06	0.00914	0.009145	0.003226	0.002936	KHB	SHB	KHB	
B16	500/400	1	-100.688	-5.90909E-07	0.2	-3E-06	0.00914	0.009143	0.003225	0.002936	KHB	SHB	KHB	
B18	500/400	0	-13.858	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B18	500/400	1	0.506	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B18	500/400	0	-28.677	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B18	500/400	1	-0.628	0	0.2	0	0.00914	0.00914	0.004081	0.0038	KHB	SHB	KHB	
B19	250/500	0	56.576	0.0000015	0.25	0.000006	0.00516	0.005166	0.001994	0.001836	SHB	SHB	SHB	**KHB**
B19	250/500	1	68.707	1.81818E-07	0.25	7.27E-07	0.00516	0.005161	0.001992	0.001834	SHB	SHB	SHB	
B19	250/500	0	-70.084	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B19	250/500	1	-93.175	-0.000004	0.25	-1.6E-05	0.0077	0.007716	0.003413	0.003176	KHB	SHB	KHB	
B20	250/500	0	68.073	4.77273E-06	0.25	1.91E-05	0.00516	0.005179	0.001999	0.001841	SHB	SHB	SHB	**KHB**
B20	250/500	1	76.596	5.13636E-06	0.25	2.05E-05	0.00516	0.005181	0.002	0.001841	SHB	SHB	SHB	
B20	250/500	0	-83.963	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B20	250/500	1	-99.964	-0.0000095	0.25	-3.8E-05	0.0077	0.007738	0.002713	0.002467	KHB	SHB	KHB	

Table A.1 (cont.) Damage zones of beams X-Direction

B21	500/400	0	3.829	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B21	500/400	1	1.262	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B21	500/400	0	-48.789	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B21	500/400	1	-48.942	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B22	500/400	0	29.438	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B22	500/400	1	22.264	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B22	500/400	0	-59.611	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B22	500/400	1	-34.826	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B23	500/400	0	34.866	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B23	500/400	1	30.272	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B23	500/400	0	-65.591	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B23	500/400	1	-42.020	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B24	500/400	0	37.590	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B24	500/400	1	38.952	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B24	500/400	0	-69.507	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B24	500/400	1	-70.049	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B26	500/400	0	25.901	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B26	500/400	1	44.000	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B26	500/400	0	-118.262	-6.81818E-06	0.2	-3.4E-05	0.00914	0.009174	0.003236	0.002946	KHB	SHB	KHB	
B26	500/400	1	-101.648	-9.09091E-07	0.2	-4.5E-06	0.00914	0.009145	0.003225	0.002936	KHB	SHB	KHB	
B28	500/400	0	23.914	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B28	500/400	1	15.738	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B28	500/400	0	-111.605	-0.0000015	0.2	-7.5E-06	0.00914	0.009148	0.003227	0.002937	KHB	SHB	KHB	
B28	500/400	1	-108.535	1.18182E-06	0.2	5.91E-06	0.00914	0.009146	0.003226	0.002937	KHB	SHB	KHB	
B29	500/400	0	87.122	0.000082	0.2	0.00041	0.005776	0.006186	0.00191	0.001715	SHB	SHB	SHB	**KHB**
B29	500/400	1	87.139	0.000124591	0.2	0.000623	0.005776	0.006399	0.001976	0.001774	SHB	SHB	SHB	
B29	500/400	0	-131.815	-4.40455E-05	0.2	-0.00022	0.00914	0.00936	0.003304	0.003008	KHB	SHB	KHB	
B29	500/400	1	-132.812	-7.39545E-05	0.2	-0.00037	0.00914	0.00951	0.003359	0.003058	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams X-Direction

B30	500/400	0	87.135	0.000114318	0.2	0.000572	0.005776	0.006348	0.00196	0.00176	SHB	SHB	SHB	**KHB**
B30	500/400	1	87.123	7.72273E-05	0.2	0.000386	0.005776	0.006162	0.001903	0.001709	SHB	SHB	SHB	
B30	500/400	0	-132.810	-8.69545E-05	0.2	-0.00043	0.00914	0.009575	0.003383	0.00308	KHB	SHB	KHB	
B30	500/400	1	-132.377	-3.11364E-05	0.2	-0.00016	0.00914	0.009296	0.003281	0.002987	KHB	SHB	KHB	
B31	500/400	0	13.215	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B31	500/400	1	30.216	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B31	500/400	0	-111.928	-1.13636E-06	0.2	-5.7E-06	0.00914	0.009146	0.003226	0.002937	KHB	SHB	KHB	
B31	500/400	1	-107.808	-2.40909E-06	0.2	-1.2E-05	0.00914	0.009152	0.003228	0.002939	KHB	SHB	KHB	
B33	500/400	0	-13.074	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B33	500/400	1	0.269	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B33	500/400	0	-26.934	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B33	500/400	1	-0.882	0	0.2	0	0.00914	0.00914	0.004081	0.0038	KHB	SHB	KHB	
B34	250/500	0	48.712	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B34	250/500	1	47.048	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B34	250/500	0	-60.623	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B34	250/500	1	-62.535	0	0.25	0	0.0077	0.0077	0.002699	0.002454	KHB	SHB	KHB	
B35	500/400	0	9.872	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B35	500/400	1	7.127	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B35	500/400	0	-49.667	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B35	500/400	1	-49.075	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B36	500/400	0	39.075	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B36	500/400	1	43.251	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B36	500/400	0	-68.989	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B36	500/400	1	-69.400	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B39	500/400	0	28.754	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B39	500/400	1	41.664	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B39	500/400	0	-97.706	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B39	500/400	1	-101.296	-9.54545E-07	0.2	-4.8E-06	0.00914	0.009145	0.003226	0.002936	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams X-Direction

B40	500/400	0	35.130	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B40	500/400	1	82.893	1.06364E-05	0.2	5.32E-05	0.005776	0.005829	0.001801	0.001617	SHB	SHB	SHB	
B40	500/400	0	-118.694	-1.78182E-05	0.2	-8.9E-05	0.00914	0.009229	0.003256	0.002964	KHB	SHB	KHB	
B40	500/400	1	-132.878	-0.000197773	0.2	-0.00099	0.00914	0.010129	0.004549	0.004237	KHB	SHB	KHB	
B41	250/500	0	89.721	0.000222227	0.25	0.000889	0.00516	0.006049	0.002341	0.002156	SHB	SHB	SHB	**KHB**
B41	250/500	1	88.114	2.04545E-05	0.25	8.18E-05	0.00516	0.005242	0.002023	0.001863	SHB	SHB	SHB	
B41	250/500	0	-117.203	-0.000208227	0.25	-0.00083	0.0077	0.008533	0.003795	0.003532	KHB	SHB	KHB	
B41	250/500	1	-112.566	-7.31818E-06	0.25	-2.9E-05	0.0077	0.007729	0.003419	0.003182	KHB	SHB	KHB	
B42	250/500	0	88.859	3.90909E-05	0.25	0.000156	0.00516	0.005316	0.002052	0.001889	SHB	SHB	SHB	**KHB**
B42	250/500	1	89.717	0.000214318	0.25	0.000857	0.00516	0.006017	0.002327	0.002143	SHB	SHB	SHB	
B42	250/500	0	-113.531	-2.4329E-05	0.25	-9.7E-05	0.0077	0.007797	0.003451	0.003211	KHB	SHB	KHB	
B42	250/500	1	-117.193	-0.000178636	0.25	-0.00071	0.0077	0.008415	0.002959	0.002693	KHB	SHB	KHB	
B43	500/400	0	81.330	1.54091E-05	0.2	7.7E-05	0.005776	0.005853	0.001808	0.001623	SHB	SHB	SHB	**KHB**
B43	500/400	1	41.756	-9.77273E-06	0.2	-4.9E-05	0.005776	0.005825	0.001799	0.001616	SHB	SHB	SHB	
B43	500/400	0	-132.896	-0.000234318	0.2	-0.00117	0.00914	0.010312	0.003653	0.003327	KHB	SHB	KHB	
B43	500/400	1	-118.353	-9.13636E-06	0.2	-4.6E-05	0.00914	0.009186	0.00324	0.00295	KHB	SHB	KHB	
B44	500/400	0	-12.392	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B44	500/400	1	0.269	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B44	500/400	0	-26.244	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B44	500/400	1	-0.964	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B45	500/400	0	9.258	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B45	500/400	1	8.299	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B45	500/400	0	-49.784	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B45	500/400	1	-46.716	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B46	500/400	0	38.945	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B46	500/400	1	44.317	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B46	500/400	0	-70.433	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B46	500/400	1	-67.202	0	0.2	0	0.00914	0.00914	0.004081	0.0038	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams X-Direction

B47	250/500	0	60.887	2.37436E-07	0.25	9.5E-07	0.00516	0.005161	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B47	250/500	1	25.796	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B47	250/500	0	-75.099	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B47	250/500	1	-19.967	0	0.25	0	0.0077	0.0077	0.002699	0.002454	KHB	SHB	KHB	
B48	500/400	0	-10.978	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B48	500/400	1	0.294	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B48	500/400	0	-27.964	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B48	500/400	1	-0.958	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B49	500/400	0	25.620	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B49	500/400	1	42.148	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B49	500/400	0	-115.148	-6.40909E-06	0.2	-3.2E-05	0.00914	0.009172	0.003235	0.002945	KHB	SHB	KHB	
B49	500/400	1	-103.855	-2.13636E-06	0.2	-1.1E-05	0.00914	0.009151	0.003228	0.002938	KHB	SHB	KHB	
B51	500/400	0	16.805	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B51	500/400	1	68.880	1.11364E-05	0.2	5.57E-05	0.005776	0.005832	0.001801	0.001617	SHB	SHB	SHB	
B51	500/400	0	-115.388	-5.72727E-06	0.2	-2.9E-05	0.00914	0.009169	0.003234	0.002944	KHB	SHB	KHB	
B51	500/400	1	-107.363	-0.000004	0.2	-0.00002	0.00914	0.00916	0.003231	0.002941	KHB	SHB	KHB	
B52	500/400	0	67.749	4.13636E-06	0.2	2.07E-05	0.005776	0.005797	0.001791	0.001608	SHB	SHB	SHB	**KHB**
B52	500/400	1	31.435	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B52	500/400	0	-105.078	-1.27273E-06	0.2	-6.4E-06	0.00914	0.009146	0.003226	0.002937	KHB	SHB	KHB	
B52	500/400	1	-30.445	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B53	500/400	0	34.694	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B53	500/400	1	70.713	7.77273E-06	0.2	3.89E-05	0.005776	0.005815	0.001796	0.001613	SHB	SHB	SHB	
B53	500/400	0	-36.339	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B53	500/400	1	-111.636	-5.95455E-06	0.2	-3E-05	0.00914	0.00917	0.003235	0.002945	KHB	SHB	KHB	
B54	500/400	0	71.382	1.87273E-05	0.2	9.36E-05	0.005776	0.00587	0.001813	0.001628	SHB	SHB	SHB	**KHB**
B54	500/400	1	21.647	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B54	500/400	0	-113.514	-1.51364E-05	0.2	-7.6E-05	0.00914	0.009216	0.003251	0.00296	KHB	SHB	KHB	
B54	500/400	1	-115.885	1.54545E-06	0.2	7.73E-06	0.00914	0.009148	0.004085	0.003804	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams X-Direction

B57	250/500	0	89.731	0.000247318	0.25	0.000989	0.00516	0.006149	0.002385	0.002197	SHB	SHB	SHB	**KHB**
B57	250/500	1	89.661	5.33182E-05	0.25	0.000213	0.00516	0.005373	0.002074	0.001909	SHB	SHB	SHB	
B57	250/500	0	-117.217	-0.000223455	0.25	-0.00089	0.0077	0.008594	0.003824	0.003559	KHB	SHB	KHB	
B57	250/500	1	-117.118	-2.64091E-05	0.25	-0.00011	0.0077	0.007806	0.003455	0.003215	KHB	SHB	KHB	
B58	250/500	0	89.736	0.000254955	0.25	0.00102	0.00516	0.00618	0.002399	0.00221	SHB	SHB	SHB	**KHB**
B58	250/500	1	89.657	4.10909E-05	0.25	0.000164	0.00516	0.005324	0.002055	0.001892	SHB	SHB	SHB	
B58	250/500	0	-117.226	-0.000228955	0.25	-0.00092	0.0077	0.008616	0.003834	0.003569	KHB	SHB	KHB	
B58	250/500	1	-117.112	-9.90909E-06	0.25	-4E-05	0.0077	0.00774	0.003424	0.003186	KHB	SHB	KHB	
B59	250/500	0	89.800	0.000450909	0.25	0.001804	0.00516	0.006964	0.002743	0.00253	KHB	SHB	KHB	**KHB**
B59	250/500	1	89.798	0.000415682	0.25	0.001663	0.00516	0.006823	0.002681	0.002473	KHB	SHB	KHB	
B59	250/500	0	-117.310	-0.000406636	0.25	-0.00163	0.0077	0.009327	0.004169	0.003883	KHB	SHB	KHB	
B59	250/500	1	-117.314	-0.000400636	0.25	-0.0016	0.0077	0.009303	0.004158	0.003872	KHB	SHB	KHB	
B60	250/500	0	89.791	0.000423955	0.25	0.001696	0.00516	0.006856	0.002696	0.002486	KHB	SHB	KHB	**KHB**
B60	250/500	1	89.788	0.000387182	0.25	0.001549	0.00516	0.006709	0.002631	0.002426	KHB	SHB	KHB	
B60	250/500	0	-117.303	-0.000406136	0.25	-0.00162	0.0077	0.009325	0.004168	0.003882	KHB	SHB	KHB	
B60	250/500	1	-117.299	-0.000359	0.25	-0.00144	0.0077	0.009136	0.003222	0.002933	KHB	SHB	KHB	
B61	500/400	0	0.565	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B61	500/400	1	-9.096	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B61	500/400	0	-53.283	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B61	500/400	1	-63.312	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B62	500/400	0	32.533	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B62	500/400	1	35.946	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B62	500/400	0	-70.236	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B62	500/400	1	-72.584	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B63	500/400	0	24.972	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B63	500/400	1	56.867	7.72727E-07	0.2	3.86E-06	0.005776	0.00578	0.001785	0.001603	SHB	SHB	SHB	
B63	500/400	0	-98.321	-0.0000015	0.2	-7.5E-06	0.00914	0.009148	0.003227	0.002937	KHB	SHB	KHB	
B63	500/400	1	-130.180	-5.07757E-05	0.2	-0.00025	0.00914	0.009394	0.003317	0.003019	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams

B65	500/400	0	50.028	1.04545E-06	0.2	5.23E-06	0.005776	0.005781	0.001786	0.001604	SHB	SHB	SHB	**KHB**
B65	500/400	1	44.543	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B65	500/400	0	-107.898	-4.04545E-06	0.2	-2E-05	0.00914	0.00916	0.003231	0.002941	KHB	SHB	KHB	
B65	500/400	1	-126.196	-0.000025	0.2	-0.00013	0.00914	0.009265	0.003269	0.002976	KHB	SHB	KHB	
B66	500/400	0	44.723	1.59091E-06	0.2	7.95E-06	0.005776	0.005784	0.001787	0.001604	SHB	SHB	SHB	**KHB**
B66	500/400	1	50.181	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B66	500/400	0	-126.143	-2.57273E-05	0.2	-0.00013	0.00914	0.009269	0.003271	0.002978	KHB	SHB	KHB	
B66	500/400	1	-105.463	-4.45455E-06	0.2	-2.2E-05	0.00914	0.009162	0.003232	0.002942	KHB	SHB	KHB	
B68	500/400	0	-6.086	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B68	500/400	1	0.949	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B68	500/400	0	-33.829	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B68	500/400	1	-1.387	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B70	500/400	0	-16.171	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B70	500/400	1	0.930	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B70	500/400	0	-48.947	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B70	500/400	1	-0.941	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B71	500/400	0	-20.954	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	**KHB**
B71	500/400	1	1.088	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B71	500/400	0	-45.604	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B71	500/400	1	-0.732	1.74091E-05	0.2	8.7E-05	0.00914	0.009227	0.004122	0.003839	KHB	SHB	KHB	
B72	250/500	0	89.811	0.000476636	0.25	0.001907	0.00516	0.007067	0.002788	0.002572	KHB	SHB	KHB	**KHB**
B72	250/500	1	89.807	0.000434955	0.25	0.00174	0.00516	0.0069	0.002715	0.002504	KHB	SHB	KHB	
B72	250/500	0	-117.327	-0.000446273	0.25	-0.00179	0.0077	0.009485	0.004244	0.003953	KHB	SHB	KHB	
B72	250/500	1	-117.329	-0.000409545	0.25	-0.00164	0.0077	0.009338	0.004175	0.003888	KHB	SHB	KHB	
B73	250/500	0	89.832	0.000536409	0.25	0.002146	0.00516	0.007306	0.002895	0.002672	KHB	SHB	KHB	**KHB**
B73	250/500	1	89.827	0.000486909	0.25	0.001948	0.00516	0.007108	0.002806	0.002589	KHB	SHB	KHB	
B73	250/500	0	-117.358	-0.000500864	0.25	-0.002	0.0077	0.009703	0.004348	0.004049	KHB	SHB	KHB	
B73	250/500	1	-117.357	-0.000491864	0.25	-0.00197	0.0077	0.009667	0.004331	0.004033	KHB	SHB	KHB	

Table A. 1 (cont.) Damage zones of beams

B74	250/500	0	7.099	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B74	250/500	1	1.082	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B74	250/500	0	-26.356	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B74	250/500	1	0.180	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B75	250/500	0	11.234	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B75	250/500	1	1.035	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B75	250/500	0	-29.041	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B75	250/500	1	0.200	0	0.25	0	0.00516	0.00516	0.001595	0.001432	SHB	SHB	SHB	
B77	500/400	0	31.246	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	**KHB**
B77	500/400	1	34.627	0	0.2	0	0.005776	0.005776	0.001784	0.001602	SHB	SHB	SHB	
B77	500/400	0	-75.494	0	0.2	0	0.00914	0.00914	0.003224	0.002935	KHB	SHB	KHB	
B77	500/400	1	-61.609	0	0.2	0	0.00914	0.00914	0.004081	0.0038	KHB	SHB	KHB	
B78	250/500	0	43.540	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B78	250/500	1	40.421	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B78	250/500	0	-51.721	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B78	250/500	1	-81.833	-7.72727E-07	0.25	-3.1E-06	0.0077	0.007703	0.003407	0.003171	KHB	SHB	KHB	
B79	250/500	0	50.685	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	**KHB**
B79	250/500	1	48.591	0	0.25	0	0.00516	0.00516	0.001992	0.001834	SHB	SHB	SHB	
B79	250/500	0	-60.533	0	0.25	0	0.0077	0.0077	0.003405	0.003169	KHB	SHB	KHB	
B79	250/500	1	-88.404	-1.54545E-06	0.25	-6.2E-06	0.0077	0.007706	0.002701	0.002457	KHB	SHB	KHB	

Appendix B

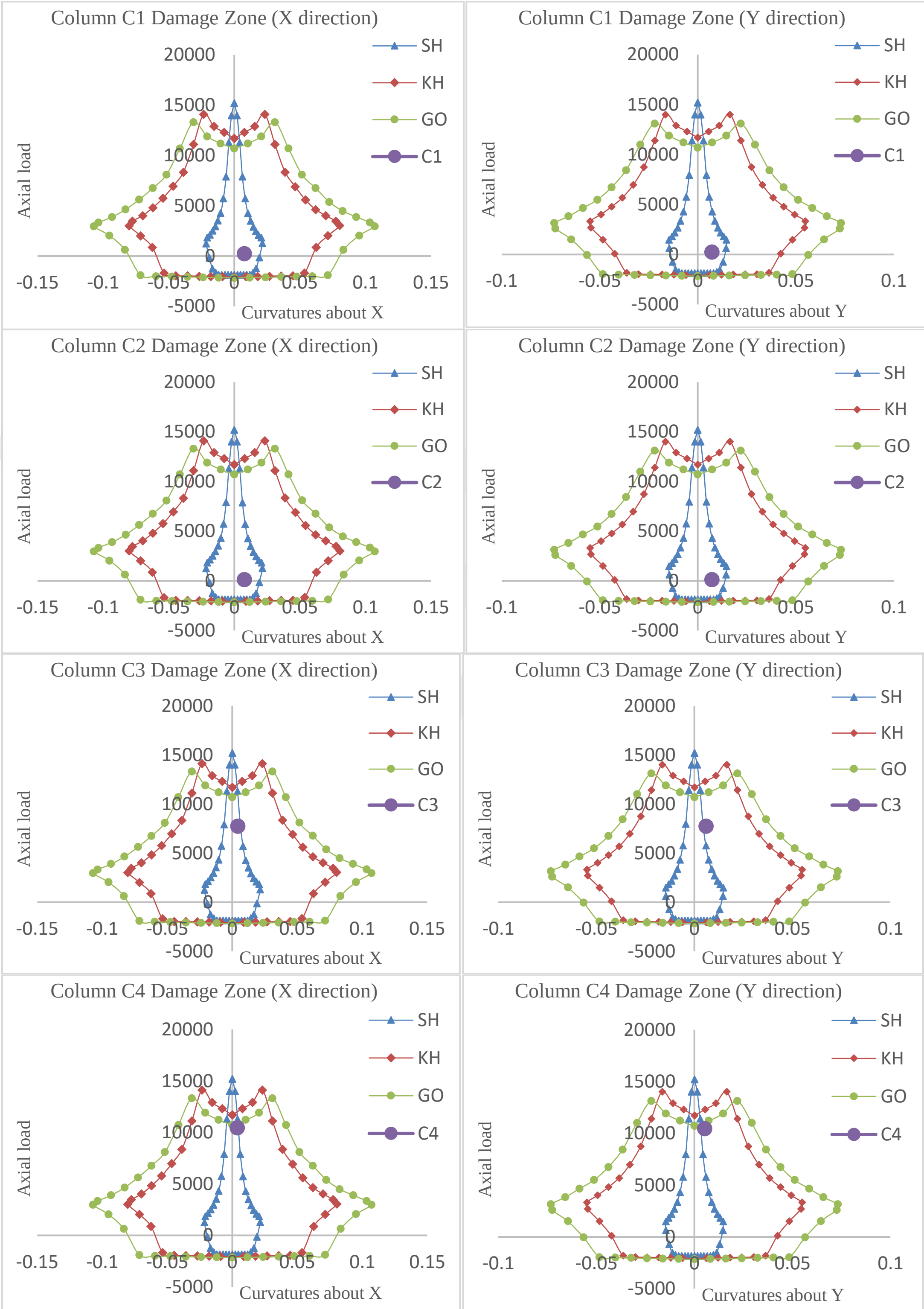

Figure B. 1 Columns Damage Zones under seismic load in both directions

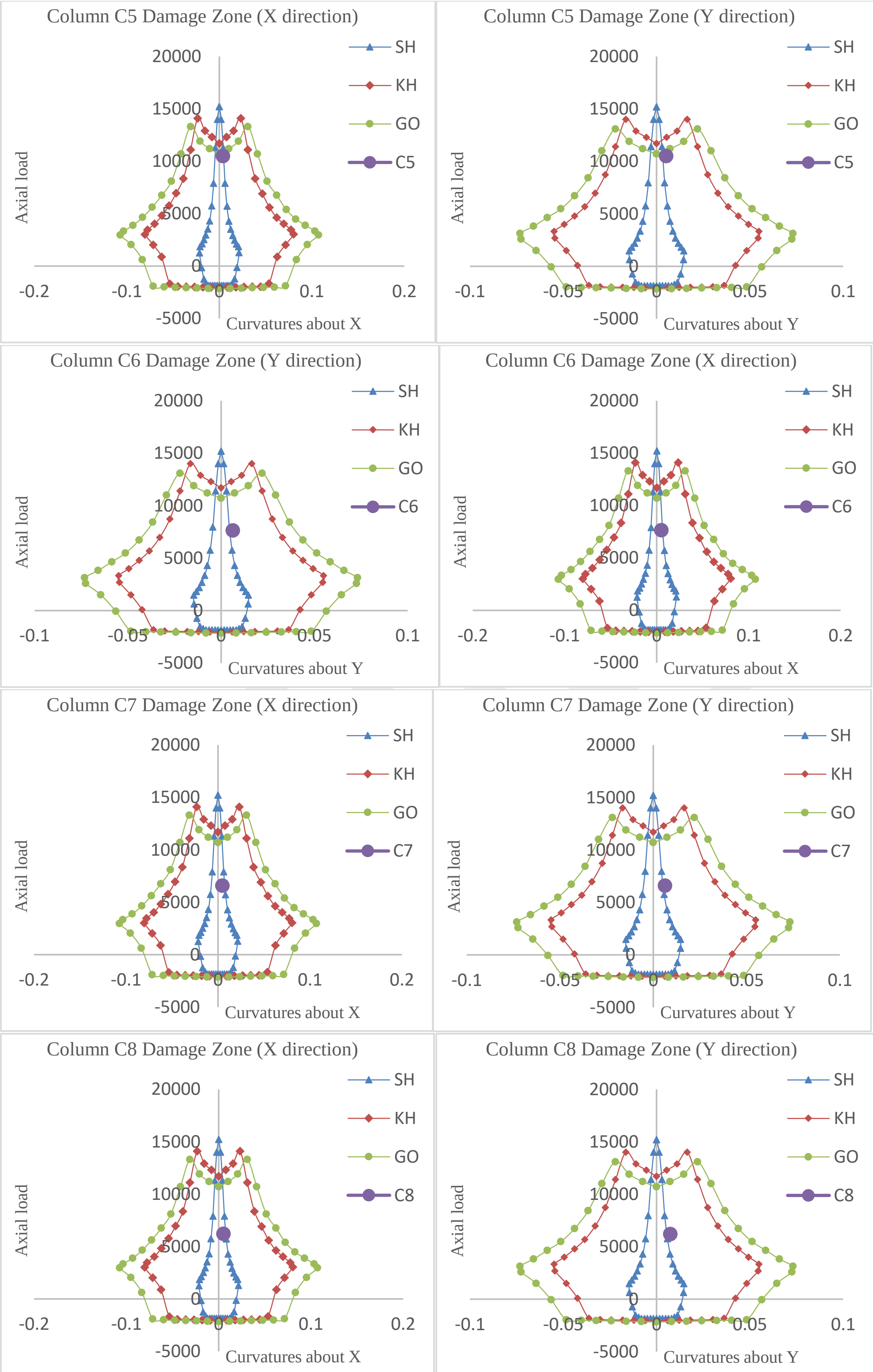

Figure B. 1 Columns Damage Zones under seismic load in both directions

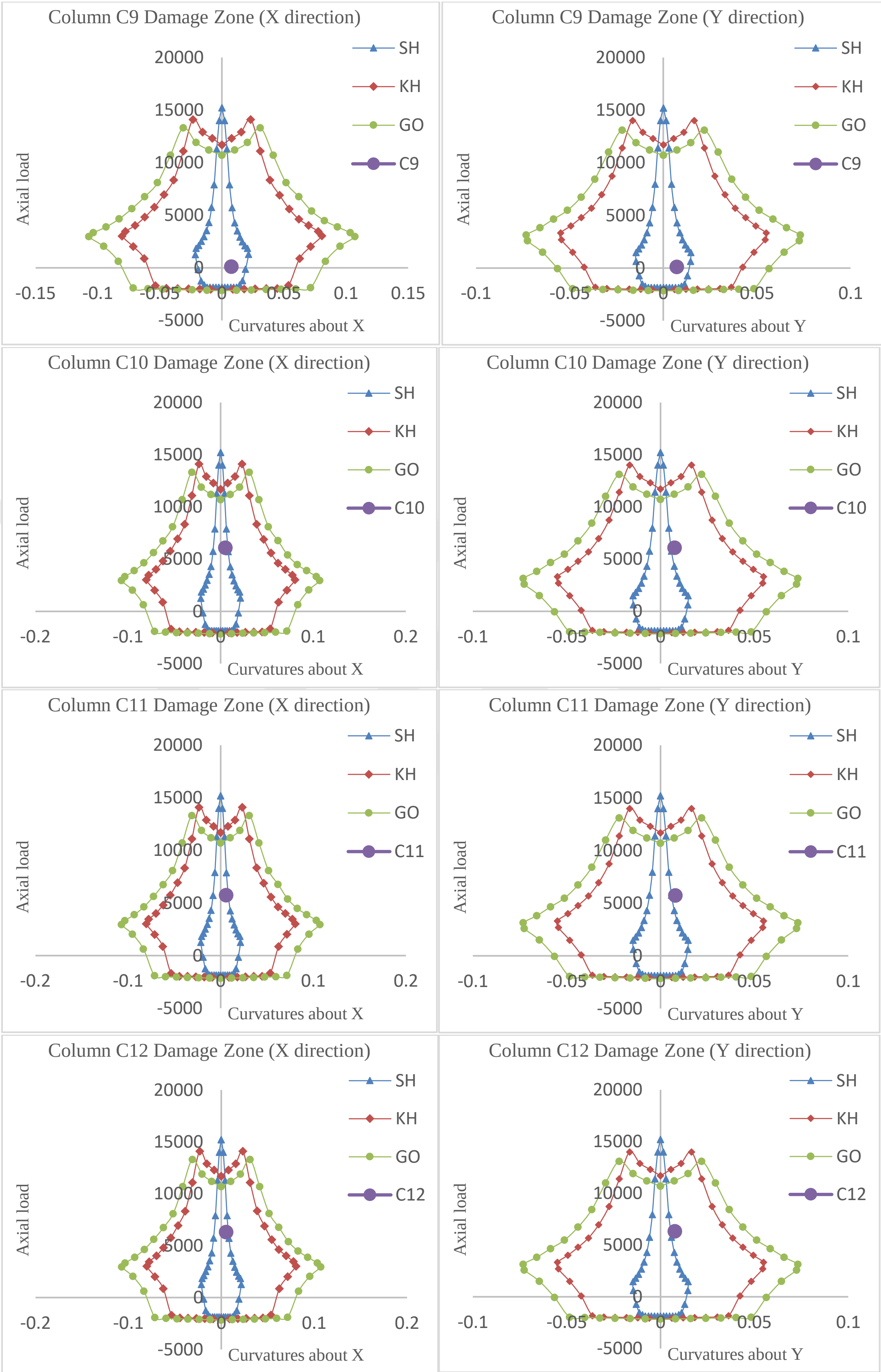

Figure B. 1 Columns Damage Zones under seismic load in both directions

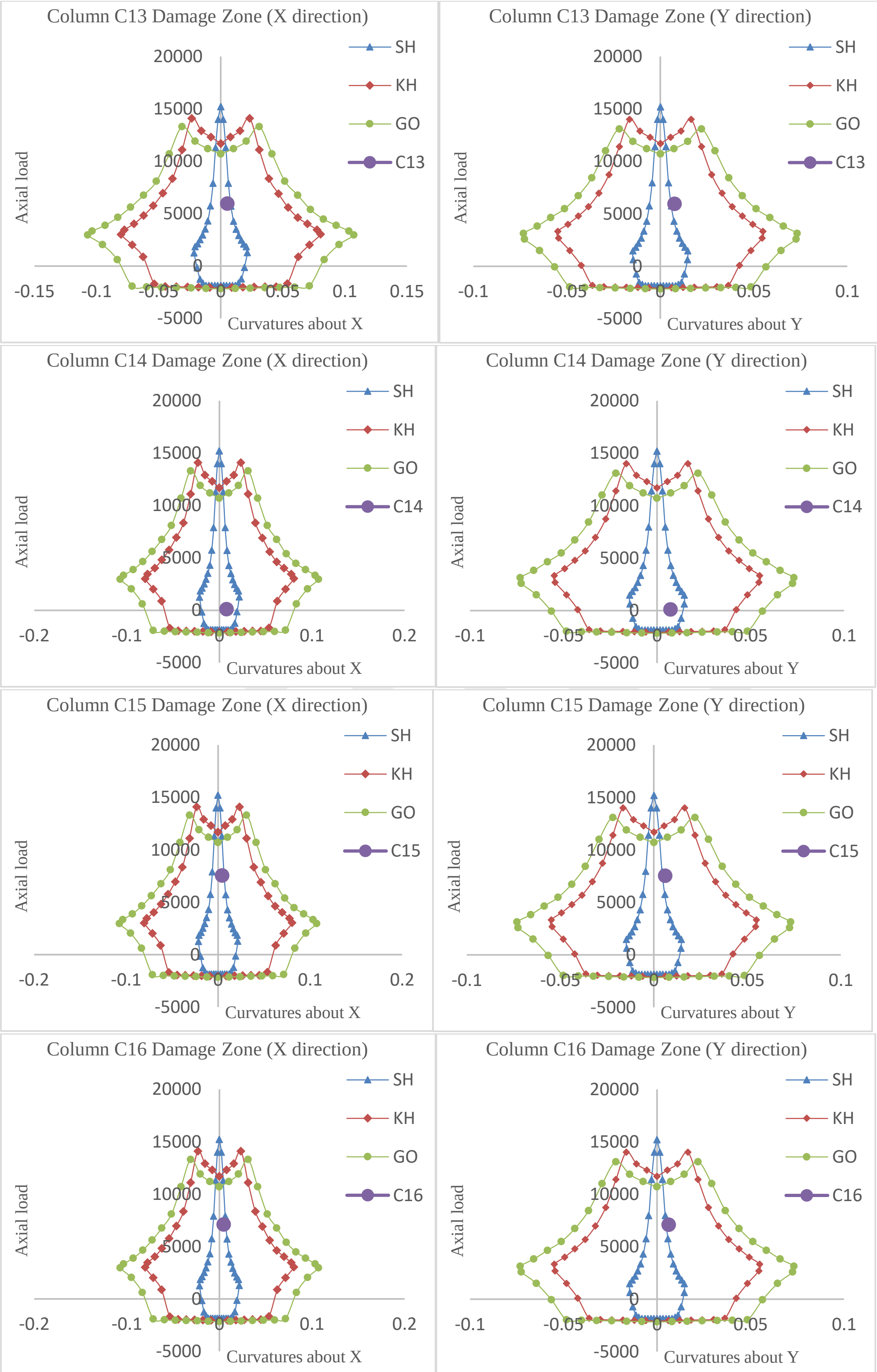

Figure B. 1 Columns Damage Zones under seismic load in both directions

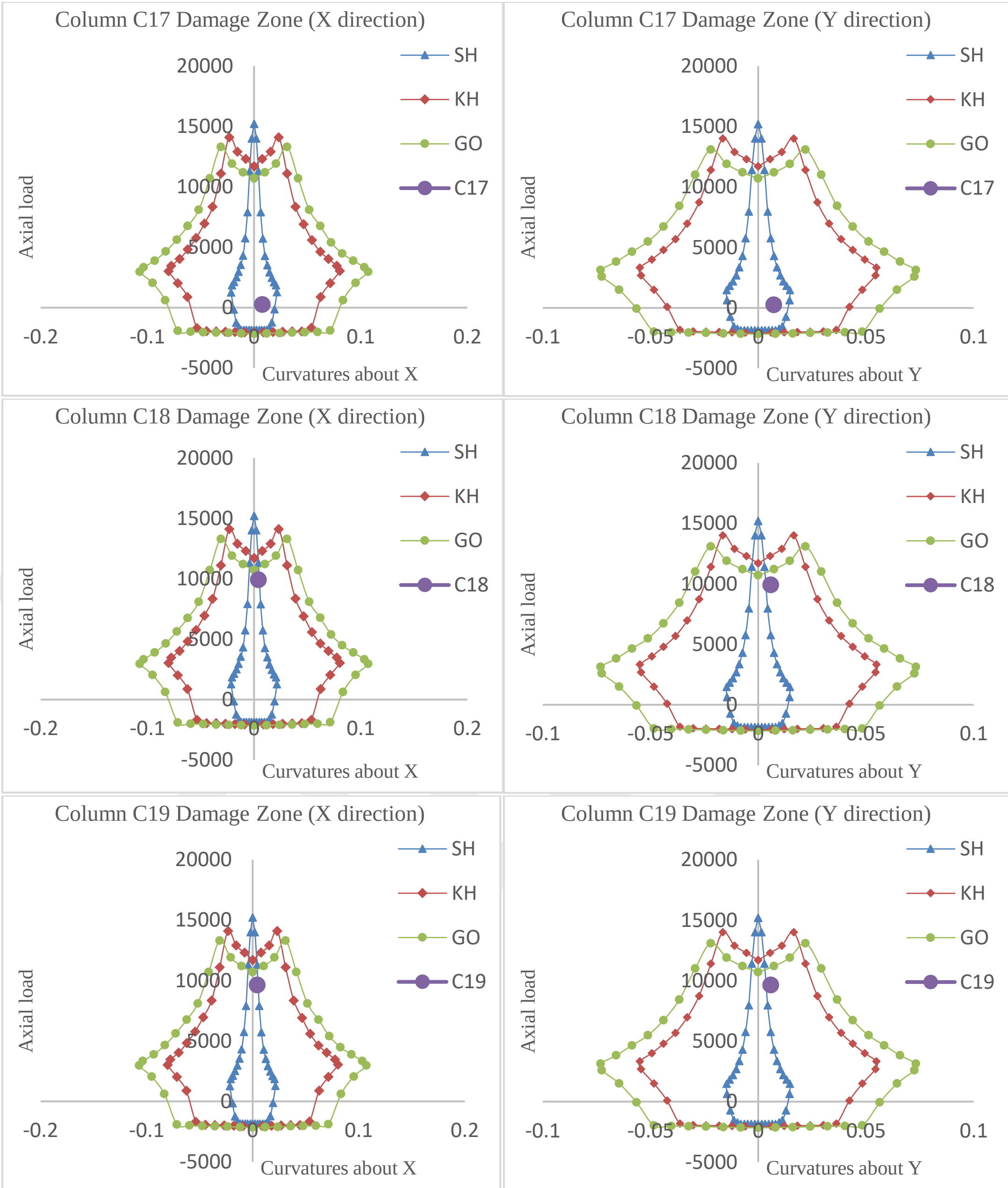

Figure B. 1 Columns Damage Zones under seismic load in both directions

Appendix C

Table C. 1 Damage Zones of Shear Walls

Story	Wall	Hinge	No	Coordinate	Material	Stress	Strain	Damage Zone	
				mm		MPa		FIBER	Section
Story1	W2	W2	1	-1150	S420	422.13	0.002784	KHB	KHB
Story1	W2	W2	2	-1150	C35	35	0.002784	KHB	
Story1	W2	W2	3	-1150	C35 W2	34.99	0.002784	KHB	
Story1	W2	W2	4	-850	S420	420.69	0.002323	SHB	
Story1	W2	W2	5	-850	C35	35	0.002323	SHB	
Story1	W2	W2	6	-850	C35 W2	34.97	0.002323	SHB	
Story1	W2	W2	7	-350	S420	315.67	0.001578	SHB	
Story1	W2	W2	8	-350	C35	33.37	0.001578	SHB	
Story1	W2	W2	9	350	C35	34.49	0.001869	SHB	
Story1	W2	W2	10	350	S420	373.87	0.001869	SHB	
Story1	W2	W2	11	850	C35 W2	34.91	0.002673	KHB	
Story1	W2	W2	12	850	C35	34.99	0.002673	KHB	
Story1	W2	W2	13	850	S420	421.79	0.002673	KHB	
Story1	W2	W2	14	1150	C35 W2	35	0.003155	KHB	
Story1	W2	W2	15	1150	C35	35	0.003155	KHB	
Story1	W2	W2	16	1150	S420	423.29	0.003155	KHB	
Story1	W3	W3	1	-1237.5	S420	422.24	0.00282	KHB	KHB
Story1	W3	W3	2	-1237.5	C35	35	0.00282	KHB	
Story1	W3	W3	3	-1237.5	C35 W3	34.99	0.00282	KHB	
Story1	W3	W3	4	-1112.5	S420	421.66	0.002633	KHB	
Story1	W3	W3	5	-1112.5	C35	34.99	0.002633	KHB	
Story1	W3	W3	6	-1112.5	C35 W3	34.99	0.002633	KHB	
Story1	W3	W3	7	-525	S420	377.43	0.001887	SHB	
Story1	W3	W3	8	-525	C35	34.56	0.001887	SHB	
Story1	W3	W3	9	525	C35	34.9	0.002084	SHB	
Story1	W3	W3	10	525	S420	416.79	0.002084	SHB	
Story1	W3	W3	11	1112.5	C35 W3	35	0.002942	KHB	
Story1	W3	W3	12	1112.5	C35	35	0.002942	KHB	
Story1	W3	W3	13	1112.5	S420	422.62	0.002942	KHB	
Story1	W3	W3	14	1237.5	C35 W3	35	0.003124	KHB	
Story1	W3	W3	15	1237.5	C35	34.99	0.003124	KHB	
Story1	W3	W3	16	1237.5	S420	423.19	0.003124	KHB	

Table C. 2 Damage Zones of Shear Walls

Story	Wall	Hinge	No	Coordinate	Material	Stress	Strain	Damage Zone	
Story1	W5	W5/W6	1	-1112.5	S420	421.26	0.002503	KHB	KHB
Story1	W5	W5/W6	2	-1112.5	C35	35	0.002503	KHB	
Story1	W5	W5/W6	3	-1112.5	C35 W5/6	34.98	0.002503	KHB	
Story1	W5	W5/W6	4	-812.5	S420	420.86	0.002375	SHB	
Story1	W5	W5/W6	5	-812.5	C35	35	0.002375	SHB	
Story1	W5	W5/W6	6	-812.5	C35 W5/6	34.99	0.002375	SHB	
Story1	W5	W5/W6	7	-331.3	S420	420.79	0.002352	SHB	
Story1	W5	W5/W6	8	-331.3	C35	35	0.002352	SHB	
Story1	W5	W5/W6	9	331.3	C35	35	0.002647	KHB	
Story1	W5	W5/W6	10	331.3	S420	421.7	0.002647	KHB	
Story1	W5	W5/W6	11	812.5	C35 W5/6	35	0.002861	KHB	
Story1	W5	W5/W6	12	812.5	C35	35	0.002861	KHB	
Story1	W5	W5/W6	13	812.5	S420	422.37	0.002861	KHB	
Story1	W5	W5/W6	14	1112.5	C35 W5/6	35	0.002994	KHB	
Story1	W5	W5/W6	15	1112.5	C35	35	0.002994	KHB	
Story1	W5	W5/W6	16	1112.5	S420	422.79	0.002994	KHB	
Story1	W6	W6	1	-1112.5	S420	424.17	0.003439	KHB	KHB
Story1	W6	W6	2	-1112.5	C35	35	0.003439	KHB	
Story1	W6	W6	3	-1112.5	C35	35	0.003439	KHB	
Story1	W6	W6	4	-812.5	S420	423.52	0.00323	KHB	
Story1	W6	W6	5	-812.5	C35	35	0.00323	KHB	
Story1	W6	W6	6	-812.5	C35	35	0.00323	KHB	
Story1	W6	W6	7	-331.3	S420	422.48	0.002896	KHB	
Story1	W6	W6	8	-331.3	C35	34.99	0.002896	KHB	
Story1	W6	W6	9	331.3	C35	35	0.00252	KHB	
Story1	W6	W6	10	331.3	S420	421.31	0.00252	KHB	
Story1	W6	W6	11	812.5	C35	35	0.002634	KHB	
Story1	W6	W6	12	812.5	C35	35	0.002634	KHB	
Story1	W6	W6	13	812.5	S420	421.67	0.002634	KHB	
Story1	W6	W6	14	1112.5	C35	35	0.002706	KHB	
Story1	W6	W6	15	1112.5	C35	35	0.002706	KHB	
Story1	W6	W6	16	1112.5	S420	421.89	0.002706	KHB	
Story1	W13	W13	1	-1375	S420	378.58	0.001893	SHB	SHB
Story1	W13	W13	2	-1375	C35	34.59	0.001893	SHB	
Story1	W13	W13	3	-1375	C35 W13	33.45	0.001893	SHB	
Story1	W13	W13	4	-1025	S420	325.93	0.00163	SHB	
Story1	W13	W13	5	-1025	C35	33.57	0.00163	SHB	
Story1	W13	W13	6	-1025	C35 W13	32.2	0.00163	SHB	
Story1	W13	W13	7	-425	S420	235.92	0.00118	SHB	
Story1	W13	W13	8	-425	C35	29.28	0.00118	SHB	
Story1	W13	W13	9	425	C35	29.26	0.001178	SHB	
Story1	W13	W13	10	425	S420	235.68	0.001178	SHB	
Story1	W13	W13	11	1025	C35 W13	32.48	0.001689	SHB	
Story1	W13	W13	12	1025	C35	33.8	0.001689	SHB	
Story1	W13	W13	13	1025	S420	337.7	0.001689	SHB	
Story1	W13	W13	14	1375	C35 W13	33.9	0.001989	SHB	
Story1	W13	W13	15	1375	C35	34.96	0.001989	SHB	
Story1	W13	W13	16	1375	S420	397.74	0.001989	SHB	

Table C. 3 Damage Zones of Shear Walls

Story	Wall	Hinge	No	Coordinate	Material	Stress	Strain	Damage Zone	
Story1	W14	W14	1	-1825	S420	417.39	0.002087	SHB	SHB
Story1	W14	W14	2	-1825	C35	34.99	0.002087	SHB	
Story1	W14	W14	3	-1825	C35 W14	34.37	0.002087	SHB	
Story1	W14	W14	4	-1325	S420	363.84	0.001819	SHB	
Story1	W14	W14	5	-1325	C35	34.3	0.001819	SHB	
Story1	W14	W14	6	-1325	C35 W14	33.1	0.001819	SHB	
Story1	W14	W14	7	-537.5	S420	279.5	0.001397	SHB	
Story1	W14	W14	8	-537.5	C35	32.6	0.001397	SHB	
Story1	W14	W14	9	537.5	C35	29.27	0.001179	SHB	
Story1	W14	W14	10	537.5	S420	235.79	0.001179	SHB	
Story1	W14	W14	11	1325	C35 W14	30.76	0.001498	SHB	
Story1	W14	W14	12	1325	C35	33.06	0.001498	SHB	
Story1	W14	W14	13	1325	S420	299.65	0.001498	SHB	
Story1	W14	W14	14	1825	C35 W14	32.54	0.001701	SHB	
Story1	W14	W14	15	1825	C35	33.85	0.001701	SHB	
Story1	W14	W14	16	1825	S420	340.27	0.001701	SHB	
Story1	W15	W20/21	1	-2125	S420	422.9	0.003029	KHB	KHB
Story1	W15	W20/21	2	-2125	C35	35	0.003029	KHB	
Story1	W15	W20/21	3	-2125	35 W20/2	34.98	0.003029	KHB	
Story1	W15	W20/21	4	-1625	S420	421.68	0.002638	KHB	
Story1	W15	W20/21	5	-1625	C35	34.92	0.002638	KHB	
Story1	W15	W20/21	6	-1625	35 W20/2	34.93	0.002638	KHB	
Story1	W15	W20/21	7	-687.5	S420	380.64	0.001903	SHB	
Story1	W15	W20/21	8	-687.5	C35	34.63	0.001903	SHB	
Story1	W15	W20/21	9	687.5	C35	30.12	0.001234	SHB	
Story1	W15	W20/21	10	687.5	S420	246.9	0.001234	SHB	
Story1	W15	W20/21	11	1625	35 W20/2	32.52	0.001698	SHB	
Story1	W15	W20/21	12	1625	C35	33.83	0.001698	SHB	
Story1	W15	W20/21	13	1625	S420	339.56	0.001698	SHB	
Story1	W15	W20/21	14	2125	35 W20/2	34.46	0.002107	SHB	
Story1	W15	W20/21	15	2125	C35	34.97	0.002107	SHB	
Story1	W15	W20/21	16	2125	S420	420.02	0.002107	SHB	
Story1	W20	W20/21	1	-2125	S420	423.94	0.003363	KHB	KHB
Story1	W20	W20/21	2	-2125	C35	34.99	0.003363	KHB	
Story1	W20	W20/21	3	-2125	35 W20/2	35	0.003363	KHB	
Story1	W20	W20/21	4	-1625	S420	423.25	0.003143	KHB	
Story1	W20	W20/21	5	-1625	C35	35	0.003143	KHB	
Story1	W20	W20/21	6	-1625	35 W20/2	35	0.003143	KHB	
Story1	W20	W20/21	7	-687.5	S420	422.07	0.002765	KHB	
Story1	W20	W20/21	8	-687.5	C35	35	0.002765	KHB	
Story1	W20	W20/21	9	687.5	C35	34.99	0.002375	SHB	
Story1	W20	W20/21	10	687.5	S420	420.86	0.002375	SHB	
Story1	W20	W20/21	11	1625	35 W20/2	34.62	0.002138	SHB	
Story1	W20	W20/21	12	1625	C35	34.99	0.002138	SHB	
Story1	W20	W20/21	13	1625	S420	420.12	0.002138	SHB	
Story1	W20	W20/21	14	2125	35 W20/2	34.22	0.002056	SHB	
Story1	W20	W20/21	15	2125	C35	34.99	0.002056	SHB	
Story1	W20	W20/21	16	2125	S420	411.13	0.002056	SHB	

Table C. 4 Damage Zones of Shear Walls

Story	Wall	Hinge	No	Coordinate	Material	Stress	Strain	Damage Zone	
Story1	W21	W20/21	1	-2125	S420	426.56	0.004205	KHB	KHB
Story1	W21	W20/21	2	-2125	C35	35	0.004205	KHB	
Story1	W21	W20/21	3	-2125	35 W20/2	34.99	0.004205	KHB	
Story1	W21	W20/21	4	-1625	S420	425.38	0.003824	KHB	
Story1	W21	W20/21	5	-1625	C35	35	0.003824	KHB	
Story1	W21	W20/21	6	-1625	35 W20/2	35	0.003824	KHB	
Story1	W21	W20/21	7	-687.5	S420	423.16	0.003113	KHB	
Story1	W21	W20/21	8	-687.5	C35	35	0.003113	KHB	
Story1	W21	W20/21	9	687.5	C35	35	0.002134	SHB	
Story1	W21	W20/21	10	687.5	S420	420.1	0.002134	SHB	
Story1	W21	W20/21	11	1625	35 W20/2	34.9	0.002197	SHB	
Story1	W21	W20/21	12	1625	C35	34.99	0.002197	SHB	
Story1	W21	W20/21	13	1625	S420	420.3	0.002197	SHB	
Story1	W21	W20/21	14	2125	35 W20/2	34.99	0.002591	KHB	
Story1	W21	W20/21	15	2125	C35	35	0.002591	KHB	
Story1	W21	W20/21	16	2125	S420	421.53	0.002591	KHB	
Story1	W23	W23	1	-775	S420	423.26	0.003145	KHB	KHB
Story1	W23	W23	2	-775	C35	35	0.003145	KHB	
Story1	W23	W23	3	-775	C35 W23	35	0.003145	KHB	
Story1	W23	W23	4	-525	S420	422.66	0.002952	KHB	
Story1	W23	W23	5	-525	C35	35	0.002952	KHB	
Story1	W23	W23	6	-525	C35 W23	35	0.002952	KHB	
Story1	W23	W23	7	-200	S420	421.88	0.002702	KHB	
Story1	W23	W23	8	-200	C35	35	0.002702	KHB	
Story1	W23	W23	9	200	C35	35	0.002505	KHB	
Story1	W23	W23	10	200	S420	421.26	0.002505	KHB	
Story1	W23	W23	11	525	C35 W23	35	0.002711	KHB	
Story1	W23	W23	12	525	C35	35	0.002711	KHB	
Story1	W23	W23	13	525	S420	421.9	0.002711	KHB	
Story1	W23	W23	14	775	C35 W23	35	0.003012	KHB	
Story1	W23	W23	15	775	C35	35	0.003012	KHB	
Story1	W23	W23	16	775	S420	422.84	0.003012	KHB	

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