DESIGN OF A HIGH-RISE REINFORCED CONCRETE BUILDING ACCORDING TO TBDY 2018

OBAIDALLAH AHMED BALGHAITH ALI QAIQAB

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OBAIDALLAH AHMED BALGHAITH ALI QAIQAB
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OBAIDALLAH AHMED BALGHAITH ALI QAIQAB

APPROVED BY:

Ass. Prof. Dr. Önder UMUT (Işık University)
(Thesis advisor)

Ass Prof. Dr. Bora AKŞAR (Işık University)
(Member)

Ass Prof. Dr. Ercan Şerif Kaya (Alanya Alaaddin Keykubat University)
(Member)

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ABSTRACT

Due to the demand for additional livable space and lodging options for the urban population, towering structures (high-rises) are becoming more and more necessary as the world's population grows every day. For generations, people have had the need to construct big buildings. Tall structures used to be extremely difficult to construct because of a lack of seismic knowledge and computer technology. However, modern advancements in computer technology, amenities like lifts, and engineers' familiarity with earthquake movements are the main factors in the success of tall skyscraper construction. Tall structures are a problem in both industrialized and developing nations today.

Engineers now employ norms and guidelines created for normal buildings for tall skyscrapers. Engineers are limited in their ability to apply easy structural solutions for tall buildings since these laws are based on the structure's strength-based design and linear elastic analysis. Engineers may choose earthquake-resistant designs and conduct more complex analyses thanks to the emergence of non-linear behavior in structural systems. Due to a lack of understanding of the non-linear behavior of buildings and the adoption of laws based on the strength that is developed under seismic threat, engineers are forced to build low-rise and mid-rise structures.

According to earthquake legislation from 2007 that is largely recognized until 2019, high-rise buildings are not distinguishable from other structures and are equated with them. The Turkish Building Earthquake Regulation is published in 2019, and it analyzed high-rise buildings separately from other types of construction. High-rise structures are divided into three separate level classes under the Turkish Building Earthquake Regulation, and it has been determined to check them in accordance with that content when it comes to design issues.

There are five chapters in the research. The introduction is in the first chapter, while information on earthquake-resistant design is in the second. The third chapter is mostly made up of the variables that will be used in this thesis, such the projected computation and design of high-rise structures in the Turkish Building Earthquake Regulation in 2019. The fourth chapter compares the outcomes of linear calculations with the Turkish Building Earthquake Regulation from 2007 and looks at the performance analysis and design of buildings up to 94 meters tall and 30 floors in compliance with the Turkish Building Earthquake Regulation in 2019. A list of resources and annexures follows the fifth chapter's conclusion part.

Keywords: Tall Building, Earthquake Effects, Performance Analysis, Earthquake Code.

BETONARME YÜKSEK BİR BİNANIN TBDY 2018'E GÖRE TASARIMI

ÖZET

Kent nüfusunun ek yaşanabilir alan ve barınma seçeneklerine olan talebi nedeniyle, dünya nüfusunun her geçen gün artmasıyla birlikte yüksek yapılar (yüksek yapılar) giderek daha gerekli hale gelmektedir. Nesiller boyunca insanların büyük binalar inşa etme ihtiyacı olmuştur. Sismik bilgi ve bilgisayar teknolojisi eksikliği nedeniyle yüksek yapıların inşa edilmesi eskiden son derece zordu. Bununla birlikte, bilgisayar teknolojisindeki modern gelişmeler, asansörler gibi olanaklar ve mühendislerin deprem hareketlerine aşinalığı, yüksek gökdelen inşaatının başarısındaki ana faktörlerdi. Yüksek yapılar bugün hem sanayileşmiş hem de gelişmekte olan ülkelerde bir sorundur.

Mühendisler artık normal binalar için yüksek gökdelenler için oluşturulan normları ve yönergeleri kullanıyor. Bu yönetmelikler, yapıların mukavemete dayalı tasarımına ve doğrusal elastik analizine dayandığından, mühendislerin yüksek binalar için basit ve anlaşılır yapısal çözümleri kullanma becerileri sınırlıdır. Yapı sistemlerinde doğrusal olmayan davranışın ortaya çıkması sayesinde mühendisler depreme dayanıklı tasarımları tercih edebilmekte ve daha karmaşık analizler yapabilmektedir. Mühendisler, yapıların doğrusal olmayan davranışlarının bilinmemesi ve sismik tehlike altında oluşan dayanıma dayalı düzenlemelerin kullanılması nedeniyle az katlı ve orta katlı yapılar inşa etmek zorunda kalmışlardır.

Yüksek binalar diğer yapılardan ayırt edilmemekte ve 2007 yılından 2019 yılına kadar yaygın olarak kabul gören deprem kanunlarına göre diğer yapılarla eş tutulmaktadır. Yapıların Yüksek yapılar, Türkiye Bina Deprem Yönetmeliği'nde üç ayrı kot sınıfına ayrılmaktadır ve tasarım konularında bu içeriğe göre kontrol edilmesi belirlenmiştir.

Bu çalışma, betonarme perde duvar ve çerçeve sistemleri ile yapılan yüksek yapılar için lineer hesapların sonuçlarını ve dikkate alınan faktörleri incelemek amacıyla 2019 (yeni) deprem yönetmeliği ile 2007 deprem yönetmeliğini karşılaştırmaktadır. Analiz sırasında. Çalışma beş bölüme ayrılmıştır. Birinci bölümde giriş, ikinci bölümde ise deprem dayanıklı tasarım hakkında bilgiler yer almaktadır. Üçüncü bölüm çoğunlukla,

2019 yılında Türkiye Bina Deprem Yönetmeliği'nde öngörülen yüksek yapıların hesaplanması ve tasarımı gibi bu tezde kullanılacak değişkenlerden oluşmaktadır. Dördüncü bölüm, 94 metre yüksekliğindeki yapıların performans analizi ve tasarımını incelemektedir. Ve 2019 Türkiye Bina Deprem Yönetmeliği'ne göre 30 kat ve 2007 Türkiye Bina Deprem Yönetmeliği ile lineer hesaplamaların sonuçlarını karşılaştırmaktadır. Beşinci bölümün sonuç bölümünü kaynaklar ve ekler listesi takip etmektedir.

Anahtar Kelimeler: Yüksek Bina, Deprem Etkileri, Performans Analizi, Deprem Yönetmeliği.

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LIST OF SYMBOL

A_c	:	Column cross-sectional area [mm2]
A_{ch}	:	Shear wall gross cross-sectional area [mm2]
A_{sh}	:	Core concrete area [mm2]
b_k	:	Distance between outermost transverse reinforcement of column or shear wall [mm]
b_w	:	Beam body width, wall body thickness [mm]
D	:	Coefficient of excess strength [dimensionless]
d'	:	Net concrete cover [mm]
d_b	:	Longitudinal reinforcement diameter [mm]
$E_d^{(H)}$	:	Horizontal earthquake effect
$E_d^{(Z)}$	:	Vertical earthquake effect
E_c	:	Concrete modulus of elasticity [MPa]
E_s	:	Reinforcing steel modulus of elasticity [MPa]
$(EI)_e$	:	Effective section stiffness
F_1	:	1.0 second period local ground effect coefficient [dimensionless]
f_{ce}	:	Average (expected) strength of concrete [MPa]
f_{ck}	:	Concrete characteristic compressive strength [MPa]
f_{ctd}	:	Concrete design tensile strength [MPa]
F_s	:	Short period local ground effect coefficient
f_s	:	Compressive stress of wrapped concrete [MPa]
f_{su}	:	Reinforcing steel breaking strength [MPa]
f_{sy}	:	Yield strength of reinforcing steel [MPa]
f_{yd}	:	Reinforcing steel design yield strength [MPa]
f_{ye}	:	Average (expected) strength of reinforcing steel [MPa]

f_{yk}	:	Reinforcing steel characteristic yield strength [MPa]
f_{ywe}	:	Average (expected) strength of transverse reinforcing steel [MPa]
f_{ywk}	:	Characteristic yield strength of transverse reinforcement [MPa]
g	:	Gravity acceleration [m/s2]
H_w	:	Total curtain height above ground level [m]
M_{DEV}	:	Shear walls tipping moment [kNm]
M_o	:	Base tipping moment [kNm]
M_P	:	Plastic moment [kNm]
m_t	:	Total building mass above ground level [kg]
n	:	Live load participation coefficient [dimensionless]
R	:	Structural system coefficient of behavior
R_a	:	Earthquake load reduction coefficient
T	:	Natural vibration period [s]
T_P	:	The building's dominant natural vibration period [s]
V_d	:	Shear force from vertical and earthquake loads [kN]
V_r	:	Section shear strength [kN]
$V_{t,min}$	:	Minimum base shear force [kN]
S_1	:	One-second periodic spectral response acceleration
S_{D1}	:	Spectral response acceleration parameter at 1 second
S_{DS}	:	Spectral response acceleration parameter at short periods
Δ	:	Story drift
Ω	:	Over strength factor
ε	:	Unit deformation
β_{tE}	:	Equivalent base shear force amplification coefficient
γ_E	:	Empirical coefficient for calculating equivalent base shear force magnification
ρ	:	Reinforcement ratio
$\theta_{II,i}^{X}$	:	Second order indicator value
φ_p	:	Plastic curvature demand [m-1]
φ_p	:	Curvature before failure [m-1]
φ_y	:	Yield curvature [m-1]
σ	:	Stress [MPa]

LIST OF ABBREVIATIONS

CTBUH	:	Council on Tall Buildings and Urban Habitat.
TBDY 2018	:	Turkey New Seismic Design Code 2018 (TBSC).
FEMA	:	Federal Emergency Management Agency.
EDC	:	Engineering Design Consultant.
IYBDY	:	Istanbul Design Code for High-Rise Building (ISDCTB).
PEER	:	Pacific Earthquake Engineering Research Center.
TEHMIWA	:	Turkish Earthquake Hazard Map Interactive Web Application.
TBI	:	Tall Buildings Initiative.
SDOF	:	Single Degree of Freedom.
SAP2000	:	Integrated Software for Structural Analysis and Design
FBD	:	Force-Based Design.
PBARD	:	Performance-Based Assessment and Re-Design.
DBD	:	Deformation-Based Design
CSD	:	Class of Seismic Design.
DTS	:	Seismic Design Categories.
BYS	:	Building Height Categories.
EDC	:	Engineering Design Consultant.
TS500	:	Design and Construction Rules of Reinforced Concrete Structures
TS498	:	Loads to be Taken in the Dimensioning of Reinforced Concrete Elements

CHAPTER 1

1. INTRODUCTION

High-rise buildings have been famous structures in recent years due to rising country populations, improved material quality, and the desire to create beautiful structures. Structures' acceptability as tall buildings varies from nation to country. The height of the building is measured from the ground level to identify it as a tall building. However, the height restrictions of tall structures are now specified differently in several codes. The Turkey Building Earthquake Code 2018 (TBDY 2018) defines buildings exceeding the height restriction as 70 m in high seismicity areas and 105 m in low seismicity areas, depending on the earthquake design class. Particular guidelines for evaluating and designing buildings in this class are included in TBDY 2018. The Council on Tall Buildings and Urban Habitat (CTBUH) defines tall buildings as those with 14 stories or more and over 50 meters in height. The CTBUH also has set height limitations for super-tall structures of 300 meters and mega-tall buildings of 600 meters.

Show near analysis approaches, which demonstrate behavior closer to reality than linear analysis methods, have begun to be employed in recent years in addition to linear analysis methods. The use of non-linear analytic methods for performance evaluation in specific constructions and high-rise buildings is required by regulations in several countries. Recently, a performance-based design approach has become more popular.

In the computation and design of structures, linear analyzers are commonly utilized. Although the linear analysis in regular or low-rise structures can produce accurate findings, the actual behavior deviates when using linear analysis in high-rise or irregular buildings. As a result, nonlinear analytic approaches are chosen to evaluate high-rise structures' performance.

In high-rise structures, studies have shown that linear analysis only offers a preliminary understanding and that the nonlinear computation method should be utilized to get the most accurate findings.

1.1 Historical Development of Tall Buildings throughout History

Tall constructions have been needed for centuries. The first tall structures are the Egyptian pyramids. The first tall structures are the Egyptian pyramids, built to see the sky and royal tombs. B.C. The 2250s pyramids have long been an emblem of power, ostentation, and civilization. Buildings have improved materials and structural engineering. Formerly built for show, high-rise buildings are today constructed to meet the needs of an ever-growing population. Building heights have grown dramatically to accommodate more people in fewer spaces. Another cause for the rapid rise in building heights is advancements in construction materials. Due to the load-bearing systems of high-rise buildings, significant building components must be used. Egypt's pyramids are among the first tall structures erected. The Egyptian kings are buried in the pyramids. The Cheops Pyramid, a 139-meter-high monument for the Egyptian pharaoh, lies near Cairo.

The most significant impediment to the construction of high-rise structures has been stairwells. After the development of safe elevators, building heights climbed fast. From 5-6-story constructions to 100-storey and beyond structures today, especially following the industrial revolution, with rising material quality.

The Home Insurance Building, seen in Figure 1.1, began construction in 1883 and is finished in 1885 in Chicago. The Home Insurance Building has been recognized as the first skyscraper in the world by the Council on Tall Buildings and Urban Habitat (CTBUH). The Home Insurance Building is a 12-story structure with a 42-meter height. Steel frameworks are used to build the structure. It is destroyed in 1931 to make way for a replacement structure (Akçora, 2020).

Figure 1.1 Home Insurance Building, Chicago (Akçora, 2020).

Because concrete technology is not suited for constructing high-rise structures in the nineteenth century, building carrier systems are primarily composed of steel. With the advancement of concrete materials in the early twentieth century, more high-rise structures combining steel and reinforced concrete began to be constructed. The Ingalls Skyscraper, seen in Figure 1.2, is the world's first reinforced concrete tall building, with 16 stories and 64 meters, built in the United States in 1903.

Figure 1.2 Ingalls Building, Cincinnati (Akçora, 2020).

Different types of carrier systems are created as a result of structural engineering research conducted in the 1960s. Engineers such as Faazlur Khan, Hal Ivengar, William LeMessurıer, and Leslıe Robertson worked together to design shear wall carrier systems and tube carrier systems, which cleared the way to construct taller structures.

The number of high-rise structures is growing every year, especially with the adoption of composite carrier systems. In 2019, the Burj Khalifa in Dubai is the highest building in the world, according to CTBUH statistics, with a height of 828 meters (Figure 1.3). The building's intended use is as an office, a house, and a hotel, and it is made of steel and reinforced concrete (Akçora, 2020).

Figure 1.3 Burj Khalifa, Dubai (Akçora, 2020).

According to CTBUH statistics, Jeddah Tower is being erected in Jeddah city at 1000 meters and 167 stories. It is the world's highest structure (Figure 1.4).

Figure 1.4 Jeddah Tower (Akçora, 2020).

1.2 Historical Development of Tall Buildings in Turkey

Until 1970, the number of stories in buildings constructed in Turkey did not exceed 25. Since the 1970s, the number of high-rise structures has been quickly increasing. Tall buildings expanded soon after 1985, especially in the United States. Eight finished constructions are longer than 200 meters. The construction of the Metropol Istanbul Tower 1 is now complete, making it Turkey's highest structure. The structure, which has 70 stories, stands at 301 meters. Table 1.1 gives information about high-rise buildings in Turkey according to Wikipedia data (Akçora, 2020).

Table 1.1 High-rise buildings completed in Turkey (Akçora, 2020).

Rank	Name	City	Height (m)	Story	Year	Material
1	Metropol Istanbul Tower 1	Istanbul	301	70	2017	Reinforced concrete
2	Skyland Office Istanbul	Istanbul	284.1	65	2017	Reinforced concrete
3	Skyland Residence Istanbul	Istanbul	284.1	64	2017	Reinforced concrete
4	Sapphire of Istanbul	Istanbul	261	54	2011	Reinforced concrete
5	Mistral Office Tower	Izmir	216	48	2017	Reinforced concrete

CHAPTER 2

2. EARTHQUAKE-RESISTANT DESIGN

2.1 Introduction

The difficulties in designing a structure under its weight and additional vertical loads and designing it according to a highly variable and random force whose time, magnitude, duration, intensity, and direction are unknown can be easily estimated without underlining. Adding to these difficulties, today's conditions force economic design, and it has become inevitable to develop calculation techniques.

While Capacity Design, a Force-Based Method, is generally used as a design principle in engineering structures, the displacement-based, Performance-Based Design method is also quite up-to-date in recent years. In addition to the design philosophy, calculation methods have also advanced in parallel.

Today, calculation methods can be divided into linear and nonlinear analyses. Nonlinear analysis is split into a whole pushover and nonlinear analysis, whereas linear analysis is divided into three categories: equivalent seismic load method, mode coupling technique, and linear analysis in time history. The present pinnacle of structural engineering may be characterized as nonlinear analysis in the time domain.

Naturally, analysis defined as the endpoint has its difficulties. These can be listed as modeling the data to be used by reality, determining the cycle model that best represents the system's behavior under the effect of reversible earthquake loads, and choosing the computer program to be used.

This section briefly summarizes the calculation methods used to analyze reinforced concrete structures under the influence of horizontal earthquake loads.

2.2 Building Design Using Earthquake-Resistant Strategies

A powerful phrase that describes the fundamentals of design is deep design. The notion encompasses the selection of design loads, analytical tools, design methodologies, a preference for particular structural systems, and the objective of maximizing the economic performance of the structure. Gaining a deeper understanding of the nature of loads and how they affect structural systems can help engineers solve design challenges by enabling designers to adapt structural systems to specific types of loading (Taranath, 2009).

High-rise buildings are structures with a height of 35 meters or more with many stories spaced at regular intervals. They are built all over the globe, and their size and expense increase in lockstep with the advancement of life and the scarcity of open space. The core design of high-rise structures is dependent on energetic wind and earthquake exams. And today the optimum technique to complete these examinations utilizes computer programs for high-rise building auxiliary design. By employing an additional computer program on display after high-rise buildings' fundamental arrangement and layout are established, the supplementary plan of high-rise buildings, which verifies basic security for individual critical personnel, does not need the remarkable additional capability (Patil et al., 2013).

However, it is not an exaggeration to say that the performance of high-rise structures is nearly entirely established during the preliminary design phases, which focus on a variety of structural forms and outline analyses. In this step, the structural designer must precisely seal the gap in the entire picture. The distributions of transverse shear stiffness and bending stiffness per story influence high-rise structures' static and dynamic structural characteristics. As a result, in the early phases of high-rise structure design, a simple but accurate analytical approach that represents the structural stiffness of the entire situation is preferable to an analytical method that requires each structural part to be calculated individually, such as FEM. For high-rise structures, a variety of simple analytical approaches are used. Because high-rise buildings have many structural components, the fundamental strategy for simplification is to replace each with a continuous simple structural member equal to the original structure. The one-dimensional rod theory is best applied to this equivalently substituted constant component. Since the shear stiffness and bending stiffness obtained from the structural property influence the dynamic behavior of high-rise structures, Axial deformation, bending deformation, transverse shear deformation, shear-lag deformation, and torsional deformation are all examples of high-rise building deformations. The issue is how to account for these deformations while maintaining simplicity. Designing structures to withstand seismic loads is the most complex field of structural engineering. It involves the magnitude of the forces created by an earthquake and the considerably more destructive force, which is unexpected and impossible to quantify. As a result, earthquakes significantly damage people and infrastructure, even in nations with current construction rules and practices (Takabatake, 2011).

The design of a high-rise building requires, like other designs, the development of a physical description of an object while abiding by a number of restrictions and requirements. The conceptual, preliminary, and detailed design stages of a high-rise construction are the first, second, and third. Finding concepts and choosing the ideal subsystems and combinations are part of conceptual design. The creation of one or more abstract models marks the start of the initial design process. Final architectural, structural, electrical, and mechanical system drawings are produced during the detailed design stage, which also outlines a comprehensive solution for all subsystems (Khajehpour, 2001).

2.2.1 Design Principles for Earthquake-Resistant Buildings

Engineers must strengthen the structure and account for seismic forces while developing an earthquake-proof construction. Because earthquakes release energy in one direction, the strategy makes the structure push in the other direction. Here are some of the techniques used to make buildings more earthquake resistant.

- The load route's structural simplicity aids in the secure delivery of seismic loads to the foundation. However, the convoluted load path makes load transfer challenging and increases stress.
- Due to the fact that symmetry and uniformity improve earthquake performance, the structure should be symmetric and homogenous.
- The building should be symmetrical in terms of layout and height, and it should be designed to resist earthquake stresses in both directions. Consequently, the system is rigid in both directions and is bi-directional.

- A design's mass and stiffness centers should be coordinated to avoid lateral-torsional problems.
- The Diaphragm's suitability at each level of the tale must be upheld. It will stop the vertical elements from swaying too much.
- During an earthquake, all structures should experience uniform excitation, and the base and superstructure should be firmly attached (Celep, 2007).

2.2.2 Design Based On Capacity

The main goal of structural capacity design is to distribute the inelastic deformation requirements among the structures in a manner that plastic hinges develop in preset locations and successions. Capacity design alternatively relies on deterministic strength and ductility allocation in structural elements for a successful response and collapse prevention during a catastrophic earthquake by logically choosing the following energy dissipation regions so that the predetermined energy is chosen dissipation mechanism remains constant throughout the seismic action. The capacity design got its name from this link between the strength generated in the weaker component and the capacity of the more important component in a yielding state (Agrawal & Shrikhande, 2006).

Build plastic hinges at the ends of each beam on each story of reinforced concrete buildings with several stories to achieve this. With the exception of the foundation, columns and shear walls that make up storeys are still naturally elastic. A large column-weak beam structure would eliminate the possibility of a building's column sway mechanism (soft level) and avoid shear failures in columns and beams. (CEB, 1998).

Due to the capacity design principles of the Turkish Seismic Code, which result in plastic hinges at beams, columns are more resilient than beams framing into the same joint. Additionally, shear capacities for beams, columns, and shear walls are maintained higher than bending capacities to guarantee tensile failure at seismic forces larger than those anticipated in seismic design. Figure 2.1 displays the flowchart for the FEMA 749 performance selection (Ilki & Celep, 2012).

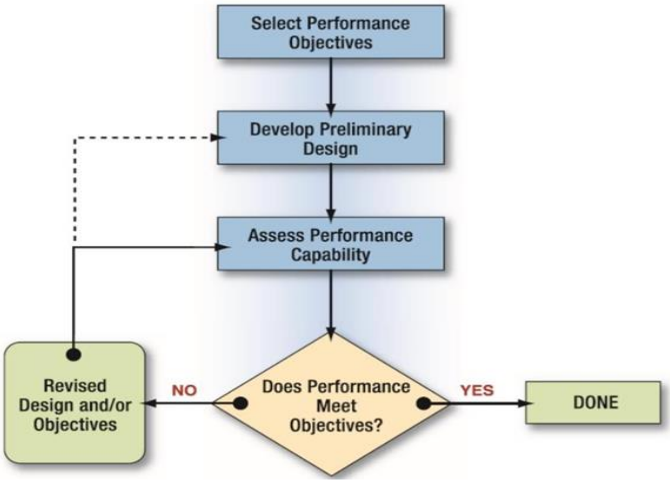


Figure 2.1 Flowchart for FEMA 749 performance selection

The deformations that occur owing to the structure's behavior and the excess forces are ignored in the capacity design. Consequently, the rules have imposed constraints such as the coefficient of the behavior of the structural system, transverse reinforcement criteria, and the need that columns be stronger than beams to ensure that the structure behaves ductility and is damaged in a controlled manner (Başot, 2010).

2.2.3 Design Based On Performance

Design based on performance is initially presented in FEMA 273/274 in October 1997 and then re-edited as FEMA 356 in November 2000. These initiatives are widely acknowledged as the first generation of laws for Performance-Based Seismic Design (PBSD). Both versions of the FEMA regulations are replaced by ASCE 41-06. (Tang et al., 2008).

Design based on performance is not created to be a replacement for established design codes. Instead, it may be viewed as a chance to condense and modify building design to serve the new customers' needs.

The performance-based design is developed to circumvent the limits of force-based seismic design methodologies to address inelastic behavior and cyclic loading effects in reinforced concrete structure approach provides a realistic and reliable evaluation of the threat to life occupancy and economic damage that future seismic events may pose. The design has two main goals: effectively calculating the uncertainties associated with the performance assessment process and adequately characterizing the structural damage related to it for direct incorporation into the design or performance evaluation approach (Zameeruddin & Sangle 2016).

Due to technological developments, computing capabilities, and building materials. This type of design or safety assessment is defined as a deformation-based design if the principle in design and evaluation ensures that the expected deformation of concrete and reinforcement under external loads in sections remains equal to or less than the limit value of the acceptable strain (evaluation). (Celep & Gençoğlu, 2009).

Using a performance-based design approach, buildings are made to function reliably at different earthquake intensities. Targeting the behavior of the systems in both the elastic and inelastic areas of deformation is crucial to achieving this. As a result, identifying the member strength hierarchy, failure mechanism, and structural strength are crucial components of a performance-based design approach (Leelataviwat et al., 1999).

Incipient deterioration and incipient collapse highlight the importance of undertaking a structural performance review. The minimal lifetime cost requirements are established for a variety of natural risks in order to achieve the highest goal dependability for performance-based design. Stochastic loads address structural redundancy in addition to demand vs. capacity uncertainties, structural configuration, ductility capacity, 3-D motions, and structural configuration. A uniform-risk redundancy factor is proposed to offer constant dependability for structural systems with various levels of redundancy.

It entails a significant number of probabilistic concerns in its purest form, including variability in seismic input, material qualities, dimensions, and gravity loads, financial consequences connected with damage, and collapse or loss of use following a seismic assault (Taranath, 2009).

A structure's performance is divided into four areas Fully Operational, Operational, Life-Safe, and Near Collapse are the four options. These performance levels must be chosen to match various design earthquake levels, which vary depending on their frequency. The SEAOC proposes four earthquake design levels: 1) frequent earthquakes, 2) occasional earthquakes, 3) uncommon earthquakes, and 4) infrequent earthquakes. The easiest way to summarize these performance goals is to see in Figure 2.2. ((Leelataviwat et al., 1999).

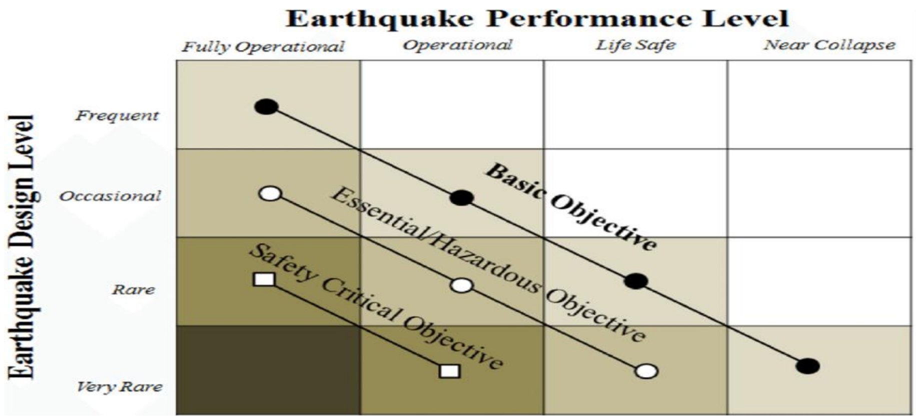


Figure 2.2 Performance Objectives for buildings (Leelataviwat et al., 1999).

The performance-based analysis is only used right now to evaluate the efficiency of existing buildings. However, it's excellent that it will soon be utilized to build structural systems. Following that, a structure will be built in cooperation between structural engineers and investors. The investor will specify their claim during the design phase.

2.2.4 Acceptable Level of Risk and Performance

The ISDCTB-2008 study on expected damages in three different earthquake levels provides the following breakdown of tall structure performance levels. The first three letters of the alphabet are (E1, E2, and E3). The acceptable damage limits for such performance levels should be quantitatively calculated separately for each structural type or element. These are the many degrees of performance:

- Minimum Damage (Uninterrupted Occupancy) The Performance Level depicts a state where no structural or nonstructural damage occurs in tall structures and their elements due to an earthquake. If damage does occur, it is very restricted.
- Damage that can be controlled (Life Safety) Under the effects of an earthquake, a Performance Level specifies a performance state in which minimal and repairable structural and nonstructural damage to tall structures and their elements is tolerated.
- Damage to a large extent (No-collapse Safety) A performance level specifies a state in which considerable damage to tall structures and their elements may occur due to an earthquake before the building collapses (Binzet et al., 2014).

Figure 2.3 illustrates performance ranges, which are the regions in between the aforementioned performance levels. The area below the (MD - UO) Performance Level is known as the minimum Damage / Uninterrupted Occupancy Performance Range. The area between the (MD - UO) Performance Level and the (CD - LS) Performance Level is known as the controlled Damage / Life Safety Performance Range. The area between the (CD - LS) Performance Level and the (ED - NC) Performance Level is known as the Extensive Damage / O-collapse Safety Performance Range. The minimal performance requirements defined for tall structures are also shown in Table 2.1. (Binzet et al.,2014).

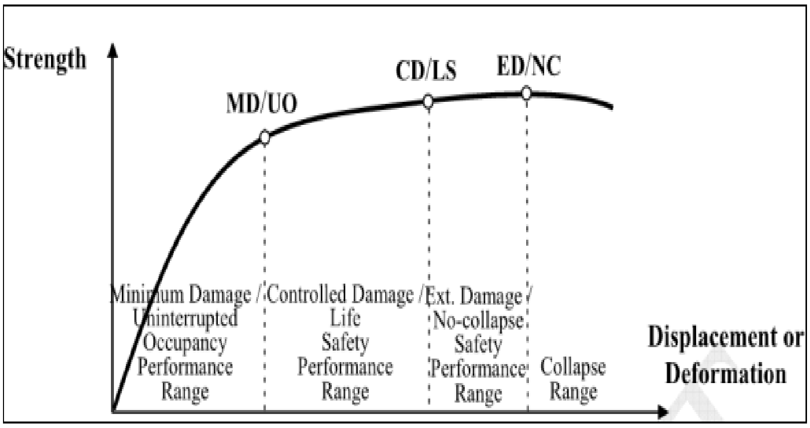


Figure 2.3 Performance levels and regions defined in ISDCTB -2008.

Design based on performance starts with the selection of design criteria that are related to the performance levels. Each performance level indicates the danger of causing various amounts of harm and indirect losses, which might be repercussions. The failures might be structural or non-structural, and they can be measured in lives lost, direct expenses, or service costs. (Binzet et al., 2014).

Table 2.1 Building class minimum performance objective relationship. Binzet et al., 2014).

Building Occupancy Class	(E1) Earthquake Level	(E2) Earthquake Level	(E3) Earthquake Level
Average occupancy class (residence, hotel, office building, etc.)	MD/UO	CD/LS	ED/NC
Particular occupancy class (health, education, public admin., Building, etc.)	-	MD/OU	CD/ KH

2.3 Design of High-Rise Structures

The design of a high-rise structure, like other designs, entails the creation of a physical description of an item while adhering to a set of limitations and standards. A high-rise structure's conceptual, preliminary, and detailed design phases are first, second, and third. Conceptual design involves identifying concepts and selecting the best subsystems and their configurations. The initial design phase begins with creating more abstract. Finally, the detailed design step creates final drawings for architectural, structural, electrical, and mechanical systems, defining a complete solution for all subsystems (Khajehpour, 2001).

High-rise building investment projects. Several nations are attempting to advance by encouraging the development of comprehensive plans to build high-rise investment projects to boost their status and economic power. Funding such programs is a critical component of their success in Malaysia, Hong Kong, the United States, the United Kingdom, and Japan. After thorough feasibility studies are completed, various and substantial investments are made to ensure that such projects get the desired results for investors at the state and economic levels. These studies' architectural, planning, marketing, and financial components have been thoroughly investigated. Planning, economic, and urban growth contribute to a country's success; it is the most crucial factor that fosters technical innovation by requiring the most up-to-date methods and materials. All of these elements contribute to the country's ability to attract money. With the turn of the century, several countries began to make headway by developing comprehensive plans for high-risk investment projects and several principles and criteria to ensure their success. Most Arab Gulf states, Hong Kong, and Malaysia have started similar procedures to improve the country on multiple levels, with feasibility studies playing a vital part in examining all aspects and circumstances that impact the project and the level of success of the investing firms. With its four-story timber residential structures, high-rise buildings began in ancient Rome. Then, brick units are used to construct such house buildings. The Monadnock Building, a sixteen-story structure in Chicago built in the nineteenth century using the loadbearing wall construction method, is completed in 1891. Facilities increased height as construction processes improved, reaching 60 stories in 1913 with the Woolworth Building in New York. Throughout history, high-rise buildings have remained appealing to builders. Because of their height, clarity, and dominance over other landscape components, high-rise structures have a special significance and visibility in the built environment (Farouk, 2011).

A tall building, as defined by Engineering Design Consultant (EDC), has a height of 35 meters or more, divided into reachable stories at regular intervals. The tower must be elevated above the ground and constructed with care to reach its maximum height in order to qualify as a tall construction. A high-rise building (tower) is a structure with 6 meters or more than 12 floors that are utilized for business, housing, or lodging. With the exception of height, it is always a relative matter. It is difficult to adequately identify high-rise structures due to the fact that the building's scale expression depends on the surrounding environmental elements. However, from a structural perspective, it may be described as a building whose height is significantly affected by side loads brought on by earthquakes and wind, to the point where these loads are taken into account during design (Farouk, 2011).

A five-story structure is once considered a high-rise building; however, Burj Dubai, with its 818 m height, is now considered a high-rise skyscraper. A tall building is defined as 60 meters or above in size, according to IYBDY regulations.

The development of technology and machinery and the production of high-strength concrete and steel, and innovative design methodologies all contributed to the construction of tall structures.

2.3.1 Istanbul's Earthquake-Resistant High-Rise Building Code

High-rise structures have exploded in Istanbul. There is no distinct rule for the seismic design of high-rise buildings in Turkey since the current Turkish Seismic Code does not contain design standards for large structures. In 2008, the Department of Earthquake Engineering at Kandilli Observatory and the Earthquake Research Institute drafted the "Istanbul Seismic Design Code for Tall Buildings (ISDCTB)" to specify a set of standards. The proposed research uses the performance-based design technique, initially introduced to the seismic engineering community in the early 2000s (Binzet et al., 2014).

The Istanbul Seismic Design Code for Tall Buildings (ISDCTB) is a three-phase procedure based on the PEER Performance-Based Design methodology. Tall buildings are classified as those that are more than 60 meters tall (Eroğlu, 2017).

The Istanbul Seismic Design Code for Tall Buildings-2008 (ISDCTB-2008) is based on a performance-based design in earthquake-prone locations. The damage that will occur in the elements of the structural system at different degrees of seismic ground motion is quantified and examined to see whether it exceeds the permitted damage limits in each component. The structure's performance objectives establish the permissible damage limits for various earthquake levels. Since the nonlinear deformations that occur beyond the elastic strain limits are commonly used to predict earthquake damage at the element level, the performance-based design approach is inextricably linked to nonlinear analytic methods and the deformation-based design idea. However, linear analytical methods are authorized in the Code in the context of a strength-based design approach for performance objectives when minor damage is predicted (Binzet et al., 2014).

The ISDCTB -2008 and ASCE41 documents specify all acceptance criteria regarding deformation and force demands on specific structural components. Global demand characteristics like story drifts are also critical indicators of nonstructural component degradation and overall building performance (Binzet et al., 2014).

In the shear walls for flexure, link beams, and outriggers, the nonlinear response is allowed in a limited range. Force-controlled activities are assumed to be shear in the core walls and axial force on the lateral columns. Axial strain in core walls, outriggers, and link beam rotation, on the other hand, are all expected to be deformation-controlled processes (Binzet et al., 2014).

When exposed to the Maximum Considered Earthquake ground motion intensity, the structure's performance objective is specified under Collapse Prevention Performance Levels in the Istanbul Seismic Design Code for Tall Buildings ISDCTB 2008 document. On the other hand, the Collapse Prevention Limit is described by ASCE41 as ensuring a minimal danger of partial or whole building collapse by restraining structural deformations and stresses until severe strength and stiffness degradation begins (Binzet et al., 2014).

2.3.2 The Design Spectrum for Earthquakes

The Earthquake Risk Maps provide spectral characteristics based on the geographical location. The local ground soil conditions are considered while modifying these spectral characteristics. These parameters are used to calculate the Design Acceleration Spectrum. These spectral curves calculate the maximum earthquake loads, displacement, and internal forces (Güler & Celep, 2020).

They incorporate the Yi design spectrum's characteristics of local soil conditions and seismicity. Changes in the range can impact the target displacements of structures. The predicted building performance and damage are more accurate representations of the values of the systems that are not removed as planned. Correctly understanding the region's local soil qualities and seismicity is essential for building design and assessment. The Turkish Earthquake Hazard Map Interactive Web Application (TEHMIWA) has been available since the beginning of 2019 to estimate earthquake parameters required in structural studies for any geographical region. This web tool may generate these characteristics by choosing the region's local ground conditions and the earthquake ground motion level of interest (Işık et al., 2020).

The ground movements are directly connected to the seismic forces that endanger the buildings. The design spectra are one of the most significant parameters for identifying these forces. The earthquake hazard map must be updated, including data for the design spectrum derived from seismic zoning. As a result, ground motion seismic zoning studies are key indications in estimating seismic risk. In this case, employing micro zoning and site-specific seismic risks studies, the impacts of ground motion on structures and the goal displacements predicted from the system may be derived more realistically. Micro zoning studies separate the territory into sub-regions, allowing for more efficient planning and tactics to reduce seismic damage. With the new Turkish Earthquake Hazard Map, microseismic zoning has begun to be implemented based on technical advancements, economic possibilities, and scientific methods. This map may now be used to do site-specific seismic hazard evaluations for any geographic area (Işık et al., 2021).

- **Concrete C40/50**

  Specified concrete compression strength: $f\ 'c$ = 40 MPa

  Weight per unit volume: 25 $kN\ m$3

  Modulus of elasticity:E = 35000 MPa

  Poisson's ratio:υ = 0.2

  Shear modulus: G = 14580 MPa

- **Steel A615Gr60**

  Minimum yield strength: fy = 413.7 MPa

  Minimum tensile strength: fu = 620.5 MPa

  Expected yield strength: fye = 455.1 MPa

  Expected tensile strength: fue = 682.6 MPa

  Weight per unit volume: 76.97 $kN\ m$3

  Modulus of elasticity:E = 199948 MPa

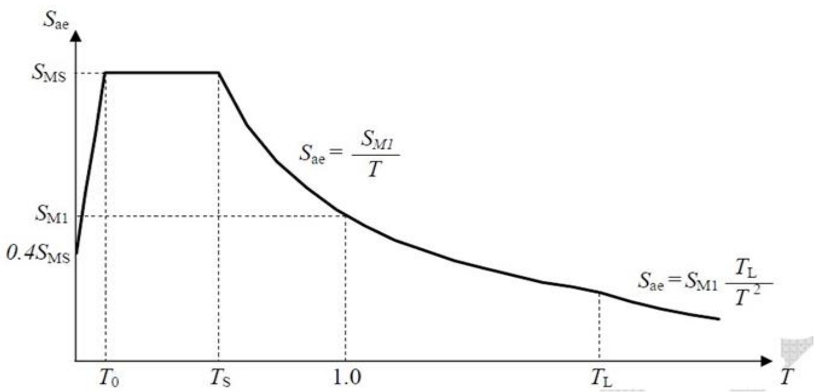


Figure 2.4 Elastic design spectrum curve according to IYBDY.

Sm1: Spectral Acceleration of 1 sec Period.

Sms: Spectral Acceleration of short period.

T: Natural seismic period.

T0: Period.

TL: Long Period.

Ts: Short Period.

An extended duration is defined as TL= 12 sec in Istanbul Province. Ta and Tb are supplied in DBYBHY according to the ground type. In IYBDY, the ground kinds are A, B, C, D, E, and F, although it is divided into Z1, Z2, Z3, and Z4 types. Figure 2.4 shows the elastic design spectrum curve according to IYBDY. In Eq.s (2.1) and (2.2), the formulas are presented (2.2).

$$S_{MS} = F_a \times S_s$$

$$S_{M1} = F_v \times S_1 \tag{2.1}$$

$$T_S = \frac{S_{M1}}{S_{MS}} \quad ; \quad T_\circ = 0.2T_S \tag{2.2}$$

2.3.3 Performance levels of tall buildings

Building Performance Levels for earthquake-resistant structural systems to TBDY-2018

- Immediate Occupancy performance level (IO-KK)

The building carrier system components have experienced no structural or minor damage at this performance level.

- Limited Damage Performance (LD-SH)

This performance level corresponds to the damage level where only minor damage to the building's structural parts occurs, i.e., limited nonlinear behavior.

- Controlled Damage Performance (CD-KH)

Damage that isn't too severe and can typically be remedied in the building's structural sections to ensure life safety is classified as this performance level.

- Collapse Prevention Performance (CP-GO)

This performance level refers to a condition before failure where the structural elements have been severely damaged. A partial or complete collapse of the structure is averted.

CHAPTER 3

3. ANALYSIS AND DESIGN CODES OF TALL BUILDING

3.1 Introduction

Wind and earthquake stresses must be taken into account when designing tall structures. When evaluating dynamic earthquakes, the conditions are different than when evaluating wind loads. The height of the structure, seismic zone, vertical and horizontal irregularities, soft and weak stories, and seismic zone are all factors that require dynamic analysis for earthquake load, according to the Bureau of Indian Standards' IS 1893(Part 1):2002. Higher mode effects are taken into account when determining the lateral pressure distribution and the height of the structure. When wind interacts with a building, there are both positive and negative stresses present. To prevent wind-related construction failure, the building must be strong enough to sustain the loads exerted by both pressures, according to IS 875(Part 3):1987. Wind pressure is influenced by the actual wind speed, topography, building height, internal pressure, and structural geometry. The structural system receives the load delivered to the building envelope and transfers it to the earth via the foundation (Raju et al., 2013).

Two calculation approaches are utilized in designing and assessing the structural system under horizontal and vertical loads (Akçora, 2020):

- **Linear Calculation Methods:**

This technique considers the carrier system to function linearly against horizontal and vertical loads. Although the carrier system's solution is assumed to be linear, some coefficients are included to account for the system's non-linear behavior. The linear system solution is more straightforward to calculate than the nonlinear system. Linear analysis is conducted with the earthquake load reduction coefficient Ra in this calculation approach, which considers the structure's non-linear behavior.

The following are the assumptions made while utilizing linear calculating methods:

- The material is supposed to act linearly and elastically.
- Second-order effects are not taken into account. This is because displacements have a negligible influence on equilibrium and continuity relations.
- Reaction forces are bidirectional, and the sections of structural elements do not change due to loads.
- The superposition principle is used.

➢ **Non-Linear Calculation Methods:**

Except in a few circumstances, buildings usually exhibit linear behavior when subjected to service loads. The load-bearing capacity of the building is approaching due to an increase in external factors impacting the load-bearing systems of the buildings. Strain and stresses diverge from linear-elastic behavior in this scenario. The displacements are not insignificant at this point, as they are in the linear calculation technique. Using linear computation methods is unlikely to match real-world behavior in this scenario. The behavior of the structural system under loads may be monitored more realistically by employing non-linear calculation methods that take into consideration the linear-inelastic capabilities of the materials, allowing for the design of more cost-effective structural systems.

The following are the two primary reasons why structural systems do not behave linearly under load:

- The stress-strain constitutive Equations are not linear because the carrier system's material is not linear-elastic.
- Geometrical deformation causes non-linearity in the equilibrium Equations.

3.2 High-Building Carrier Systems

Different carrier systems have been developed in high-rise buildings due to advancements in material technology from the past. When the high buildings built in the past are examined, it is clear that they are primarily made of masonry. Carriers made of iron, steel, or reinforced concrete have been developed due to advancements in material technology. Composite structures are created after that, combining steel and reinforced concrete. The cross-sections of structural system elements have lowered due to improvements in concrete and steel, and higher structures have been constructed.

Building load-carrier systems must be sufficiently rigid horizontally to withstand earthquakes, winds, and vertical loads. The horizontal wind and earthquake loads grow dramatically as the height of the building increases. As a result, novel structural system designs that provide appropriate horizontal strength have been created. (Günel & Ilgin, 2014).

Different carrier systems can be utilized in buildings depending on their height and architecture. Most high-rise buildings have basements with low-rise above-ground levels and tower parts above them. To fulfill the load-carrier systems of today's high-rise structures and the horizontal loads on the network, core shear wall systems are utilized in the center portion of the plan. The stairwell and elevator are usually surrounded by these core bulkheads.

Different carrier systems are chosen as the building's height grows to limit the structure's displacement under horizontal loads. Carrier systems in reinforced concrete high-rise structures can be divided into five categories. These;

1. Rigid Frame Systems
2. Braced Frame And Shear-Walled Frame Systems
3. Outrigger Systems
4. Framed-Tube Systems
5. Braced-Tube Systems
6. Bundled-Tube Systems

Except for 'outrigger systems,' all of the structural systems listed above may be used in reinforced concrete owing to breakthroughs in concrete technology, such as creating ultra-high-strength concrete.

The need for tall building projects made of composite and concrete is predicted to increase shortly. Tall buildings will also be introduced with "mixed systems" and "bundled systems" structural categories. 'Mixed systems' integrate two or more of the six abovementioned six categories under systems,' on the other hand, utilize bundled structural systems, such as the bundled shear wall system used in the Burj Dubai, also known as a 'buttressed core system (Gunel & Ilgin, 2007).

Depending on the number of stories and the height of the building, these technologies are employed to strengthen the horizontal stiffness of the carrier system (Figure 3.1). On the other hand, the number of stories is crucial when choosing a carrier system.

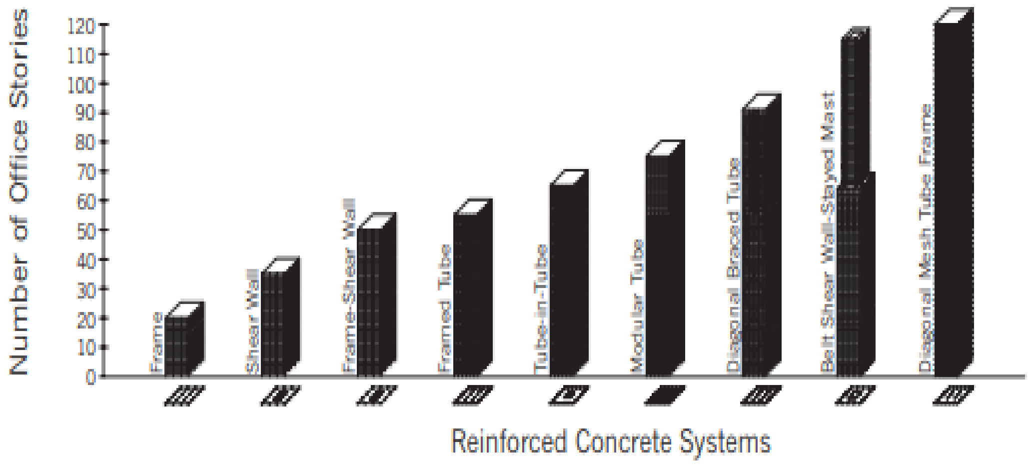


Figure 3.1 Structural systems based on the number of stories (Sarkisian, 2016).

3.2.1 Rigid Frame Systems

Both steel and reinforced concrete construction employ rigid frame technologies. The building design has long acknowledged rigid frame systems for their ability to resist lateral and vertical loads. In order to retain the original angles between the crossing components, stiff framing, or more particularly moment framing, is based on beam-to-column connections that are sufficiently stiff. The rigid frame is appropriate for reinforced concrete structures because of the joist's inherent rigidity resulting from its natural desire to form monolithic structures. In contrast, the wooden construction is modified by making the joints more robust in order to retain enough rigidity in steel structures (Gunel & Ilgin, 2007).

The strength and stiffness of a rigid frame are related to the sizes of its beams and columns and inversely proportional to the distance between those columns. The placement of columns is done so as to cause the least amount of design disruption while yet allowing for a shallow narrative level. The building's façade needs deep beams and well placed columns to effectively frame activity. Buildings constructed in seismic zones should pay particular attention to the design and details of the joints since stiff frames are more flexible and less resistant to solid earthquakes than steel braces or shear-walled constructions (Gunel & Ilgin, 2007).

Frame action in buildings up to 30 stories typically involves lateral resistance, except in delicate constructions. For skyscrapers with more than 30 stories, the structural system often cannot withstand lateral sway produced by wind and seismic events.

3.2.2 Braced frame and shear-walled frame systems

Rigid frame solutions are inefficient for constructions taller than 30 stories because the lateral displacement generated by column bending causes the drift to be too severe. Steel bracing or shear walls with or without rigid frame (brace systems and shear wall systems), on the other hand, improve the overall stiffness of the structure, and the resulting system is known as a braced frame or shear-walled frame system. Steel bracing systems, shear walls, and systems that interact with rigid frames might all be considered upgrades to rigid frame systems. These systems are more rigid than rigid frame systems and can support buildings up to 30 stories tall, while they are most often used for constructions up to 50 stories tall. However, these systems have been known to reach nearly 100 stories. Both reinforced concrete and composite construction employ a sheer-walled frame system, while steel construction uses a braced frame system (Gunel & Ilgin, 2007).

- **Braced frame systems**

Steel structures employ braced frame systems. This method offers a low-cost, high-efficiency option for horizontal stress resistance. Extra bracings are utilized to increase a rigid frame's efficiency by almost eliminating column and girder bending. It has a construction comparable to a vertical truss. It consists of gravity-bearing columns and beams as well as diagonal bracing elements. To withstand the horizontal load, the whole set of elements forms a vertical cantilever truss (Gunel & Ilgin, 2007).

Based on architectural and structural characteristics, braces may be classed into four groups depending on the hook, based on architectural and structural features: 1. the most common bracings are X, diagonal, K, and knee bracings. Where frame diagonals may be contained inside permanent walls, and the opening requirements often determine the bracing pattern, the braces are ideally positioned around elevators, stairs, and service shafts.

Braces are used to protect the world's most iconic buildings from falling laterally, notably New York's 77-story-high Chrysler Building (1930) and 102-storey-high Empire State Building (1931).

- **Shear-walled frame systems**

Shear-walled frame systems are used in both reinforced concrete and composite construction. Vertical cantilevered beams that endure lateral wind and seismic pressures imparted to a structure and transmitted to them by the story diaphragms are known as shear walls. Shear walls are often used in elevator and service cores and frames to stiffen and strengthen the structure. These components may be round, curvilinear, oval, box-like, triangular, or rectilinear shapes. This system functions as a concrete structure with shear walls that withstand lateral loads. This style is well exemplified by the 68-story Metropolitan Tower in New York, completed in 1987. The 88-story-high Petronas Towers (1998), the world's highest building until Taipei 101-story-high in 2004, employed this approach in composite construction. (Gunel & Ilgin, 2007).

3.2.3 Outrigger Systems

In steel and composite constructions, outrigger systems are a mix of braced and shear-walled frame systems. The outrigger system is a structural system that consists of a central core with braced frames or shear walls, as well as horizontal "outrigger" trusses or girders that connect the center to the outer columns. Furthermore, the exterior columns are usually connected by outside belt girders. If the structure is subjected to horizontal tension, the column-restrained outriggers prevent the body from rotating. The outriggers and belt girders should be at least one and often two stories deep to ensure enough stiffness. Consequently, they're usually installed at the plant level to reduce the number of obstacles they create (Gunel & Ilgin, 2007).

When compared to single-story outrigger buildings, multi-story outrigger structures provide higher lateral resistance and, as a result, structural efficiency. Each new outrigger story on the other hand, reduces lateral stiffness compared to the previous one (Gunel & Ilgin, 2007).

Outrigger structures may be required for constructions with more than 100 storeys. The 42-storey-high First Wisconsin Center, completed in 1974, is an excellent example of this method. Depending on the number of tiers of outriggers and their stiffness, the perimeter columns of an outrigger structure perform a composite behavior with the core (Gunel & Ilgin, 2007).

3.2.4 Framed-Tube Systems

In exceptionally tall buildings with braced frames and shear-walled frame systems, framed-tube systems become isteful. As a result, the framed tube is an alternative. Closely spaced perimeter columns connected by deep spandrels identify a tube, enabling the whole structure to serve as a giant vertical cantilever to resist overturning forces (Gunel & Ilgin, 2007).

It's an excellent way to provide lateral resistance, whether you have inner columns. Many stiff joints that function along the tube's edge, resulting in a large tube, contribute to the system's efficacy. The outer tube carries the full lateral load. The gravitational stress is shared by the tube and any interior columns or walls, if present. Apart from its structural efficiency, framed-tube constructions offer an internal story plan typically devoid of core bracing and heavy columns, resulting in more net usable story space due to the perimeter framing system that carries the whole lateral load. The densely arranged surrounding columns, on the other hand, may hinder views from the interior of the building (Gunel & Ilgin, 2007).

Because of the rectangular window arrangement, the most common approach for achieving tubular behavior is to utilize columns on close centers linked by a deep spandrel. There are two standard versions of this composite construction method: one uses composite columns and concrete spandrels, while the other employs structural steel spandrels instead of concrete spandrels.

The height-to-width ratio, plan dimensions, spacing, and size of columns and spandrels of the buildings all impact the system's efficiency. The tube form is created for rectangular or square constructions, and it is probably the most cost-effective of those forms. Still, it may also be utilized in circular, triangular, and trapezoidal designs.

The three types of framed-tube systems are as follows:

1. Systems without interior columns, shear walls, or steel bracings;
2. Systems with interior columns, shear walls, or steel bracings;

3. Tube-in-tube systems.

When lateral wobbling becomes an issue and determines the design, the "framed tube" may be reinforced with a tube in the core to create a "tube-in-tube" system that can be developed up to 100 stories tall. The World Trade Center Twin Towers (1972), which stand 110 stories tall, is an exceptional example of framed-tube technology.

3.2.5 Braced tube systems

Braced-tube systems may benefit steel, reinforced concrete, and composite construction. By adding multistory diagonal bracings to the tube's face, the stiffness and efficiency of the framed tube may be improved. Consequently, the braced-tube system that results from a trussed tube or outside the diagonal-tube system may be employed for taller ceilings and broader column spacing. It achieves an excellent result by using a small number of diagonals on each face of the tube, which meet at the same point as the corner columns. Steel structures make use of steel diagonals/trusses. Reinforced concrete structures create diagonals by filling window openings with reinforced concrete shear walls to provide the same effect as diagonal bracing (Gunel & Ilgin, 2007).

The first reinforced concrete structure to adopt this design is New York's 50-story-high 780 Third Avenue Building (1985). Another example of a concrete system is in Chicago's 58-story-high Ontario Center (1986).

On the other hand, the bracing ensures that the perimeter columns support both gravitational and horizontal wind forces. Consequently, a very stiff cantilever tube is formed, with lateral stress behavior comparable to that of a pure rigid tube. Another good example of composite construction is the 100-story-high John Hancock Center (1969). This construction is first used in a steel skyscraper and is ideally suited for tall, thin structures with small story surfaces.

A braced tube lowers the risk of excessive axial stress on the corner columns. In this regard, strengthening the exterior frames may tackle one of the main issues in the framed tube. Although substituting vertical columns with closely spaced diagonals in both directions is the most effective braced-tube action, the braced-tube system is not often used owing to concerns over shear wall features. For constructions with more than 100 stories, this strategy may be employed (Gunel & Ilgin, 2007).

3.2.6 Bundled Tube Systems

Bundled-tube systems are beneficial to steel, reinforced concrete, and composite construction. When the building proportions increase in height and width, a single framed tube loses structural efficiency. In particular, the wider the structure in the design is, the less accessible the tube is. A bundled tube, also known as a modular tube with more massively spaced columns, is used in these cases. The need for vertical modulation prompted this design, which consists of a cluster of tubes combined with standard interior panels to generate a perforated multi-cell tube. Many story patterns may be constructed by simply ending a tube at any desired height while preserving structural integrity since this system is made up of individual tubes. This feature allows for a variety of different forms and sizes of setbacks. It aids in the organization of asymmetrical bodies (Gunel & Ilgin, 2007).

The cells may be triangular, hexagonal, or semicircular since the "bundled-tube" architecture is created by arranging individual tubes. The negative is that a series of columns running the length of the construction divides the storying into tiny compartments. Because the columns are more massively separated and the spandrels are more minor, this configuration allows for more massive window apertures than the single-tube style. Additionally, since any tube module may be taken out whenever the interior space design necessitates it, this strategy makes the structure's architectural plan more versatile.

3.3 Design of a High-Rise Building According To TBDY 2018

3.3.1 Introduction

In recent years, Turkey has seen an increase in the design and construction of high-rise structures. However, during the design phase of these buildings, a prevalent concern is how these structures would continue to serve occupants after a significant earthquake. During the design phase, many issues arise. The structural system must deal with two significant difficulties as the building's height rises. The wind is one of them, and an earthquake is the other. With the invention of dynamic isolation technology and one-by-one model building tests, design engineers could win the battle against these two challenges to some extent. However, little care is given to how the structures will continue to serve after the earthquake and how the investor will represent these. There is various research on post-earthquake building usage performance studies, notably in recent years. The structural structure of all high and low buildings in Turkey is traditionally reinforced concrete shear walls and frames. In Turkey, a new earthquake law went into effect on January 1, 2018, regulating the design of tall structures in an earthquake (TBDY 2018). Aside from that, the worldwide resource "Tall Buildings Initiative" (TBI) from 2010 is employed. These laws mandate performance-based design approaches for tall structures and nonlinear time history analysis to design the structural system (Dedeoğlu & Zülfikar, 2020).

When we look at the building stock in Turkey and the new buildings being built, we can see that there is a lot of activity. Most structures that meet the criteria are constructed to offer life safety rather than avoid injuries, reduce damage, or assure quick regeneration. For example, under the new earthquake rule, "Controlled Damage" performance at the DD-2 level, often known as "design earthquake," is needed. Controlled damage is defined as a degree of damage that is not too severe and generally repairable to protect life safety in the regulation. The design engineer's primary responsibility is to assure the building's life safety. People's expectations of an engineer, on the other hand, are significantly higher. For building owners, terms like durability, planning, preparedness, and post-disaster usage are becoming increasingly relevant. As a result, procedures are implemented to assess the status of the structures following the earthquake. The USRC Rating System, for example, estimates the building dwelling and fills in crucial gaps to help persons who use the form, as well as planners, building owners, and insurers (Dedeoğlu & Zülfikar, 2020).

There are several design approaches and preferences available nowadays. Buildings made of reinforced concrete, towering structures made of steel, and composite (mixed) constructions are examples. Steel's speed of construction and strength, concrete's cost and fire resistance, and the usage of composite constructions, which may solve several issues at once, especially in towering buildings, all play a role in design preferences (Dedeoğlu & Zülfikar, 2020).

Advanced analytical methods and the structural system are used to study the buildings in the techniques above; for example, reinforced concrete shear walls are developed with perimeter beams and frame systems. Before establishing the building's analysis model, how to represent a reinforced concrete shear wall, beams, and columns on a system basis is thoroughly examined. These are referred to as conventional approaches, divided into two categories (Dedeoğlu & Zülfikar, 2020).

Tall structures are defined as those with a Building Height Class (BYS) of 1, according to TBDY 2018. For a building to be designated a tall structure, the law specifies height restrictions based on Seismic Design Categories (DTS). These are the restrictions (Akçora, 2020):

- For DTS=1, 1a, 2, 2a, structures having a height H_N > 70 m.
- For DTS=3, 3a, structures height H_N > 91 m.
- For DTS=4, 4a, structures having a height H_N > 105 m.

Before starting the design, it's essential to figure out what the building will be used for. The usage class of my structures is listed in (Table 3.1). Building necessary coefficients vary based on the building utilization class, as seen in this table. Using this coefficient, 30 critical structures are constructed to withstand more stresses and be more stable.

Table 3.1 Seismic Design Classes given in TBDY 2018.

Seismic Design Categories (DTS) S_{DS} (g)	Building Use Categories (BKS)	
	BKS = 1	BKS = 2, 3
SDS< 0.33	DTS = 4a	DTS = 4
$0.33 \leq S_{DS} < 0.50$	DTS = 3a	DTS = 3
$0.50 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

Table 3.2 Building height classification (BYS) given in TBDY 2018.

Building Height Categories (BYS)	Building Height Ranges Defined by Building Height Classes and Earthquake Design Classes [m]		
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a	DTS = 4, 4a
BYS = 1	$H_N > 70$	$H_N > 91$	$H_N > 105$
BYS = 2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
BYS = 3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
BYS = 4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
BYS = 5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
BYS = 6	$10.5 < H_N \leq 17{,}5$	$17.5 < H_N \leq 28$	
BYS = 7	$7 < H_N \leq 10{,}5$	$10.5 < H_N \leq 17{,}5$	
BYS = 8	$H_N \leq 7$	$H_N \leq 10{,}5$	

Table 3.3 The Importance Factor I given in TBDY 2018.

Building Usage Classification (BKS)	Building Usage Purpose	Building Importance Factor (I)
BKS = 1	1) Places that will be used immediately after the earthquake. Like Hospitals, firefighting buildings, PTT, Power stations, rescue stations, police stations, communication, operation centers, and structures containing highly toxic materials. 2) Schools, dormitories, jails, and military buildings. 3) Museums 4) toxic, blasting material containing buildings	1.5
BKS = 2	Shopping Centers, Sports, Complexes, Cinema, Theatre	1.2
BKS = 3	The building which is not included in the first and second categories like residence buildings, offices, hotels, industrial buildings	1.0

Building use is classified based on the significance attributed to the structure. BKS=1 is for essential imports such as hospitals and emergency rooms, schools, museums, and hazardous waste plants. BKS=2 refers to structures that temporarily house huge groups of people, such as music halls, stadiums, and shopping malls. BKS=3 refers to all other structures (Sucuoğlu, 2018).

The short-period spectral acceleration determines the seismic design category, which shows PGA at the site when divided by 2.5. The difference in the table between X and Xa is enforcing different standards in critical facility design (Sucuoğlu, 2018).

The design technique is chosen based on the building height categories.

3.3.2 Earthquake Effect Definition

The main effects of earthquakes are ground shaking, ground rupture, landslides, tsunamis, and liquefaction. Fired is unquestionably the most prominent quake-related side effect.

3.3.2.1 Earthquake Levels

- **Ground Motion Level 1 of an Earthquake (DD-1)**

With a 2% probability of achieving the spectral magnitudes in 50 years and a 2475-year repetition duration, the DD-1 Earthquake Ground Motion is a very rare earthquake ground motion. This earthquake's ground motion is also referred to as the one that is the most considerably regarded.

- **Ground Motion Level 2 of an Earthquake (DD-2)**

Rare seismic ground motion is described by the DD-2 code, which has a 475-year repetition time and a 10% probability of exceeding spectral magnitudes in 50 years. This kind of earthquake ground motion is also known as standard design earthquake ground motion.

- **Ground Motion Level 3 of an Earthquake (DD-3)**

Earthquake ground motion classified as DD-3 occurs often, with a 72-year recurrence period and a 50% risk of exceeding spectral magnitudes in 50 years.

- **Ground Motion Level 4 of an Earthquake (DD-4)**

With a 68 percent risk of exceeding spectral magnitudes in 50 years (50 percent in 30 years) and a 43-year recurrence time, earthquake ground motions classified as DD-4 are frequent. This kind of earthquake ground motion is also known as a service earthquake.

Table 3.4 Earthquake Ground Motion Levels given in TBDY 2018.

Earthquake Level	Repetition Period (Year)	Probability of Exceedance (in 50 Years)	Description
DD-1	2475	2%	Largest earthquake ground motion
DD-2	475	10%	Standard design earthquake ground motion
DD-3	72	50%	Frequent earthquake ground motion
DD-4	43	68%	Service earthquake movement

3.3.2.2 Earthquake Ground Motion Spectrum Definition

Using map spectral acceleration coefficients, local ground effect coefficients, a specific earthquake ground motion level, and a 5% damping rate, TBDY 2018 defines the earthquake ground motion spectrum. When a particular description of the spectrum is necessary, site-specific seismic hazard studies can be performed.

Turkey Earthquake Hazard Maps developed map spectral acceleration coefficients for four earthquake ground motion levels. The map spectral acceleration coefficients are computed using the reference ground conditions. By dividing the abovementioned map spectral acceleration values by the gravitational acceleration, dimensionless map spectral acceleration coefficients are produced.

Peak responses from several Single Degree of Freedom (SDOF) systems with different durations are collected in the typical earthquake ground motion response spectrum. The natural period of vibration and the best absolute acceleration that an SDOF system obtains from subsurface motion are connected by the ground motion acceleration response spectrum. The peak displacement of several SDOF systems with different periods relative to the ground, however, is frequently visible in a displacement response spectrum. As a result, investigating several SDOF systems is required in order to develop a response spectrum. The value of each point on the spectrum represents the highest response of a single degree of freedom system at a certain moment. There are five fundamental points that make up this spectrum. The Eq. shows that all recipes are connected to Ss and S1 eq.(3.1). These may be discovered by putting the building's latitude and longitude into the Turkey Earthquake Hazard map.

- Short period Spectral Acceleration factor
- sec period Spectral Acceleration factor

$$S_{DS} = S_S \times F_S \qquad\qquad S_{D1} = S_1 \times F_1 \tag{3.1}$$

Fs and F1 are local ground effect factors taken from Table 3.5 and Table 3.6 below.

Table 3.5 Fs Table is given in TBDY 2018.

Soil Classification	Fs Factor For Short Period Area Ss					
	Ss ≤ 0.25	Ss = 0.5	Ss = 0.75	Ss = 1	Ss = 1.25	Ss ≥ 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	Specific site soil behavior analysis should be done.					

Table 3.6 F_1 Table Is given In TBDY 2018.

Soil Classification	F1 Factor For Short Period Area S_1					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
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	Specific site soil behavior analysis should be done.					

According to TBDY 2018, the Elastic design spectrum curve is shown in (Figure 3.2). Eq.s (3.2) through (3.7) may be used to compute them.

$$S_{ae}(T) = \left[0.4 + 0.6 \times \frac{T}{T_A}\right] S_{DS} \qquad 0 \leq T \leq T_A \tag{3.2}$$

$$S_{ae}(T) = S_{DS} \qquad T_A \leq T \leq T_B \tag{3.3}$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad T_B \leq T \leq T_L \tag{3.4}$$

$$S_{ae}(T) = \frac{S_{D1} T_L}{T^2} \qquad T_L \leq T \tag{3.5}$$

$$T_A = 0.2 \frac{S_{D1}}{S_{DS}} \tag{3.6}$$

$$T_B = \frac{S_{D1}}{S_{DS}} \qquad T_L = 6s \tag{3.7}$$

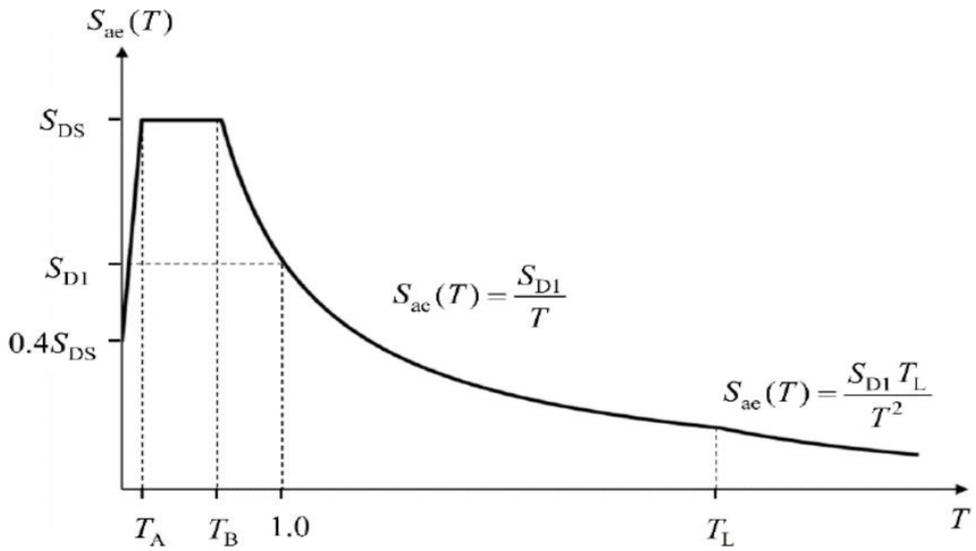


Figure 3.2 Elastic design spectrum curve is given in TBDY 2018.

Due to the enormous stresses that would result from a complete earthquake loading, the structural design is both uneconomical and unfeasible. The essential concepts of ductility and inelastic energy dissipation are not used. Each and every construction has dead and live loads. We promise that, in order to avoid substantial structural cracking under gravity loads, static demand will always be more than member yield capability. We also guarantee that plot vibrations won't exceed preset limits. However, seismic loading varies. Since the structure is designed to withstand incredibly severe events, we may let it shatter and move through the yield deformation. Planning a building that will stand up in the face of such a powerful earthquake is ludicrous. The building may not even experience the seismic stress that we planned for. Residential and commercial structures have a maximum life expectancy of 100 years, and substantial shaking occurs once every 500 years (in most regions).

The pushover curve for elastic and perfectly elastic displacement is given in Figure 3.3 and Figure 3.4 below.

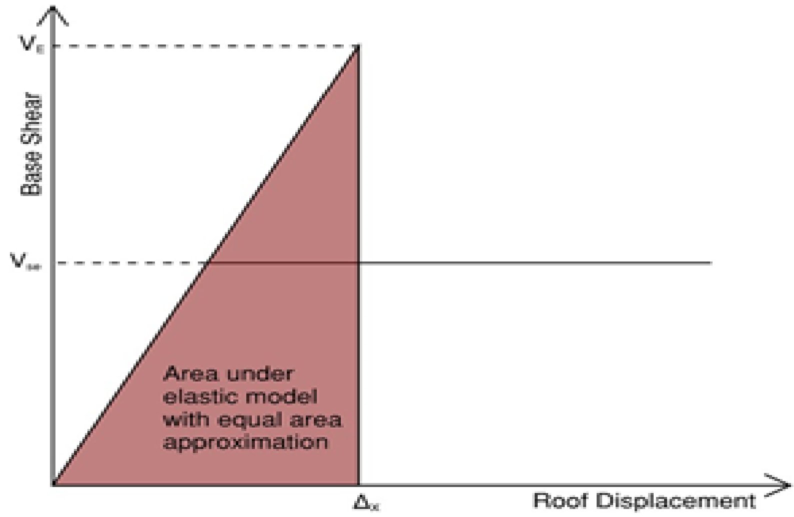


Figure 3.3 Push-over curve of elastic-perfectly plastic EPP.

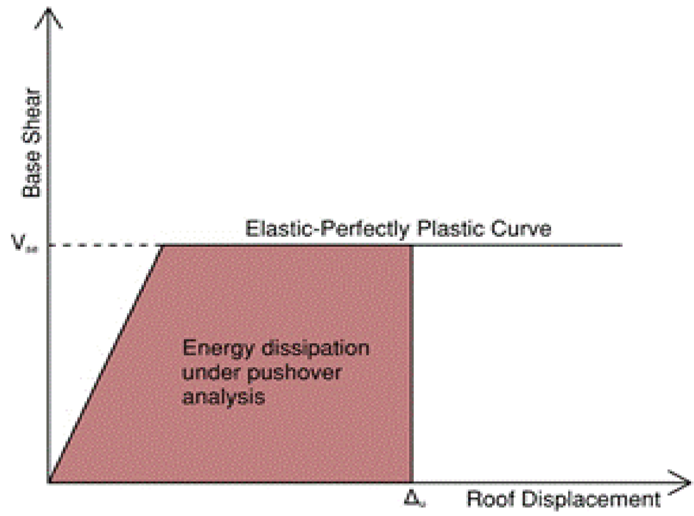


Figure 3.4 Push-over curve of elastic displacement.

Additionally, every 20 years, there is a large earthquake in places like Taiwan and Mexico. According to the second image, which shows a linear elastic model, the base shear must increase to accrue the same amount of energy as the inelastic model. Due to the linear elastic model not entering a plastic state, the external force increases as the displacement increases. (Remember that in linear analysis, force and displacement are always proportionate.) The area under the curve in this elastic model achieves the same value as the area under the bend in the EPP model at a certain force and displacement value. Ve is the base shear that corresponds to this value of Vse, the base shear at which the structure yields.

R factor is calculated using Eq.s (3.8), (3.9), and (3.10) given below.

$$R = \frac{V_e}{V_{se}} \tag{3.8}$$

$$R_a = \frac{R}{I} \qquad T > T_B \tag{3.9}$$

$$R_a(T) = D + \left[\frac{R}{I} - D\right]\frac{T}{T_B} \qquad T \leq T_B \tag{3.10}$$

3.3.2.3 Time Domain Definition of Earthquake Effect

Earthquake recordings should be chosen and scaled to produce the analyses as accurately as possible. The following are the guidelines for selecting and scaling earthquake recordings in TBDY 2018.

- When choosing earthquake recordings, special care will be taken to select suitable records for the design of seismic ground motion. The distance between the structure and the fault, the ground conditions, and the fault source mechanism will all be considered when selecting recordings. If there are previous recordings of earthquakes in the area, these data will be given priority.
- If identifying earthquakes under the conditions above is challenging, simulated ground motion recordings in the time past can be employed. The source mechanism, ground characteristics of the building, and wave dispersion will all be considered while constructing artificially manufactured earthquake ground movements. The parameters utilized for artificially manufactured earthquake records should be consistent with the natural earthquake characteristics encountered in the region.
- The number of records to be picked for one- or two-dimensional computations, that is, if only horizontal earthquake data are employed, will be at least eleven.
- At least eleven earthquake record sets should be chosen for three-dimensional calculations, which consider the horizontal and vertical components of the earthquake.

- There are guidelines for two alternative techniques of scaling earthquake recordings in the regulation. These are basic spectral matching scaling and scaling algorithms. The scaling method's rules are listed below.

- In the event of a one-dimensional or two-dimensional computation, the amplitudes of all records between 0.2Tp and 1.5Tp will be scaled so that they cannot be less than the amplitudes in the same period 37 range in the design spectrum.
- The square root of the sum of the squares of the spectra of the horizontal component of the selected earthquake recordings will be taken in the case of a three-dimensional computation, and the resulting spectrum will be compared with the range described for the earthquake ground motion. The amplitudes of the resulting spectrum between 0.2Tp and 1.5Tp will be scaled to be at least 1.3 times those of the scope described for earthquake ground motion of the same amplitude. The scale coefficient produced will be applied to horizontal and vertical earthquake recordings.
- Earthquake data can also be resized to make them spectral-compatible. The spectrum averages should not be less than the design spectrum ordinates if this approach is utilized. Programs are now used to convert earthquake recordings to offer spectral compatibility. Applications such as SEISMOMATCH (2018), SAP2000 (2019), and ETABS can be used to scale earthquake recordings (2019).

3.3.3 Material Identification

Concrete and steel material models used in reinforced concrete structures are defined in the regulation.

3.3.3.1 Models of Concrete

A concrete model is established in the rule to be utilized in design and assessment based on deformation. Figure 3.5 depicts the stress-strain diagram for wrapped and uncoated concrete, considering the concrete models and covering effect. (Cao, Y. G., Jiang, C., & Wu, Y. F. (2016)).

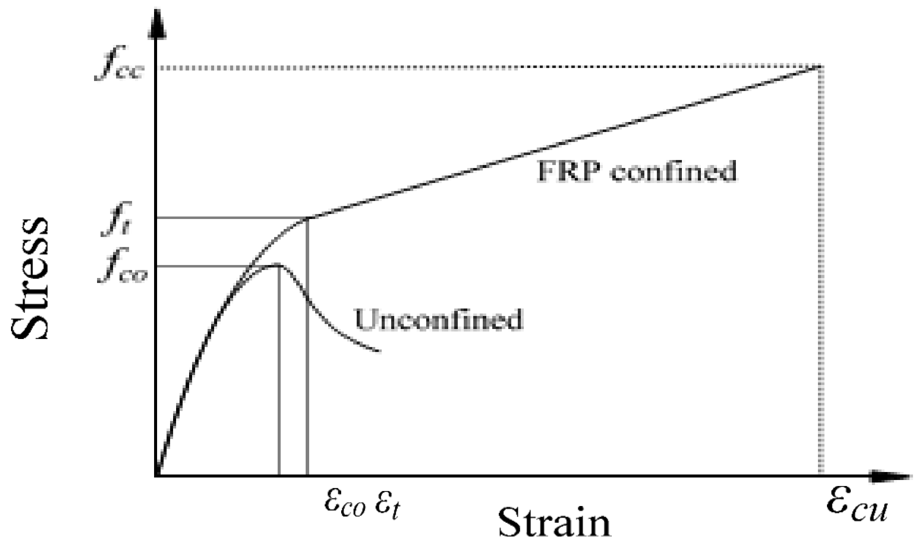


Figure 3.5 The typical stress-strain curve for FRP-confined concrete with strain hardening. (Cao, Y. G., Jiang, C., & Wu, Y. F. (2016)).

Figure 3.5 shows that when the wrapping effect of the concrete rises, the shear strain of the wrapped concrete increases. It is defined as fc=0 when it reaches εc=0.005 in the unwrapped concrete model.

3.3.3.2 Models of Reinforced Steel

The relation eq.(3.11) is used to establish the reinforcing steel model, which is then employed in the design and assessment of the strain. Figure 3.6 depicts the developed model's stress-strain diagram. At the same time, Table 3.7 contains information on steel.

$$
\begin{aligned}
f_S &= E_S\,\varepsilon_S \quad , && (\varepsilon_s \le \varepsilon_{sy}) \\
f_S &= f_{sy}\,e^{x} \quad , && (\varepsilon_{sy} < \varepsilon_s \le \varepsilon_{sh}) \\
f_S &= f_{Su} - (f_{Su} - f_{Sy})\frac{(\varepsilon_{su}-\varepsilon_s)^2}{(\varepsilon_{su}-\varepsilon_{sh})^2} && (\varepsilon_{sh} < \varepsilon_s \le \varepsilon_{su})
\end{aligned}
\tag{3.11}
$$

Table 3.7 Information about reinforcement steel given in TS498.

Class	*f*sy (MPa)	*εsy*	*εsh*	*εsu*	*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

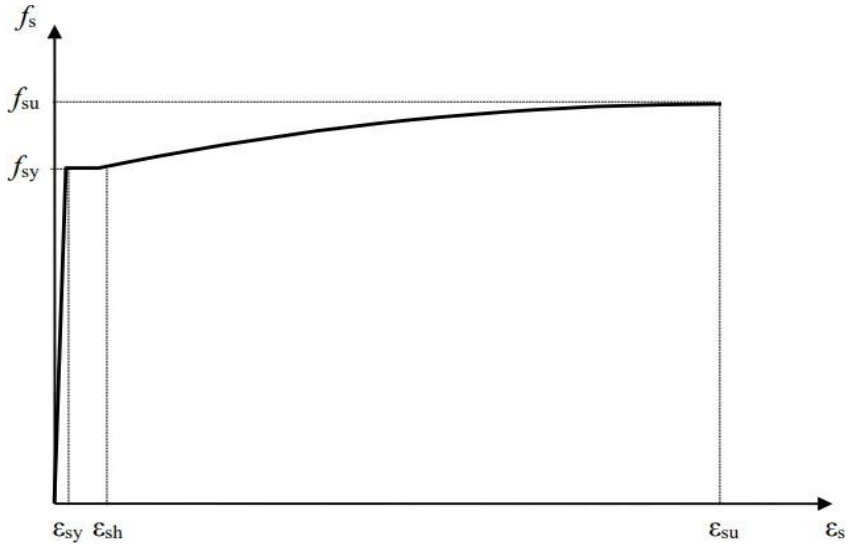


Figure 3.6 Stress-strain diagram of reinforced steel. (Celep, Z. (2017))

3.3.4 Tall Building's Performance Goals

Even though the 2018 Code's fundamental design process is force-based, the design aims are expressly established by means of building performance levels. Below is a summary of them (Sucuoğlu, 2018).

Several performance objectives are necessary during high-rise structures' design and evaluation phases. The rule divides performance levels into four categories.

- **Continued Operation Performance (CO):** There is no damage to structural elements (hairline cracks in concrete)
- **Limited Damage Performance (LD):** Minimal inelastic behavior due to minor damage to structural components.
- **Controlled Damage Performance (CD):** There is severe damage to structural elements, but it is repairable.
- **Collapse Prevention Performance (CP):** Although structural components have been severely damaged, the building's partial or entire collapse has been avoided.

At particular seismic levels, performance objectives for high-rise structures should be reached. High-rise building design and an appraisal are done in three phases. In the first step, a controlled damage performance level under the design earthquake, i.e., at the DD-2 seismic level, should be given. The building's relevance coefficient is used to determine the performance level in the second phase. The DD-4 earthquake level is employed for the usual performance objective to assure the building's continuous use. The DD-3 earthquake level should be utilized for the advanced performance goal based on the restricted damage performance level. In the second analysis phase, linear analytic methods are allowed for the average performance objective, but only nonlinear calculation methods are qualified for the advanced performance target. Finally, the performance level of prevention of resolution is necessary for the regular performance goal at the DD-1 earthquake level. A controlled damage performance target is required for the advanced performance level, depending on the importance of the structures. Table 3.8 lists these concerns.

Table 3.8 Performance Objective and Analysis Phases for Tall Buildings (Celep, 2018)

Design Phase	I. Phase	II. Phase		III. Phase	
Design Type	Pre-design (Size)	Evaluation		Evaluation	
	Pre-design	DTS=1, 2, 3, 3a, 4a	DTS=1a, 2a	DTS=1, 2, 3, 3a, 4a	DTS=1a, 2a
Earthquake Level / Performance Objective	DD-2/CD	DD-4 /CO	DD-3 /LD	DD-1/CP	DD-1/CD
Analysis Type	Modal calculation methods with Force-Based Design (FBD) to be used	Force-Based Design (FBD) and modal calculation methods will be used. R/I=1 and D=1 will be accepted.	Non-linear calculation methods in the time history will be used with Deformation-Based Design (DBD)		

The design and assessment processes are described below the performance goals set out in TBDY 2018. The guidelines to consider in modeling and analysis are summarized at each level. The design and assessment criteria offered for reinforced concrete in the regulation are tested in the current study owing to the use of reinforced concrete. Table 3.9 shows the design phases based on the performance objectives specified in the rule.

Building design objectives are connected to the combinations of target performances and ground motion levels are taken into account in the design. The standard Force-Based Design (FBD) is then required, or, if necessary, the Performance-Based Assessment and Re-Design (PBARD) (Sucuoğlu, 2018).

An ordinary performance objective is set for all structures not classed as tall buildings, "Controlled Damage" under DD-2 or the 475-year design ground motion. "Advanced" target performance is specified if the building has a DTS of 1a or 2a (important buildings under high-intensity GMs). Under the DD-1 and DD-3 design spectra, performance-based approaches are used to analyze building performance, whereas the DD-2 design spectrum suggests a force-based preliminary design. Limited Damage under the service earthquake DD-3 (43 years) and Controlled Damage under the maximum predicted earthquake DD-1 is advanced performance objectives (2475-year) (Sucuoğlu, 2018).

When a building is classed as a tall building (BYS=1) based on its height and seismic design category in the category table, it has a dual ordinary performance objective. The tall building should stay linear elastic under the service earthquake DD-3 (43 years) and meet the collapse prevention limit under the maximum predicted earthquake DD-1 (2475-year). A force-based analysis can be used to evaluate service level performance. Still, a performance-based approach that requires nonlinear time history analysis should evaluate collapse prevention performance. Table 3.9 shows these relationships (b) (Sucuoğlu, 2018).

The initial design can be force-based with a controlled damage goal under the 475-year design earthquake, as recommended in Table 3.9 (b). However, this design is frequently limited by architectural restrictions and dimensions recognized in practice. Tall building designers prefer to start with design forces derived from the 43-year DD-4 service earthquake spectrum rather than using the 475-year earthquake and decreasing the resulting points with R factors that are practically justified for tall buildings.

Table 3.9 Performance targets and design procedures for new buildings given in TBDY 2018.

(a) Cast-in-place reinforced concrete, precast concrete, and steel buildings

EQGM LEVEL	DTS = 1, 1a(1) , 2, 2a(1), 3, 3a		DTS = 1a(2) , 2a(2)	
	Ordinary performance target	Design procedure	Advanced performance target	Design procedure
DD-3	-	-	LD	PBD
DD-2	CD	FBD	CD	FBD(3)
DD-1	-	-	CD	PBD

(b) Tall Buildings (BYS = 1)

Seismic ground motion level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a , 2a	
	Normal Performance	Check and Design Approach	Advanced performance	Check and Design Approach
DD-4	IO (KK)	SBD(DGT)	-	-
DD-3	-	-	LS (SH)	DBD(SGDT)
DD-2	CD (KH)	SBD(DGT)	CD (KH)	SBD(DGT)
DD-1	CP (GO)	DBD(SGDT)	CD (KH)	DBD(SGDT)

Advanced performance goals are unlikely to be met by tall buildings.

In the 2018 Seismic Code, comparable performance figures are supplied for structures that are already in place. The assessment/design technique is performance-based, with controlled damage (CD) under a 475-year DD-2 earthquake as the standard performance aim. A unique displacement-based linear elastic approach is being created for existing structures (Sucuoğlu, 2018).

3.3.5 Design Phase-I

At this point, the structure's preliminary design will be completed. The initial setup of the building based on the plan will be made with linear calculation methods and strength, taking into account Section-4 of the regulation, under the DD-2 earthquake action, which TBDY 2018 accepts as the design earthquake, as the first step in the design of high-rise buildings. This design step aims to provide the structure with a regulated damage performance level. In the preliminary design of structural system elements, TBDY 2018-Chapter seven will be used as a guide for reinforced concrete structures.

3.3.5.1 Carrier System Modeling

The building model will be modeled according to linear behavior in the first step to produce the system's preliminary design. Although the structure is treated as linear, various coefficients and constraints accommodate nonlinear behavior. Unless otherwise specified, the system should always be represented in three dimensions with a damping ratio of 5%. The following are the points to consider while designing a model for linear calculating methods:

- **Column and beam modeling:**

- All beams and columns will be represented as finite rod elements. At ,this degrees of freedom will be examined at the beam and column intersections degrees of freedom associated with the stiff movement will be eliminated if the slabs are treated as a rigid diaphragm.
- In all columns and beams, effective section stiffness shall be employed. For seismic-free load connections, these effective section stiffness will not be applied.

- **Shear walls modeling:**

- Walls will be treated as vertical load-bearing components with a length-to-thickness ratio of 6 or greater.
- The shear frame model, which models the wall elements like columns and the body region as beams, is not employed.

- Shell finite elements will simulate reinforced concrete walls with rectangular, I, T, L, U, or C cross-sections in the plan.
- The corresponding bar model can be utilized for modeling convenience if the length of the shear wall in the plan to the overall height of the shear wall is less than 1/2. The corresponding bar model must pass through the section's center of gravity for shear components.
- The shear wall must fulfill the criterion of Eq.(3.12) to be a hollow shear wall. Each shear wall will be referred to as a blank shear wall if this requirement is not satisfied.

$$\Omega = \frac{cN_V}{M_{DEV}} \geq {}^{1}/_{3} \tag{3.12}$$

➢ **Slabs Modeling:**

- Slabs in structures with irregularities of A2 (slab discontinuity) and A3 (plan protrusions) or where beds are not meant to operate as a rigid diaphragm will be modeled with two-dimensional finite elements. On the other hand, rigid diaphragms should not be employed in reinforced concrete slabs systems that do not include beams.
- There should be no A2 and A3 abnormalities in the slabs if characterized as rigid diaphragms. It is not desirable to depict the building slabs as rigid diaphragms in the face of these imperfections.
- The effective section stiffnesses for slabs can be taken from Table 3.10.

➢ **Masses Modeling:**

- Individual masses at nodes are specified as in eq.(3.13).

$$W_J^{(S)} = W_{GJ}^{(S)} + nw_{QJ}^{(s)} \qquad ; \qquad m_j^{(S)} = \frac{w_j^s}{g} \tag{3.13}$$

- Based on the building's intended usage, the live load mass participation coefficient (n) will be chosen from Table 3.11.
- Thirty percent of the snow loads will be factored into the roof weight calculation.
- If a stiff diaphragm is assumed, the story masses are specified to the center of mass. The live load mass participation coefficient should be considered when calculating the loads operating on the stories.

Table 3.10 Rigidity factors for Strength-based design (DGT) given in TBDY 2018.

Concrete Member	**Effective Stiffness Multiplier**	
Wall – Slab (In-Plane)	**Axial**	**Shear**
Shear Wall	0.50	0.50
Basement Shear wall	0.80	0.50
Slab	0.25	0.25
Wall – Slab (Out Of Plane)	**Flexure**	**Shear**
Shear Wall	0.50	1.00
Basement Shear wall	0.25	1.00
Slab	0.50	1.00
Frame member	**Flexure**	**shear**
Coupling beam	0.15	1.00
Frame beam	0.35	1.00
Frame column	0.70	1.00
Wall (equivalent strut)	0.50	0.50

Table 3.11 Live load participation coefficient given in TBDY2018.

Purpose of Use	**n**
Warehouse, etc.	0.8
School, student dormitory, sports facility, cinema, theatre, concert hall, place of worship, restaurant, store, etc.	0.6
Residence, workplace, hotel, hospital, parking lot, etc.	0.3

3.3.5.2 Combinations of Earthquake Loads

Internal forces based on the design must be calculated using the load combinations described in Eq.s (3.14) and (3.15).

$$G + Q + 0.2S + E_d^{(H)} + 0.3E_d^{(Z)} \tag{3.14}$$

$$0.9G + H + 0.2S + E_d^{(H)} - 0.3E_d^{(Z)} \tag{3.15}$$

Here S is snow loading, $E_d^{(H)}$ is the horizontal earthquake effect, $E_d^{(Z)}$ is the vertical earthquake effect, and H is horizontal ground thrust.

The horizontal earthquake effect will be calculated using Eq.s (3.16) and (3.17);

$$E_d^{(H)} = \pm E_d^{(X)} \pm n3E_d^{(Y)} \tag{3.16}$$

$$E_d^{(H)} \pm n3E_a^{(X)} \pm E_a^{(Y)} \tag{3.17}$$

- In the following way, the vertical earthquake impact is considered in the Earthquake Code.

  For DTS=1, 1a, 2 and 2a;

- Structures incorporating beams with a horizontal projection of 20 meters or more,
- Structures having cantilevers that have a horizontal projection of 5 meters or more significant,
- Structures with columns resting on beams,
- Vertical earthquake estimates will be performed for structures with inclined columns relative to the vertical.
- The vertical elastic acceleration spectrum will be applied for these components in the vertical earthquake computation. The entire carrier system R/I=1 and D=1 will be used in this situation.

In structures that lack the stated requirements or components, $E_d^{(Z)} = \left(\frac{2}{3}\right) S_{DS} G$ will be used as the vertical earthquake effect, and the design will be taken into account in seismic load combinations.

3.3.5.3 Calculation of Earthquakes

The following are the earthquake calculation phases that must be completed as part of Design Phase-I:

- Before starting with the earthquake calculation, vertical load and wind calculations will be performed, considering the building phases and creep analyses in reinforced concrete structures.
- High ductility solutions will be chosen for tall building support systems. Mixed ductility systems can only be used in structural systems of tall structures with DTS=4. Systems with limited ductility are not permitted in any situation.
- TBDY 2018 will be used to calculate R and D coefficients. The following scenarios will be considered when determining the carrier system behavior coefficient.

- The total base overturning moment $\boldsymbol{M_o}$ resulting from earthquake loads for the whole building in that direction should not exceed 1/3 of the final base overturning moment M_{DEV} of a single shear wall or steel braced frame in the structural system.

- The total base overturning moment M_o resulting from the seismic loads for the whole building in that direction is less than 1/6 of the final base overturning moment M_{DEV} or M_{DEV}s taken by the shear(s) or steel braced frame(s) on each side axis of the structure. It is not going to happen.
- If any of the following requirements aren't satisfied, $(4/5)R$ will be utilized instead of R. The coefficient of extra strength will remain unchanged.

➢ If $\Omega \geq 1/3$ is provided by the hollow shear wall, the shear wall will be treated as a single shear wall.

➢ Design Phase-I earthquake calculations will be performed in Time History using the Mode Combination Method or the Mode Summation Method under DD-2 earthquake ground motion.

➢ The base shear force calculated using Modal Calculation Methods will be enhanced to meet high-rise structures' minimum base shear force. The Eq. will be used to magnify (3.18).

$$V_{t,min} = 0.04\alpha_H m_t S_{DS} g \quad ; \quad \beta_{tE} = \frac{\gamma_E V_{t,min}}{V_t} \geq 1 \tag{3.18}$$

$V_{t,min}$: Minimum base shear force

β_{tE}: Equivalent base shear force amplification coefficient

m_t : Total mass of plinth and tower

S_{DS} : Short period design spectral acceleration factor for DD-2

g: Gravity acceleration.

Including

$\alpha_H = 1.0$ $\quad H_N \leq 105\ m$

$\alpha_H = 2.5 - 0.01H_N$ $\quad 105\ m < H_N \leq 155\ m$

$\alpha_H = 0.5$ $\quad 155\ m < H_N$

The Mode Combination Method or the Computational Mode Aggregation Method in the Time Domains are the computation techniques to apply at this step. According to TBDY 2018, the required number of modes to be considered in analyses employing modal calculation techniques will be assessed as not less than 95 percent of the modal participation rate.

3.3.5.4 Sizing the Carrier System

The dimensioning and design guidelines only apply to reinforced concrete structures since the current research focuses on them. The highest cross-sectional impacts acquired after the seismic calculations for Design Phase-I are used to dimension reinforced concrete components and do reinforcement calculations. Because only high ductility levels are permitted in tall structures, this chapter exclusively covers the criteria for increased flexibility.

The practical section stiffnesses must be employed in the structural system calculations. According to the regulations, it is not permissible to use a concrete class lower than C25. The concrete type should be in the range of C25 to C80. Compressive strengths will be calculated using TS EN 206 if a higher concrete class is required. B420 and B500C reinforcing steel will be utilized in reinforced concrete components within the rule's scope. The "tensile/yield strength" ratio should not be less than 1.35, and the corresponding carbon ratio should not exceed 0.55 percent if the S420 reinforcing class is utilized. It is prohibited to use S420 reinforcing steel to fulfill these requirements.

Special earthquake stirrups should be employed in reinforced concrete structures with high and restricted ductility, columns, column-beam junction regions, shear end areas, and beam wrapping areas. At the same time, all crossties utilized in these areas must be earthquake-resistant. Figure 3.7 depicts the special earthquake stirrup and special earthquake crossties requirements.

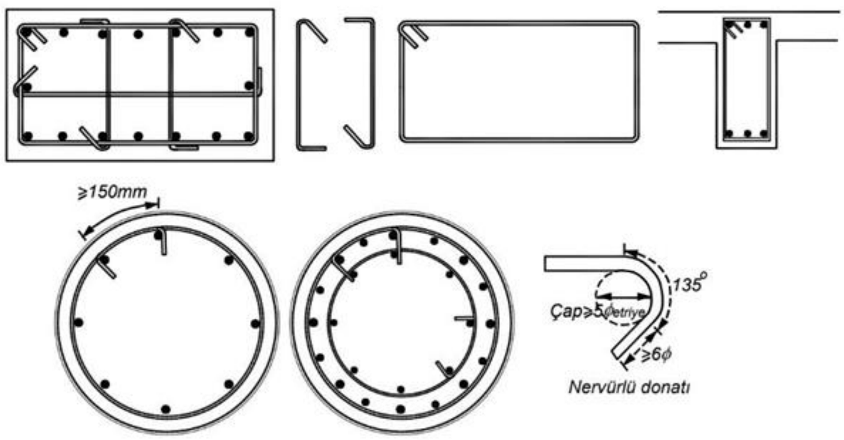


Figure 3.7 Special earthquake stirrups and crossties (Celep, 2017).

Columns Having a High Level of Ductility:

- The smallest cross-section of rectangular columns should be 30 cm, and the diameter of circular columns should be at least 35 cm.
- The condition $Ac \geq N_{dm}/(0.40 f_{ck})$ will be determined by considering the highest axial force in the columns due to the G+Q+E earthquake.
- The reinforcing ratio for the columns will range between 1% and 4%. There will be a minimum of 14 utilized. In the overlap zones, the reinforcement ratio will be 6%.
- In the column wrapping regions, a minimum of ϕ8 should be utilized. The spacing of stirrups and crossties in the confinement zones will be larger than $1/3$ of the smallest section size 15 cm. However, the space between the transverse reinforcements must be at least 50mm and six times the longitudinal support's diameter.
- In rectangular stirrup rectangular columns with $N_d > 0.20 A f_{ck}$ (pressure), the minimum stirrup reinforcement area is computed using eq.(3.19) and eq.(3.20) in the confining areas. To offer the unfavorable one in the Eq.s, reinforcement should be done.

$$A_{sh} \geq 0.30\ s\ b_k\ \left[\left(\frac{A_c}{A_{ck}}\right) - 1\right]\left(\frac{f_{ck}}{f_{ywk}}\right) \tag{3.18}$$

$$A_{sh} \geq 0.075\ s\ b_k \left(\frac{f_{ck}}{f_{ywk}}\right) \tag{3.19}$$

- If $N_d \leq 0.20 A_c f_{ck}$ the transverse reinforcement area in the column confinement zones should be at least 2/3 of the unfavorable transverse reinforcement area calculated using the Eq.s in eq.(3.18) and eq.(3.19).
- In the column center of the column, transverse reinforcement of less than ϕ8 should not be employed. The distance between transverse struts must be half the shortest cross-section length and no less than 20 cm.
- The total of the bearing day moments of the columns at all column-beam connection sites shall be 20% larger than the sum of the bearing capacity moments of the beams in systems where frame or shear and frame systems are combined.

$$(M_{ra} + M_{r\ddot{u}}) \geq 1.20\left(M_{ri} + M_{rj}\right) \tag{3.20}$$

- It is not essential to utilize Eq. (3.20) at the junction region if the nodes' columns are $N_d \leq 0.10A_c f_{ck}$ and do not continue in single-story structures or higher levels.
- The shear force V_e must be considered while calculating the transverse column reinforcement is computed as follows: eq.(3.21).

$$V_e = (M_a + M_{ü})/l_n \quad (3.21)$$

- If the columns do not meet the criteria of being more potent than the beams, or if a more accurate calculation is not performed, Ma and Mü may be calculated using (3.22).

$$M_{pa} \cong 1.4M_{ra} \quad ; \quad M_{pü} \cong 1.4M_{rü} \quad (3.22)$$

- The shear force estimated Ve, the vertical loads multiplied by the load coefficients, and the shear force computed under the combined action of seismic loads will not be less than V_e and will satisfy the requirements outlined in eq.(3.23). Eq.(3.24). the section dimensions should be increased if eq.(3.23) cannot be accomplished.

$$V_e \leq V_r \quad (3.23)$$

$$V_e \leq 0.85A_w\sqrt{f_{ck}} \quad (3.24)$$

Beams Having a High Level of Ductility:

- Reinforced concrete beams shall have a minimum body width of 250 mm. The total beam height and the width of the column or shear wall perpendicular to the beam cannot be larger than the width of the beam body.
- Beam height must be at least three times the thickness of the story and 300 mm. A story thickness of fewer than three times or less than 300 mm is fitted as beams and modeled with story components, but they are not regarded as frame beams. Furthermore, the height of the beam cannot exceed 3.5 times the breadth of the body.
- Body reinforcement will be installed on both sides of the beam body if the beam height exceeds (1/4) of the free span. In this situation, the area of the body reinforcement to be put in the support sections of the beam should not be less than 30% of the total of the lower and upper mount. The reinforcement spacing should not be more than 300 mm, and the diameter of the body reinforcement should not be less than 12 mm. Crossties should be spaced no more than $600mm$

apart throughout the beam's length. It should not exceed $400mm$ along the beam's axis.

- The $N_d \leq 0.10\, A_C\, f_{CK}$ requirement must be met by the beams. These items should be constructed as columns if not otherwise.
- Tensile reinforcement in the beams' support region must satisfy a minimum of $\rho \geq 0.8\, f_{ctd}/f_{yd}$ requirement. The longitudinal struts must have a minimum diameter of 12 mm. In addition, there should be a minimum of two mounts on the bottom and top of the beam. (reinforcement for mountıng)
- In systems with DTS = 1, 1a or 2, 2a, at least half of the upper reinforcement in the support regions should be used as support lower mount. This percentage might be as high as 30% with some Seismic Design Categories (DTS) systems.
- The maximum reinforcement ratio must meet the TS500 standards in the opening regions. Furthermore, the total amount of reinforcement in these shadows should be 2%.
- In the winding shadow of the beams, the minimum diameter of stirrups should be 8 mm. The first stirrup should be set 50 mm from the column face at the most.
- For all earthquake scenarios, the shear force Ve, which should be considered in calculating transverse reinforcement in beams, will be computed using eq.(3.25). The most unfavorable one will be chosen (Figure 3.8).

$$V_e = V_{dy} \pm \left(M_{p\iota} + M_{pj}\right)/l_n \tag{3.25}$$

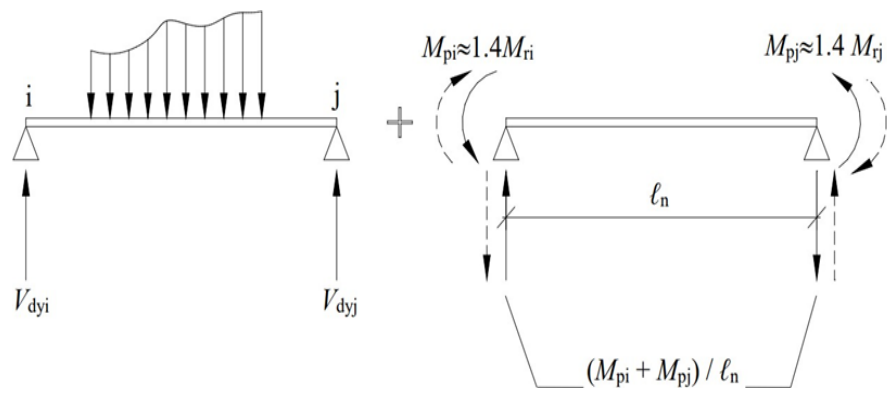


Figure 3.8 Design shear force in beams (Celep, 2017).

- Unless accurate calculations are conducted, the moment capacities at the beam ends may be assumed to be $M_{pi} \cong 1.4M_{ri}$ and $M_{pj} \cong 1.4M_{rj}$. The total of the shear forces, computed with the vertical loads in mind and multiplied by the coefficient D, should be compared to Ve, and if it is smaller, this shear force should be employed.
- V_e must fulfill eq.(3.26) and eq.(3.27) when computed using the rules mentioned above eq.(2.30). The cross-section dimensions should be raised if (2.30) cannot be reached.

$$V_e \leq V_r \tag{3.26}$$

$$V_e \leq 0{,}85 b_w\, d \sqrt{f_{ck}} \tag{3.27}$$

Shear Walls Having a High Level of Ductility:

- The net wall cross-sectional area must fulfill the $Ac \geq N_{dm}(0.35fck)$ requirement, which considers the most considerable axial compressive forces resulting from the combined impact of seismic and vertical loads.
- The long side-to-thickness ratio must be six for a vertical carrier element to be considered a shear wall.
- The body thickness of the shear walls shall be 1/16 of the story height and 250 mm thick, except in exceptional circumstances the ended.
- In case $Hw/lw > 2.0$ end zone will be created in reinforced concrete shear elements. Here Hw represents the total shear wall height from the ground story. Hw denotes the height from the top of the foundation or to the point where the length of the wall in the plan decreases by 20% or the section width changes by more than half.
- The most unfavorable result derived from the calculation is $2lw \geq Hcr \geq max\ [lw; Hw/6]$ which is the critical height of the wall from the top of the foundation or the level where the length of the wall in plan drops by more than 20%.
- Figure 3.9 shows the principles for the wall end zones and body reinforcements.
- The critical height of the wall will be determined by the bending moments for the design and the bending moment computed as a consequence of the analysis at the bottom of the border for walls with $Hw/lw > 2.0$. The actual wall height will be estimated using a linear moment diagram parallel to the line linking the

moments measured from the wall base to the top of the wall. The bending moments derived from earthquake calculations will be equivalent to the design bending moment for shear walls with $Hw/lw \leq 2.0$. (Figure 3.9).

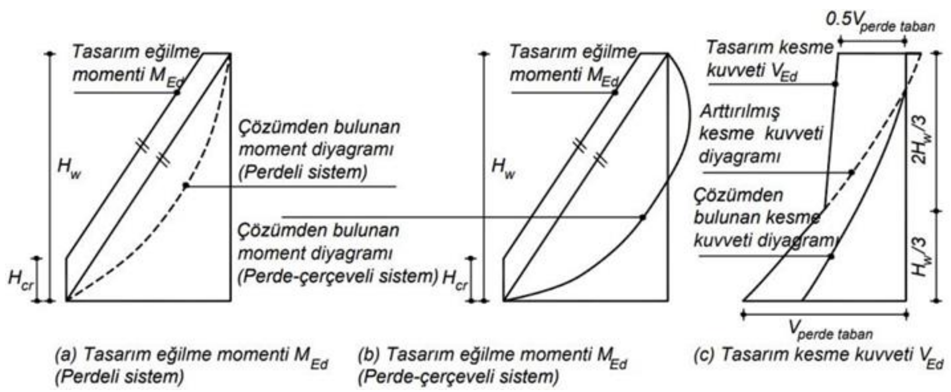


Figure 3.9 Shear wall design Flexure moments (Celep, 2017).

- The design shear force Ve will be utilized to calculate the transverse wall reinforcements that fulfill the requirement $Hw/lw > 2.0$ is calculated by the Eq. (3.28).

$$V_e = \beta_v \frac{(M_P)t}{(M_d)t} V_d \tag{3.28}$$

- In (2.31), if the entire earthquake load is met with reinforced concrete shears $\beta_v = 1$ will be taken. In other cases, $\beta_v = 1.5$ will be taken. It can be accepted as $(Mp) \leq 1.25(Mr)$ in the absence of an exact calculation. Considering the vertical loads, the shear force calculated from the earthquake should be compared with Ve which is defined as $1.2D$ (non-spaced shears) or $1.4D$ (spaced shears) times, and if it is small, this shear force should be used instead of Ve.
- Shear strength in shear walls is calculated by the Eq.$V_r = A_{ch}(0.65 f_{ctd} + P_{sh} f_{ywd})$.
- The design shear force must satisfy the Eq.s given below. If these Eq.s cannot be met, the reinforcement or section dimensions should be increased. Example reinforcement layout according to TBDY 2018 is shown in Figure 3.10.

$$V_e \leq V_r \tag{3.29}$$

$$V_e \leq 0{,}85 A_{ch} \sqrt{f_{ck}} \qquad (Gapless\ Curtains) \tag{3.30}$$

$$V_e \leq 0.65 A_{ch} \sqrt{f_{ck}} \qquad (Tie - Beam\ Curtains) \tag{3.31}$$

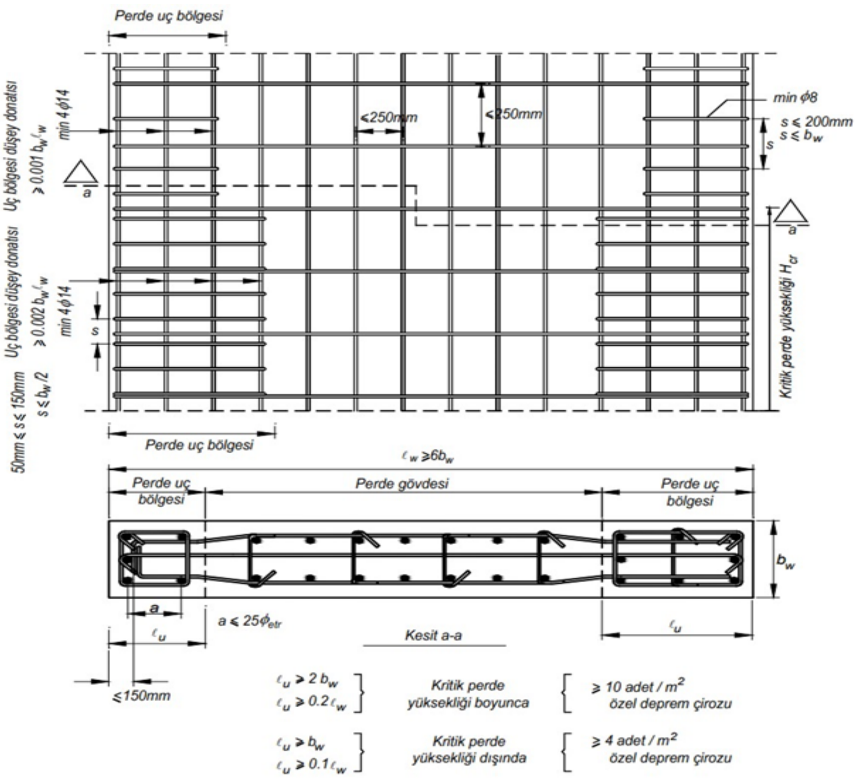


Figure 3.10 Shear walls body and end reinforcements (Celep, 2017).

3.3.6 Design Phase-II

This is the phase in which the pre-designed building is evaluated. Under the DD-4 earthquake, the typical performance objective of the building is to offer the uninterrupted usage performance class based on the Design Based on Strength (DGT) method. If an advanced performance objective is needed, the DD-3 earthquake should be used with the Evaluation and Design Based on Strain (DGT) principles to offer a restricted damage performance level. Because the structure under consideration in this research is assessed for average performance, explanations are given under this title utilizing the DGT technique for design phase II. The SGDT method has been defined under the heading of design phase III.

Because DGT is used as a foundation in this design phase, the modeling and seismic calculation techniques outlined in Design Phase-I also apply to this phase. The concepts that vary depending on the preliminary design level are described below.

This is the phase in which the pre-designed building is evaluated. Under the DD-4 earthquake, the typical performance objective of the building is to offer the uninterrupted usage performance class based on the Design Based on Strength method.

If an advanced performance objective is needed, the DD-3 earthquake should be used with the Evaluation and Design Based on Strain principles to offer a restricted damage performance level. Because the structure under consideration in this research is assessed for average performance, explanations have been provided under this category for the design phase II utilizing the Design Based on the Strength technique. Evaluation and Design Using the Shapeshifting Approach has been detailed under Design Phase III.

Because Design Based on Strength is used as the foundation in this design phase, the modeling, and earthquake calculation techniques described in Design Phase-I apply to this phase. The concepts that vary depending on the preliminary design level are described below.

3.3.6.1 Carrier System Modeling

The design ideas from phase one apply to structural system modeling. Some regulations, however, are different. Below is a list of these distinctions.

Table 3.12 Effective stiffness of concrete members for Design Phase-II given in TBDY2018.

Concrete Member	**Effective Stiffness Multiplier**	
Wall – Slab (In-Plane)	**Axial**	**Shear**
Shear Wall	0.75	1.00
Basement Shear wall	1.00	1.00
Slab	0.50	0.80
Wall – Slab (Out Of Plane)	**Flexure**	**Shear**
Shear Wall	1.00	1.00
Basement Shear wall	1.00	1.00
Slab	0.50	1.00
Frame member	**Flexure**	**shear**
Coupling beam	0.30	1.00
Frame beam	0.70	1.00
Frame column	0.90	1.00
Wall (equivalent strut)	0.80	1.00

- The impact of increased eccentricity will be ignored.
- The carrier system is supposed to behave linearly or nearly linearly. As a result, a damping ratio of 2.5 percent is allowed.

3.3.6.2 Calculation of Earthquakes

The Design Phase-II earthquake calculations are described here:

- Because Design by Strength is employed, earthquake calculations should be performed using linear techniques. Modal calculation techniques will be used to do linear analyses.
- $R/I = 1$ And $D = 1$ will be used when calculating internal forces in a linear seismic calculation.
- In Design Phase, I, the minimal base shear force rule will not be employed. Because earthquake calculations need 2.5 percent damping, the design spectrum should be expanded to account for this damping ratio. The horizontal elastic design spectral acceleration numbers for the DD-4 earthquake will be multiplied by 1.25 to get flat flexible design spectral acceleration values for the 2.5 percent damping ratio.
- When utilizing the Perfect Quadratic Model Combination Method to calculate cross-correlation coefficients, the damping ratio should be 2.5 percent.

3.3.6.3 Evaluation of Performance

The internal forces of the sections are calculated during Design Phase II. These are the internal forces employed in the assessment. The guidelines for evaluating reinforced concrete components are as follows.

- The Demand /Capacity (D/C) ratio of the components with ductile behavior stated below should not exceed 1.5.
- P-M-M-effect flowing at the base portions of the shear walls or in the area up to a specific height is referred to as reinforced concrete shears.
- Ductile elements are beam and column elements with a bending effect at the beam ends and a P-M-M impact on the column base sections.
- In elements where the ductile behavior indicated below is not in dispute, the Demand /Capacity (D/C) shall not exceed the limit value of 0.7.

- Internal shear stresses in walls, basement walls, columns, and beams do not exhibit ductile behavior.
- Internal forces in columns exclusively subjected to axial load do not exhibit ductile behavior.
- Internal forces that do not have tensile behavior cause the punching effect in slabs or raft foundations without punching reinforcement.
- Internal forces in foundations that are not ductile are known as shear forces.
- Internal forces are not malleable and transmitted from stories or stories in transfer stories to vertical load-bearing components.

For shear pressures that might produce brittle collapse, the D/C ratio must not exceed 0.7, as specified.

In addition to analyzing the components, the tensile behavior of columns, beams, and walls should be assessed. It should, nevertheless, be regarded as a non-ductile element when analyzing the ratio of shear demand in the components to shear capacity.

If any elements do not fulfill the requirements mentioned in the details, the required arrangements should be made at this point, and the design phases should be redone. At this point, achieving the requirements specified by the calculations performed using the DGT technique indicates that the elements have conducted the performance goal of continuous usage.

3.3.7 Design Phase-III

In the final step, the first two phases of the construction are finished and assessed using nonlinear calculating techniques and the SGDT methodology. The most significant earthquake considered in the standard, DD-1, is employed at this step. The structural components must give the performance level of avoiding migration to meet the typical performance objective at this earthquake level. The carrier system parts must provide a regulated damage performance level for the advanced performance objective.

The deformations, internal forces, and relative story drift values produced in the structural system components shall not exceed the limit values provided in the regulation as a consequence of nonlinear analysis in the time history of the high-rise building model developed at this phase.

3.3.7.1 Carrier System Modeling

The ideas presented in Design Phase-I apply to structural system modeling. However, there are several exceptions to the norms. The carrier system, which is constructed and assessed in the previous step, will now be modeled in the time history for the nonlinear calculation approach. At this point, modeling should be done by SGDT. The structure should be represented in three dimensions. Two horizontal earthquakes occurring one after the other will undoubtedly be considered. For systems with a high damping ratio, 2.5 percent will be taken. The material's predicted (average) strength will be utilized instead of the material's characteristic strength in the performance assessment of new high-rise structures. The material's predicted strengths are shown in Table 3.13.

Table 3.13 Predicted Strengths of materials.

Material	Predicted Strengths
Concrete	$f_{ce} = 1.3f_{ck}$
Reinforcement Steel	$f_{ye} = 1.2f_{yk}$
Structural Steel (S235)	$fye = 1.5fyk$
Structural Steel (S275)	$fye = 1.3fyk$
Structural Steel (S355)	$fye = 1.1fyk$
Structural Steel (S460)	$fye = 1.1fyk$

The present strengths of the material will be used as a foundation for evaluating performance in existing structures. The modeling of construction blocks will follow the first phase.

Columns and beams modeling:

- Rod finite element models will be used to represent all beams and columns. A spread or layered plastic behavior model might be utilized as a nonlinear behavior model. Because of its simplicity of use and faster analysis durations, the layered plastic behavior model is suitable.
- At the column-beam connections, there will be six degrees of freedom. The latitudes against rigid movement may be eliminated when employing a rigid diaphragm.

- Plastic hinges may be used at the ends of net apertures in rod-element columns and beams.
- The practical section stiffness to be employed in this location will be computed using Eq.s in (3.32) and (3.33) since the rod element acts linearly between plastic hinges (2.35).

$$(EI)_e = \frac{M_y}{\theta_y}\frac{L_s}{3} \tag{3.32}$$

$$\theta_y = \frac{\phi_y L_s}{3} + 0.001\eta\left(1 + 1.5\frac{h}{L_s}\right) + \frac{\phi_y d_b f_{ye}}{8\sqrt{f_{ce}}} \tag{3.33}$$

- In Eq.s (3.32) and (3.33), My shows the yield moment, θy yield rotation, Ls shear opening, ϕy yield curvature, db average reinforcement diameter, fye expected yield strength of reinforcement, and fce expected concrete strength.

Shear Walls and Story Modeling:

- When building the nonlinear behavior model of the shear walls, the diffuse plastic behavior model should be employed. Because nonlinear analysis in the time history is required in high-rise structures, applying layered plastic behavior on shear walls is not permitted.
- The section should be split into adequate section cells for modeling these components.
- The finite element mesh utilized in shear walls should be separated to provide appropriate precision.
- When modeling basement shear walls, a non-linear model is not required. Only if the building has more than one basement level would it be more appropriate to use a non-linear behavior model to simulate the shear wall components in the first basement story following the ground story.
- Nonlinear behavior is not required for modeling tiles. As a result, slabs are represented linearly to speed up the model's solution.
- For linearly modeled basement walls and floors, the effective section stiffness factors given in Table 3.10 will be used.

Effect of additional eccentricity

- Suppose the torsional irregularity coefficient is $\eta_{bi} > 1.5$ in the building. In that case, the horizontal earthquake effects coming to the story mass center will be surprised by +5% and -5% of the story plan size, and earthquake analysis will be performed separately for these cases.

3.3.7.2 Calculation of Earthquakes

In the temporal description, earthquake accounts should be produced using the nonlinear account approach. Due to nonlinear behavior, the stiffness matrix of the system is considered in time throughout this procedure.

At this point, 11 earthquake ground movement parts should be chosen. Section 3.3.7.3 explains the concepts that must be considered when calculating earthquake motions. Horizontal earthquake components will influence the carrier system simultaneously. Nine hundred rotations of the earthquake story movement acceleration recordings will next be used to solve the system. As a result, 2x11 = 22 accounts must be created.

The following are the problems to examine at the Earthquake account phase:

- The dynamic energy loss of non-structured and linear components in the carrier system elements is treated as a viscous damping matrix in motion Eq.s. Rayleigh proportional damping matrix or modal damping matrix will generate the dense damping matrix. The damping rate will be accepted at 2.5 percent throughout the development of this damping matrix (Figure 3.11).

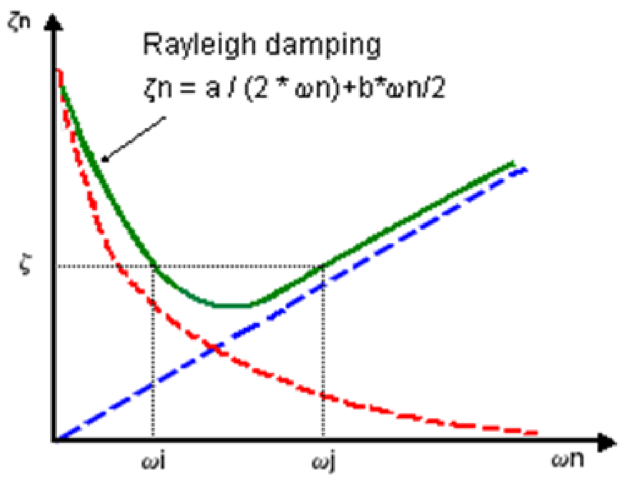


Figure 3.11 Rayleigh damping function(Akçora, 2020).

- Non-linear incremental static analysis should be done under out-of-earthquake installations before earthquake analyses. As a consequence of this account, non-linear paths may now be created. This analysis should be used as the nonlinear account's beginning value in the temporal description.

The needs for ductile behavior are the average of the most significant absolute values of all analyses (2x11 = 22 accounts) due to the calculations based on the above principles. This will be compared to the obtained limit value, and the element's performance class will be selected.

By adding the standard deviation value of the most significant absolute values of the values obtained from all accounts, the critical internal forces of the elements indicated in *3.3.7.3* will be determined. However, the estimated number must not exceed 1.5 times the average value and must not fall below 1.2 times the average value.

3.3.7.3 Evaluation of performance

The numbers acquired from the calculations in the final step of high structures should be compared to the regulation's limit values. At this point, the form will be examined for typical performance targets, and the carrier system parts will be necessary to avoid the migration performance limit. It is desired that the details of the carrier system do not exceed the limits of controlled damage if the structure is prompted to assess the forward performance objective.

- For concrete and reinforcing steels, the prevention of immigration (CP), controlled damage (KH), and limited damage (SH) classes are listed below when using the propagating plastic behavior model.

For Collapse Prevention (CP):

a) Concrete Unit Shortage

$$\varepsilon_s(CP) = 0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018 \ \ (Rectangular\ Section)$$

$$\varepsilon_s(CP) = 0.0035 + 0.07\sqrt{\omega_{we}} \leq 0.018 \ \ (Circular\ Section) \tag{3.34}$$

b) Reinforcing steel unit deformation

$$\varepsilon_s(CP) = 0.4\varepsilon_{su} \tag{3.35}$$

<u>Since ε_{su} = 0.08 for S420:</u>

$$\varepsilon_s(CP) = 0.4x0.08 = 0.032$$

For Controlled Damage (CD):

$$\varepsilon c(CD) = 0.75\varepsilon_c(CP) \quad , \quad \varepsilon s(CD) = 0.75\varepsilon_s(CP) \tag{3.36}$$

For Limited Damage (LD):

$$\varepsilon_c(LD) = 0.0025 \quad , \quad \varepsilon_s(LD) = 0.0075 \tag{3.37}$$

- In the case of using the stacked plastic behavior model, the plastic rotation limits are given below for the Prevention of Collapse (CR), Controlled Damage (CH), and Limited Damage (SH) classes.

For Collapse Prevention (CP):

$$\theta_P^{(CP)} = \frac{2}{3}\left[(\emptyset_u - \emptyset_y)L_P\left(1 - 0.5\frac{L_P}{L_s}\right) + 4.5\emptyset_u d_b\right] \tag{3.38}$$

For Controlled Damage (CD):

$$\theta_P^{(CD)} = 0.75\theta_P^{(CP)} \tag{3.39}$$

For Limited Damage (LD):

$$\theta_P^{(LD)} = 0$$

Plastic spins are not allowed at the Limited Damage (LD) performance level.

- As a result of earthquake calculations in the time domain, the average relative story drift ratio should not exceed the limited value of 0.03. The relative story drift obtained in an earthquake can be a maximum of 0.045.

3.3.8 Finalization of design

Phase-III account principles must not be reduced to the reinforcements or dimensions of the vertical carrier system components in Design Phase -I, the preliminary design of the structure. If a reduction is required, all phases should be repeated—the vertical carrier element without beams. If the details are only needed at the end, they may be omitted. Only the final phase of the analysis will be repeated in this scenario.

CHAPTER 4

4. CASE STUDY: DESIGN OF A HIGH-RISE RC BUILDING ACCORDING TO TBSC 2018.

A high-rise structure is linearly studied and constructed in this case study by Turkey's newly released code TBDY 2018. A time history analysis is performed after the analysis and design, and the performance is evaluated and compared with local code IYBDY 2007.

4.1 General Information about the Building

The high-rise building to be analyzed is located within the borders of Istanbul province, Maslak district, with a longitude and latitude of (29.017682°, 41.131012°). Earthquake data took from Turkey Earthquake Hazard Maps using these latitude and longitude values. The size of this building plan is 39 m x 45 m. This structure has parking and is used for residential purposes. Two basement stories, and 28 ordinary stories. The ordinary stories are 3 meters high, while the basements are 5 meters high. The structure is 96 meters tall in total, with basement stories included. Figure 4.1 shows the story plan. The building will be used as a place for people to live. 3D view of the entire structure in Figure 4.2's structural system in the Etabs program. Its location is marked on the map in Figure 4.3.

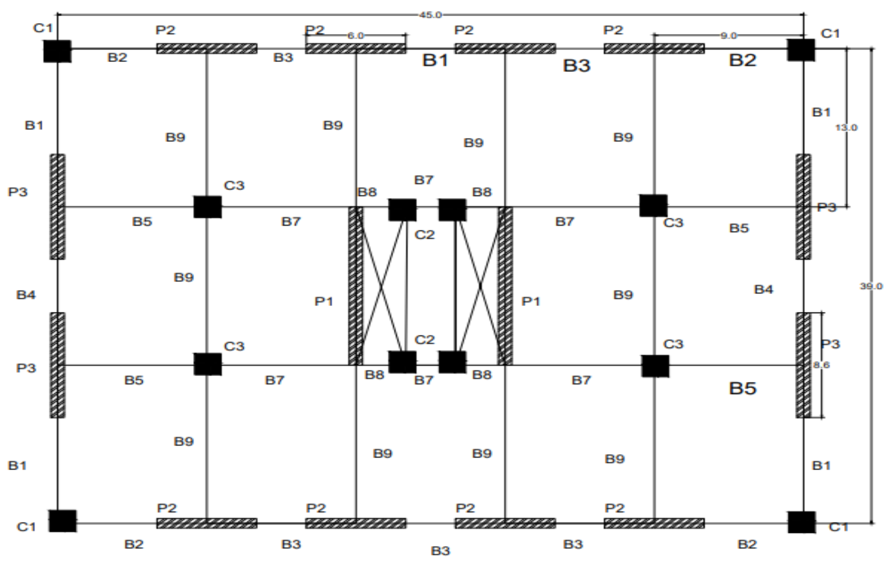


Figure 4.1 Shows the story plan.

Figure 4.2 3D view of the entire structure.

Figure 4.3 The building's location.

4.2 Structural system information of the building

The Etabs program is used to analyze and design the building. First, the program gave the structure of all of the information. The system models the beam and column as frame elements, the shear walls as shells, and the slabs as a shell. Then, as is said above, all loads are put on the system. First, the spectrum is calculated and assigned to the program for earthquake load. Then, the modal analysis is used to figure out the fundamental periods. Then slabs are thought of as the stiff diaphragm and the slabs' thickness is 20 cm on each story. The structure has exterior boundary shear walls and core shear walls at the center of the building. The shear wall thickness is 50cm in all of the building. The sizes of structure elements are given below in Table 4.1. The plan and elevation view of the building is given in Figure 4.1 and Figure 4.2.

Table 4.1 Frame element's names and dimensions in the building.

Exterior Beams		
Name	b (cm)	h (cm)
B1	50	70
B2	50	70
B3	50	70
B4	50	70
Interior Beams		
Name	b (cm)	h (cm)
B5	55	70
B6	45	65
B7	45	65
B8	45	65
B9	50	70
Columns		
Name	b (cm)	h (cm)
C1	95	95
C2	85	85
C3	85	85
Shear walls		
Name	b (cm)	a (cm)
P1	50	1300
P2	50	600
P3	50	860

4.3 Determining the Performance Levels of the Building

In order to determine the performance targets of the building, the S_S short period spectral acceleration coefficient for the DD-2 earthquake level and the S_1 1.0 second period spectral acceleration coefficients are taken from the Turkey Earthquake Hazard Maps.

$S_S = 0.678$

$S_1 = 0.199$

Map spectral acceleration coefficients will be converted into design spectral acceleration coefficients (S_{DS} andS_{D1}) by multiplying them with the local ground effect coefficients to be taken from Tables 4.2 and Table 4.3.

Table 4.2 Fs Table is given in TBDY 2018.

Soil Classification	F_s Factor For Short Period Area S_s					
	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1$	$S_s = 1.25$	$S_s \geq 1.50$
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	Specific site soil behavior analysis should be done.					

Table 4.3 F1 Table Is given In TBDY 2018

Soil Classification	F_1 Factor For Short Period Area S_1					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
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	Specific site soil behavior analysis should be done.					

F_S=0.900 for Local Ground Class ZB and S_S =0.678

F_1=0.800 for Local Ground Class ZB and S_1 =0.199

Design spectral acceleration coefficients:

$S_{DS} = S_S\ F_S = 0.678 \text{ x } 0.900 = 0.610$

$S_{D1} = S_1\ F_1 = 0.199 \text{ x } 0.800 = 0.159$

Earthquake design class, building usage class and DD-2 level will be determined from Table 3.4 depending on the short period design spectral acceleration coefficient.

Since the purpose of use of our building is residential, the building usage class (BKS) is 3. It also calculated our S_{DS} value as 0.610. As a result of these, DTS = 2 from Table 4.4.

Table 4.4 Seismic Design Classes.

Seismic Design Categories (DTS) S_{DS} (g)	Building Use Categories (BKS)	
	BKS = 1	BKS = 2, 3
SDS< 0.33	DTS = 4a	DTS = 4
$0.33 \leq S_{DS} < 0.50$	DTS = 3a	DTS = 3
$0.50 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

Depending on the determined building design class and building height, the building height class will be determined from Table 3.5 Since our DTS value is 2 and our total building height is 96 meters, the building height class found in Table 4.5 is 1. Only buildings with BYS=1 are accepted as tall structures in the Turkish Building Earthquake Code.

Table 4.5 Building height classification (BYS) given in TBDY 2018.

Building Height Categories (BYS)	Building Height Ranges Defined by Building Height Classes and Earthquake Design Classes [m]		
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a	DTS = 4, 4a
BYS = 1	$H_N > 70$	$H_N > 91$	$H_N > 105$
BYS = 2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
BYS = 3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
BYS = 4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
BYS = 5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
BYS = 6	$10.5 < H_N \leq 17,5$	$17.5 < H_N \leq 28$	
BYS = 7	$7 < H_N \leq 10,5$	$10.5 < H_N \leq 17,5$	
BYS = 8	$H_N \leq 7$	$H_N \leq 10,5$	

Finally, for the determined BYS=1 value, the performance targets and design approaches that the building should provide will be determined from Table 4.6.

Table 4.6 Performance targets and design procedures for new buildings.

Seismic ground motion level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a , 2a	
	Normal Performance	Check and Design Approach	Advanced performance	Check and Design Approach
DD-4	IO (KK)	SBD(DGT)	-	-
DD-3	-	-	LS (SH)	DBD(SGDT)
DD-2	CD (KH)	SBD(DGT)	CD (KH)	SBD(DGT)
DD-1	CP (GO)	DBD(SGDT)	CD (KH)	DBD(SGDT)

The building design selected as an application within the framework of the criteria determined in Table 4.6 will consist of three phases.

In design phase, I, the preliminary design and dimensioning of the structure will be made by applying the design rules according to the strength to achieve the controlled damage performance target. Calculations will be made for earthquake level DD-2.

In design phase II, the performance of the building will be evaluated by applying design rules according to strength to achieve the uninterrupted use performance target. Calculations will be made for earthquake level DD-4.

In design phase III, the structure's performance will be evaluated by applying design rules according to deformation to achieve the performance target of preventing migration. Calculations will be made for earthquake level DD-1.

4.4 Design Phase-I

In the first phase of the structural analysis, the creep calculation of the structure is made. In this calculation, the construction phases will be taken into account. Etabs program is used for all analyzes to be made. The creep calculation of the building is made in two phases, considering the construction phases and without considering the construction phases. The vertical story displacements are given in Figure 4.4. In the first analysis, the creep strain values are calculated by ignoring the time spent constructing the structure. In the second analysis, the creep strains are calculated with the assumption that the preparation phase of each story is 7 days and that it will wait for 3 days after concrete pouring. Figure 4.4 shows Creep calculation of vertical story displacements.

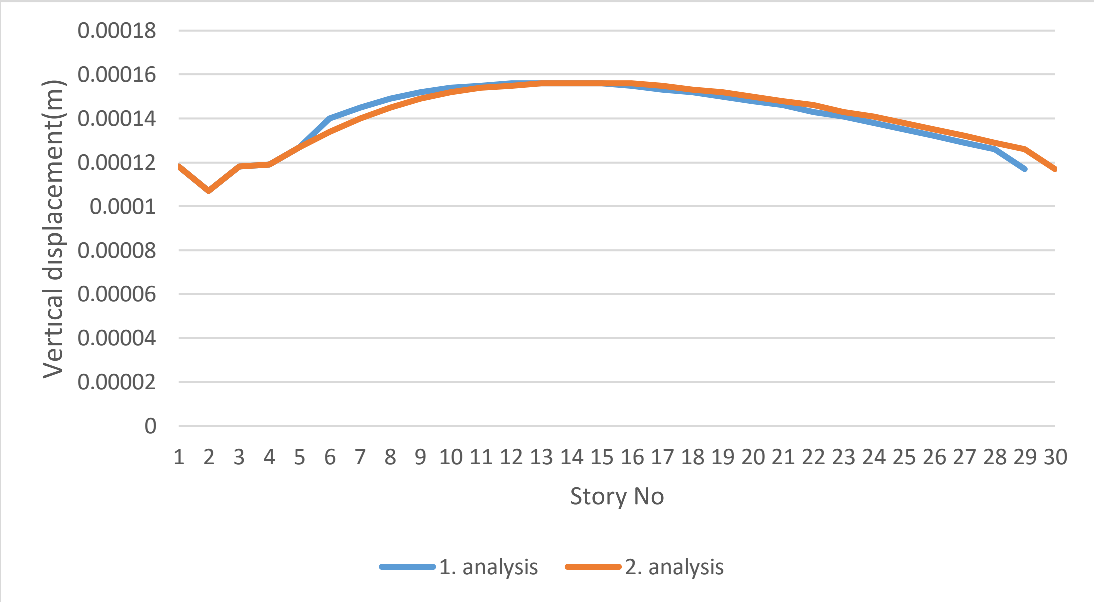


Figure 4.4 Creep calculation of vertical story displacements.

Considering the construction phases, it can be stated that further extension of the waiting and preparation times will increase these deformations. For this reason, shortenings in the vertical carrier system should be measured during the construction of each story, especially in high-rise buildings, and this shortening should be taken into account when necessary.

The wind calculation of the building is made according to the TS 498 regulation. In the regulation, it is foreseen that the wind speed will be 42 m/s for structures of 94 meters. The maximum story displacements obtained as a result of these calculations are given in Figure 4.5.

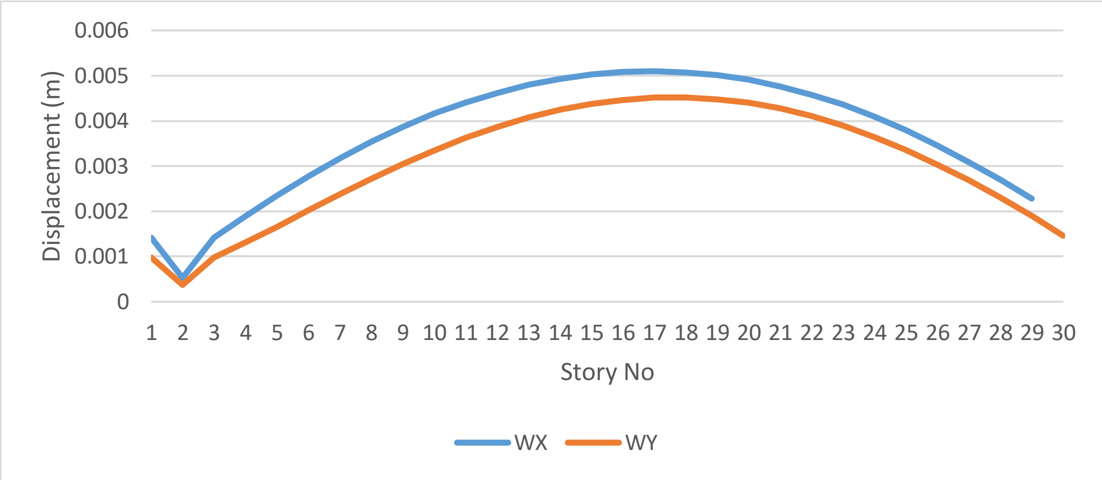


Figure 4.5 Wind calculation maximum floor displacements

4.4.1 Material Properties

C40/50 class concrete and B420C class reinforcing steel are used in all structural elements of the building.

- Characteristic strengths for C40/50;

  $f_{ck} = 40\ MPa$ (Characteristic compressive strength of concrete)

  $E_c = 32\ GPa$ (Module of elasticity of concrete)

  $f_{ctk} = 0.35\sqrt{f_{ck}} = 0.35\sqrt{40} = 2.2\ MPa$ (Characteristic tensile strength of concrete)

- Design strengths for C40/50;

  The material coefficient of concrete is $\gamma_{mc} = 1.50$

  $f_{cd} = \frac{40}{1.5} = 26.67\ MPa$ (Design compressive strength of concrete)

  $f_{ctd} = \frac{2.2}{1.5} = 1.47\ Mpa$ (Design tensile strength of concrete)

- Strengths for B420C reinforcing steel;

  $f_{yk} = R_e = 420\ MPa$ (Yield strength)

  Tensile Strength / Yield Strength $1.15 \leq f_{su}/f_{yk} \leq 1.35$

  $f_{su} = R_m = 525\ MPa$ (Tensile strength)

  $E_s = 200\ GPa$ (Module of elasticity)

  The material coefficient of reinforcing steel is $\gamma_{mc} = 1.15$

  $f_{yd} = 420/1.15 = 365.22\ MPa$

4.4.2 Earthquake Parameters

The earthquake level used for the first design phase of tall buildings is DD-2. The structure is located in Istanbul Province at latitude 41.131012° and longitude 29.017682°. The S_S and S_1 values for the horizontal elastic spectrum are taken from the Turkey Earthquake Hazard Maps.

$S_s = 0.678$

$S_1 = 0.199$

The local ground class is assumed to be ZB. The short-period local ground effect coefficient is 0.9 and the 1.0 second period local ground effect coefficient is 0.8 from Table 4.1 and Table 4.2. From here, the design spectral acceleration coefficients

$S_{DS} = S_s\, F_s = 0.678\, x\, 0.900 = 0.610$

$S_{D1} = S_1\, F_1 = 0.199\, x\, 0.800 = 0.159$

To plot the horizontal design spectrum, the corner periods will be calculated:

$T_A = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.159}{0.610} = 0.052\text{ s}$

$T_B = \frac{S_{D1}}{S_{DS}} = \frac{0.159}{0.610} = 0.261\text{ s}$

$T_L = 6\, s$

The horizontal elastic design spectrum is defined in Figure 4.6, depending on the natural vibration period:

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

$S_{ae}(T) = S_{DS}$ $\qquad (T_A \le T \le T_B)$

$S_{ae}(T) = \frac{S_{D1}}{T}$ $\qquad (T_B \le T \le T_L)$

$S_{ae}(T) = \frac{S_{D1} T_L}{T^2}$ $\qquad (T_L \le T)$

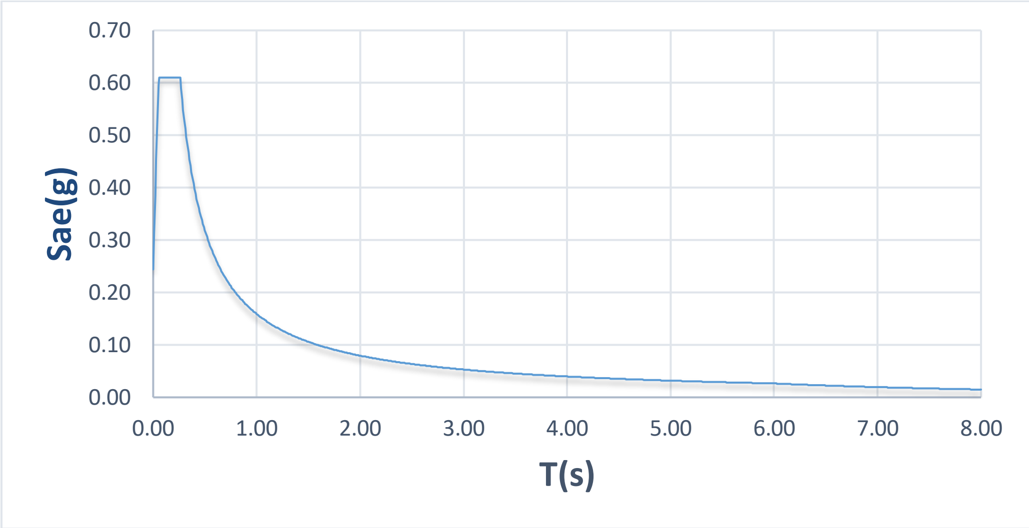


Figure 4.6 DD-2 earthquake level horizontal elastic design acceleration spectrum.

To create the vertical elastic design acceleration spectrum, SaeD (T) vertical elastic design spectral accelerations are determined depending on the period and the short period design spectral acceleration coefficient. The vertical elastic design spectrum is defined in Figure 4.7

$$S_{aeD}(T) = \left(0.32 + 0.48\frac{T}{T_{AD}}\right) S_{DS} \qquad 0 \leq T \leq T_{AD}$$

$$S_{aeD}(T) = 0.8 S_{DS} \qquad T_{AD} \leq T \leq T_{BD}$$

$$S_{aeD}(T) = 0.8 S_{DS} \frac{T_{BD}}{T} \qquad T_{BD} \leq T \leq T_{LD}$$

Calculation ofT_{AD}, T_{BD} and T_{LD}, which are the corner periods of the vertical spectrum:

$$T_{AD} = \frac{T_A}{3} \qquad ; T_{BD} = \frac{T_B}{3} \qquad ; T_{LD} = \frac{T_L}{2}$$

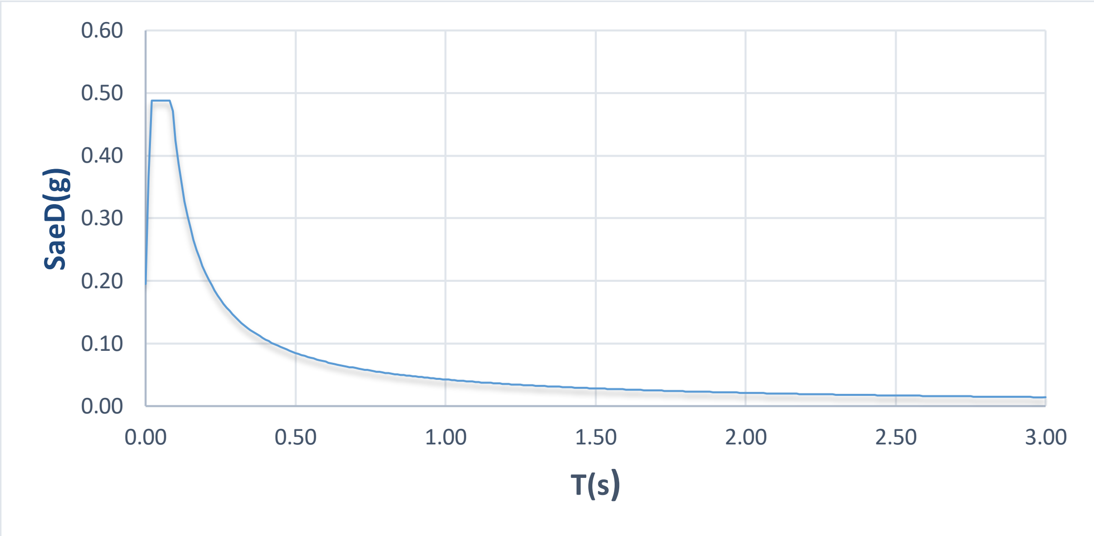


Figure 4.7 DD-2 earthquake level vertical elastic design acceleration spectrum.

4.4.3 Load and Load Combinations

- Loads to be used in the design:

  Fixed weight of reinforced concrete element: 25 KN/m^3

  Fixed pavement load: 2 KN/m^2

  Beam wall load: 6 KN/m

  Live load: 3.5 KN/m^2

  Snow load = 0.5 KN/m

- Load combinations to be used in the design:

  G: Dead Load.

  Q: Live Load.

  W: Wind Load

  E_X: Earthquake Load X Direction.

  E_Y: Earthquake Load Y Direction.

  EZ: Earthquake Load Z Direction.

  1.4G+1.6Q

  G+1.3Q+1.3WX

  G+1.3Q-1.3WX

  G+1.3Q+1.3WY

  G+1.3Q-1.3WY

0.9G+1.3WX

0.9G-1.3WX

0.9G+1.3WY

0.9G-1.3WY

G+1.2Q

G+Q+EX

G+Q-EX

G+Q+EY

G+Q-EY

0.9G+EX

0.9G-EX

0.9G+EY

0.9G-EY

: Vertical Earthquake Effect Ez ⇒ (2/3)*Sds*G

EX+0.3EY

EX-0.3EY

-EX-0.3EY

-EX+0.3EY

EY+0.3EX

EY-0.3EX

-EY-0.3EX

-EY+0.3EX

G+Q+0.2S+EX+0.3EY+0.3EZ

G+Q+0.2S+EX-0.3EY+0.3EZ

G+Q+0.2S-EX-0.3EY+0.3EZ

G+Q+0.2S-EX+0.3EY+0.3EZ

G+Q+0.2S-EY-0.3EX+0.3EZ

G+Q+0.2S-EY+0.3EX+0.3EZ

G+Q+0.2S+E_Y-0.3E_X+0.3EZ

G+Q+0.2S+EY+0.3EX+0.3EZ

0.9G+EX+0.3EY-0.3EZ

0.9G+EX-0.3EY-0.3EZ

0.9G-EX-0.3EY-0.3EZ

0.9G-EX+0.3EY-0.3EZ

0.9G+EY+0.3EX-0.3EZ

0.9G+EY-0.3EX-0.3EZ

0.9G-EY+0.3EX-0.3EZ

0.9G-EY-0.3EX-0.3EZ

4.4.4 Structural System Behavior Coefficient and Strength Excess Coefficient

The coefficient of the behavior of the structural system and the coefficient of excess strength are given in Table 4.7 according to the building height class. R = 6 and D = 2.5 are taken since all the earthquake effects in our building are covered by shear walls and BYS = 1.

Table 4.7 Structural system coefficient of behavior, coefficient of excess strength.

Building Carrier System	R	D	BYS
A. BUILT-IN-SITE REINFORCED CONCRETE BUILDING SYSTEMS			
A1. Structural Systems with High Ductility Levels			
A13. Buildings where reinforced concrete shears meet all the effects of earthquakes with a high ductility level	6	2,5	$BYS \geq 2$

4.4.5 Earthquake Load Reduction Coefficient

Earthquake load reduction coefficient R_a(T), Eq.s for different values of T. Calculated from Eq. (4.1).

$$R_a(T) = \frac{R}{I} \qquad T > T_B$$

$$R_a(T) = D + \left(\frac{R}{I} - D\right)\frac{T}{T_B} \qquad T \leq T_B \qquad 4.1$$

Earthquake load reduction coefficient with horizontal elastic design spectral accelerations Eq. By dimensioning with Eq. (4.2), the horizontal flexible design acceleration spectrum with reduced DD-2 earthquake level in Figure 4.8 is obtained.

$$S_{aR}(T) = \frac{S_{ae}(T)}{R_e(T)} \qquad 4.2$$

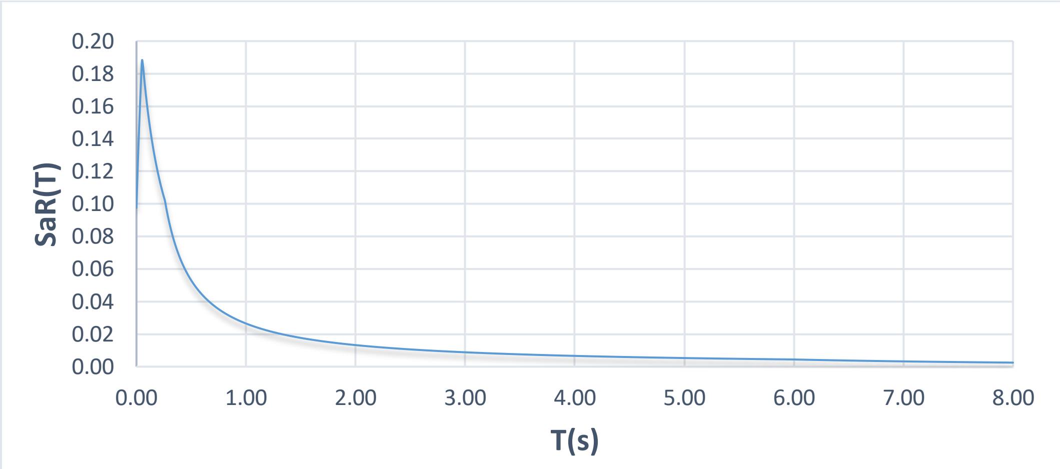


Figure 4.8 DD-2 earthquake level reduced horizontal elastic design acceleration spectrum.

4.4.6 Effective Section Rigidities

The values given in Table 4.8 are used in the modeling of the structural system elements in the design according to the strength.

Table 4.8 Rigidity factors for Strength-base design (DGT).

Concrete Member	**Effective Stiffness Multiplier**	
Shear Wall – Slab (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)	**Flexure**	**Shear**
Shear Wall	0.50	1.00
Basement Shear wall	0.25	1.00
Slab	0.50	1.00
Frame member	**Flexure**	**shear**
Coupling beam	0.15	1.00
Frame beam	0.35	1.00
Frame column	0.70	1.00
Wall (equivalent strut)	0.50	0.50

4.4.7 Story Masses and Weights

Since the purpose of use of the building to be analyzed is residential, the live load reduction coefficient (n) is taken as 0.30. Equivalent. The story masses and weights are calculated using the formula 4.3 given in Table 4.9.

$$w_j^{(s)} = w_{Gj}^{(s)} + n w_{Qj}^{(s)}$$

$$m_j^{(s)} = \frac{w_j^{(s)}}{g} \tag{4.3}$$

Table 4.9 Story weights and story masses.

Story No	Story Weight (ton)	Story Mass (ton)	Story No	Story Weight (ton)	Story Mass (ton)
30	2038.7749	207.826188	15	32620.3974	3325.2189
29	4077.5497	415.652365	14	34659.1723	3533.04509
28	6116.3245	623.478542	13	36697.9471	3740.87126
27	8155.0994	831.30473	12	38736.722	3948.69745
26	10193.8742	1039.13091	11	40775.4968	4156.52363
25	12232.649	1246.95708	10	42814.2717	4364.34982
24	14271.4239	1454.78327	9	44853.0465	4572.17599
23	16310.1987	1662.60945	8	46891.8214	4780.00218
22	18348.9735	1870.43563	7	48930.5962	4987.82836
21	20387.7484	2078.26181	6	50969.3711	5195.65455
20	22426.5232	2286.08799	5	53008.146	5403.48073
19	24465.2981	2493.91418	4	55046.9208	5611.30691
18	26504.0729	2701.74036	3	57085.6957	5819.1331
17	28542.8477	2909.56653	2	59456.3372	6060.78871
16	30581.6226	3117.39272	1	61826.9787	6302.44431
			Total	61826.9787	6302.44431

4.4.8 Determination of Linear Calculation Method

Within the scope of TBDY, it allowed using of the mod combination method for all buildings. In Table 4.10, the period and mass participation rates of the 30 modes of the structure are given.

Table 4.10 Period and mass participation rates.

Model	Period (s)	UX	UY	RZ	∑UX	∑UY
Mod 1	2.749	0	0.6908	0	0	0.6908
Mod 2	2.32	0.7554	0	0	0.7554	0.6908
Mod 3	1.815	0	0	0.7359	0.7554	0.6908
Mod 4	0.683	0.1264	0	0	0.8818	0.6908
Mod 5	0.642	0	0.1695	0	0.8818	0.8603
Mod 6	0.507	0	0	0.1335	0.8818	0.8603
Mod 7	0.34	0.0452	0	0	0.927	0.8603
Mod 8	0.27	0	0.0616	0	0.927	0.9219
Mod 9	0.238	0	0	0.0524	0.927	0.9219
Mod 10	0.208	0.0242	0	0	0.9513	0.9219
Mod 11	0.153	0	0.0299	5.462E-07	0.9513	0.9519
Mod 12	0.14	0.0141	0	0	0.9654	0.9519
Mod 13	0.139	0	6.904E-07	0.0275	0.9654	0.9519
Mod 14	0.102	0.000001171	0.016	6.624E-07	0.9654	0.9678
Mod 15	0.102	0.0089	0.000002072	0	0.9744	0.9678
Mod 16	0.092	0	8.865E-07	0.0156	0.9744	0.9678
Mod 17	0.078	0.006	0	0	0.9804	0.9678
Mod 18	0.075	0	0.0093	8.268E-07	0.9804	0.9771
Mod 19	0.067	0	0.000001277	0.0096	0.9804	0.9771
Mod 20	0.063	0.0044	0	0	0.9848	0.9771
Mod 21	0.059	0	0.0059	0.000001695	0.9848	0.983
Mod 22	0.052	0.0035	0	8.058E-07	0.9882	0.983
Mod 23	0.051	0	0.000004026	0.0063	0.9882	0.983
Mod 24	0.049	0	0.0041	0.000005511	0.9882	0.9871
Mod 25	0.045	0.003	0	0	0.9912	0.9871
Mod 26	0.042	0	0.0031	0.0001	0.9912	0.9901
Mod 27	0.041	0	0.0001	0.0043	0.9912	0.9902
Mod 28	0.039	0.0027	0	0	0.9939	0.9902
Mod 29	0.037	0	0.0026	0.000001347	0.9939	0.9928
Mod 30	0.035	0.0023	0	0.000001176	0.9962	0.9928

4.4.9 Scaling Mod Combine Accounts

Magnification of the reduced internal forces and displacements relative to the equivalent base shear force will be calculated according to the minimum base shear force given in Eq.(4.4).

$$V_{t,min} = 0.04 \propto_H S_{DS} W \tag{4.4}$$

The α_H coefficient will be calculated by Eq.(3.5) depending on the building height.

$$\begin{aligned} \alpha_H &= 1 & H_N &\leq 105\,m \\ \alpha_H &= 2.05 - 0.01 H_N & 105\,m &< H_N \leq 155\,m \\ \alpha_H &= 0.5 & H_N &< 155\,m \end{aligned} \tag{4.5}$$

Since our total building height is 94 meters, the first relation will be valid.

$\alpha_H = 1$

$V_{t,min} = 0.04 \propto_H S_{DS} W = 0.04 \times 1 \times 0.610 \times 618269.787 = 15085.78\ kN$

The base shear forces obtained by the mode coupling method are $V_{tx} = 7775.24\ kN$ and $V_{ty} = 5540.048\ kN$. If the equivalent base shear force coefficient is calculated from here:

$$\beta_{tE,x} = \frac{\gamma_E V_{t,min}}{V_{t,x}} = \frac{1\text{x}15085.78}{16051.77} = 0.94 \leq 1$$

$$\beta_{tE,y} = \frac{\gamma_E V_{t,min}}{V_{t,y}} = \frac{1\text{x}15085.78}{16051.77} = 0.94 \leq 1$$

γ_E value will be taken 1 for high structures.

The base shear forces found by the mode coupling method in the mathematical model don't need to be amplified by the calculated $\beta_{tE,x}$ and $\beta_{tE,y}$ values.

4.4.10 Calculation of Relative Story Offsets

In the calculation of the relative storey drifts of the building, firstly, the displacement differences of any column or shear wall between two consecutive storeys in X- and Y-directed earthquakes are calculated with Eq.(4.6). This calculated value is the reduced relative storey drift.

$$\Delta_i^{(x)} = u_i^{(x)} - u_{i-1}^{(x)} \tag{4.6}$$

Effective relative story drifts will be obtained by using the calculated reduced relative story drift value Eq.(4.7).

$$\delta_i^{(x)} = \frac{R}{I} \Delta_i^{(x)} \tag{4.7}$$

The maximum of $\delta_i^{(x)}$values calculated for columns and shear walls on each story will be determined and compared with the limit values given in Eq. (4.8). Limit values are divided into two according to whether the flexible joint is used or not in connection of infill walls to load-bearing system elements.

$$\lambda \frac{\delta_{i,max}^{(x)}}{h_i} \leq 0.008\kappa \text{ (No flexible joint applied)}$$

$$\lambda \frac{\delta_{i,max}^{(x)}}{h_i} \leq 0.016\kappa \text{ (Flex joint applied)} \tag{4.8}$$

The λ coefficient given in the Eq. is defined in Eq. (4.9) as the ratio of the DD3 level of the dominant vibration period in the earthquake direction and the elastic design spectral acceleration values in the DD2 level earthquakes. The κ coefficient will be used as 1.0 for reinforced concrete buildings. Figure 4.9 shows Reduced relative story offsets.

$$\lambda = \frac{S_{ae-DD3}(T_{PX})}{S_{ae-DD2}(T_{PX})} \tag{4.9}$$

(a) X-X Direction Relative Story Offset Calculation

X-X direction dominant vibration period = 2.325

$S_{ae-DD3}(T_{PX}) = 0.028$, $S_{ae-DD2}(T_{PX}) = 0.068$,

$\lambda = \frac{0.028}{0.068} = 0.412$

Table 4.11 X-X direction relative story drifts.

Story No	h (m)	h (mm)	$\Delta_i^{(x)}$ (mm)	$\delta_i^{(x)}$ (mm)	$\delta_i^{(x)}$ / h	λ ($\delta_i^{(x)}$ / h)
1	5	5000	1.092	6.552	0.00131	0.0005399
2	5	5000	2.387	14.322	0.002864	0.0011801
3	3	3000	1.834	11.004	0.003668	0.0015112
4	3	3000	2.059	12.354	0.004118	0.0016966
5	3	3000	2.233	13.398	0.004466	0.00184
6	3	3000	2.37	14.22	0.00474	0.0019529
7	3	3000	2.474	14.844	0.004948	0.0020386
8	3	3000	2.553	15.318	0.005106	0.0021037
9	3	3000	2.608	15.648	0.005216	0.002149
10	3	3000	2.647	15.882	0.005294	0.0021811
11	3	3000	2.669	16.014	0.005338	0.0021993
12	3	3000	2.678	16.068	0.005356	0.0022067
13	3	3000	2.675	16.05	0.00535	0.0022042
14	3	3000	2.661	15.966	0.005322	0.0021927
15	3	3000	2.638	15.828	0.005276	0.0021737
16	3	3000	2.607	15.642	0.005214	0.0021482
17	3	3000	2.568	15.408	0.005136	0.002116
18	3	3000	2.523	15.138	0.005046	0.002079
19	3	3000	2.471	14.826	0.004942	0.0020361
20	3	3000	2.414	14.484	0.004828	0.0019891
21	3	3000	2.353	14.118	0.004706	0.0019389
22	3	3000	2.288	13.728	0.004576	0.0018853
23	3	3000	2.22	13.32	0.00444	0.0018293
24	3	3000	2.151	12.906	0.004302	0.0017724
25	3	3000	2.081	12.486	0.004162	0.0017147
26	3	3000	2.012	12.072	0.004024	0.0016579
27	3	3000	1.947	11.682	0.003894	0.0016043
28	3	3000	1.888	11.328	0.003776	0.0015557
29	3	3000	1.838	11.028	0.003676	0.0015145
30	3	3000	1.793	10.758	0.003586	0.0014774

Since the relative story offsets calculated for the X-X earthquake direction in Table 4.11 are below the value of 0.008, there is no need to use flexible joints between the structural system elements of the building and the infill walls or facade elements. Relative story drifts in X-X directions are presented graphically in Figure 4.10.

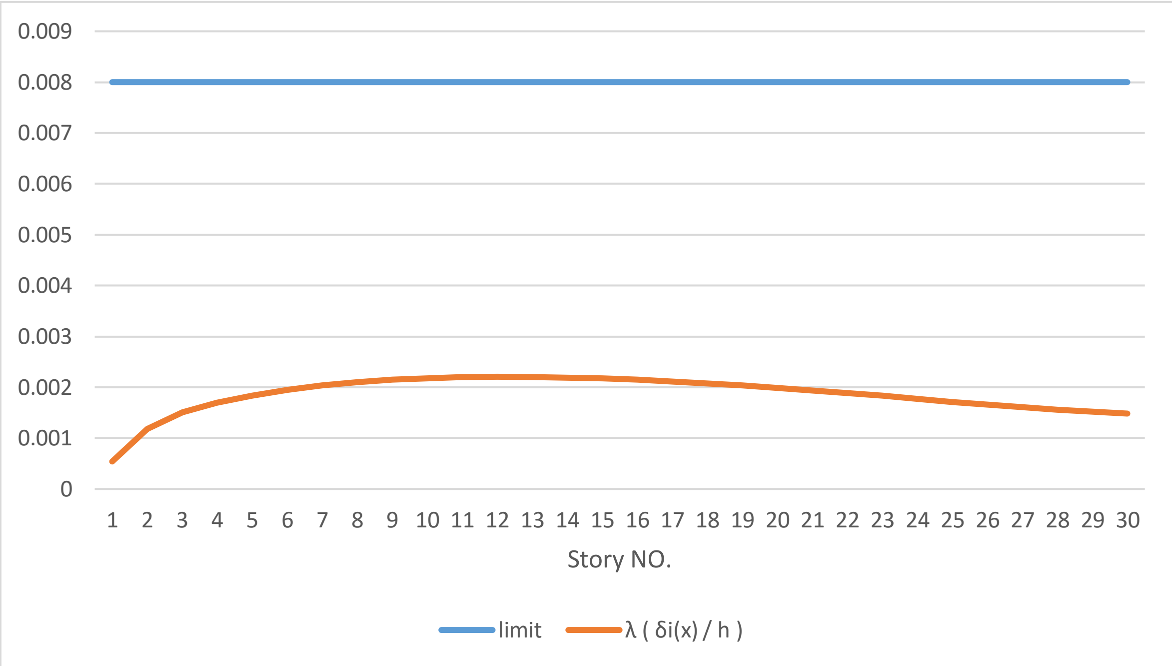


Figure 4.10 The relative story drifts in X-X directions.

(b) Y-Y Direction Relative Story Offsets Calculation

Y-Y direction dominant vibration period = 2.749

$S_{ae-DD3}(T_{PX}) = 0.024, S_{ae-DD2}(T_{PX}) = 0.058$

$$\lambda = \frac{0.024}{0.058} = 0.414$$

Table 4.12 Y-Y direction relative storey drifts.

Story No	h (m)	h (mm)	$\Delta_i^{(y)}$ (mm)	$\delta_i^{(y)}$ (mm)	$\delta_i^{(y)}$ / h	λ ($\delta_i^{(y)}$ / h)
1	5	5000	0.959	5.754	0.001151	0.000478
2	5	5000	2.207	13.242	0.002648	0.001099
3	3	3000	1.837	11.022	0.003674	0.001525
4	3	3000	2.181	13.086	0.004362	0.001811
5	3	3000	2.487	14.922	0.004974	0.002065
6	3	3000	2.763	16.578	0.005526	0.002294
7	3	3000	3.009	18.054	0.006018	0.002498
8	3	3000	3.227	19.362	0.006454	0.002679
9	3	3000	3.419	20.514	0.006838	0.002838
10	3	3000	3.587	21.522	0.007174	0.002978
11	3	3000	3.732	22.392	0.007464	0.003098
12	3	3000	3.857	23.142	0.007714	0.003202
13	3	3000	3.961	23.766	0.007922	0.003288
14	3	3000	4.048	24.288	0.008096	0.003361
15	3	3000	4.116	24.696	0.008232	0.003417
16	3	3000	4.17	25.02	0.00834	0.003462
17	3	3000	4.209	25.254	0.008418	0.003494
18	3	3000	4.235	25.41	0.00847	0.003516
19	3	3000	4.249	25.494	0.008498	0.003527
20	3	3000	4.251	25.506	0.008502	0.003529
21	3	3000	4.245	25.47	0.00849	0.003524
22	3	3000	4.23	25.38	0.00846	0.003512
23	3	3000	4.209	25.254	0.008418	0.003494
24	3	3000	4.182	25.092	0.008364	0.003472
25	3	3000	4.15	24.9	0.0083	0.003445
26	3	3000	4.117	24.702	0.008234	0.003418
27	3	3000	4.082	24.492	0.008164	0.003389
28	3	3000	4.04	24.24	0.00808	0.003354
29	3	3000	4.012	24.072	0.008024	0.003331
30	3	3000	3.981	23.886	0.007962	0.003305

Since the relative story offsets calculated for the Y-Y earthquake direction in Table 4.12 are below the value of 0.008, there is no need to use flexible joints between the structural system elements of the building and the infill walls or facade elements. Relative story drifts in Y-Y directions are presented graphically in Figure 4.11..

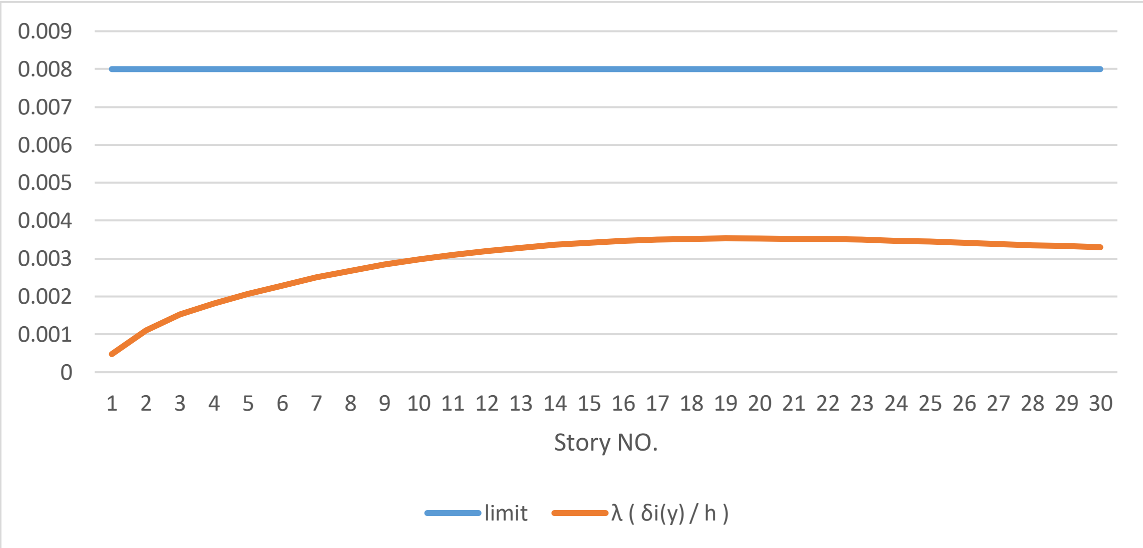


Figure 4.11 Relative story drifts in Y-Y directions.

4.4.11 Control of Second-Order Effects

Eq. (4.10) will calculate the second-order indicator value for all stories in both earthquake directions.

$$\theta_{I,II}^{(X)} = \frac{(\Delta_i^{(x)})_{average} \sum_{k=i}^{N} W_K}{V_i^{(x)} h_i} \tag{4.10}$$

$(\Delta_i^{(x)})_{average}$:Average of storey drifts relative to the reduced in story

$\sum_{k=i}^{N} W_K$:Total seismic weight $\qquad V_i^{(x)}$:Story shear force

The calculated second order indicator values will be compared with the maximum $\theta_{II,max}^{(X)}$ value to be calculated by Eq. (4.11) in Tables 4.13 and Table 4.14.

$$\theta_{I,II}^{(X)} \leq 0.12 \frac{D}{C_h R} \tag{4.11}$$

C_h value will be taken as 0.5 for reinforced concrete buildings.

Table 4.13 X-X direction second order calculation.

Story No.	h (m)	$(\Delta_i^{(x)})_{av.}$	$W_{K,i}(T)$	$\sum_{k=i}^{N} W_K$	$V_i(T)$	$\theta_{I,II}$	$\leq 0.12\frac{D}{C_h R}$
30	3	1.793	2038.775	2038.775	71.2798	0.017095	0.1
29	3	1.838	2038.775	4077.55	148.7558	0.016794	0.1
28	3	1.888	2038.775	6116.325	223.4159	0.017229	0.1
27	3	1.947	2038.775	8155.099	295.2285	0.017927	0.1
26	3	2.012	2038.775	10193.87	364.156	0.018774	0.1
25	3	2.081	2038.775	12232.65	430.1579	0.019726	0.1
24	3	2.151	2038.775	14271.42	493.1926	0.020748	0.1
23	3	2.22	2038.775	16310.2	553.2182	0.021817	0.1
22	3	2.288	2038.775	18348.97	610.1951	0.022934	0.1
21	3	2.353	2038.775	20387.75	664.0868	0.024079	0.1
20	3	2.414	2038.775	22426.52	714.8621	0.025244	0.1
19	3	2.471	2038.775	24465.3	762.4959	0.026428	0.1
18	3	2.523	2038.775	26504.07	806.9709	0.027622	0.1
17	3	2.568	2038.775	28542.85	848.2791	0.028803	0.1
16	3	2.607	2038.775	30581.62	886.4222	0.029981	0.1
15	3	2.638	2038.775	32620.4	921.4138	0.031131	0.1
14	3	2.661	2038.775	34659.17	953.2799	0.032249	0.1
13	3	2.675	2038.775	36697.95	982.0601	0.03332	0.1
12	3	2.678	2038.775	38736.72	1007.809	0.034311	0.1
11	3	2.669	2038.775	40775.5	1030.596	0.0352	0.1
10	3	2.647	2038.775	42814.27	1050.51	0.03596	0.1
9	3	2.608	2038.775	44853.05	1067.655	0.036521	0.1
8	3	2.553	2038.775	46891.82	1082.154	0.036875	0.1
7	3	2.474	2038.775	48930.6	1094.154	0.036879	0.1
6	3	2.37	2038.775	50969.37	1103.819	0.036479	0.1
5	3	2.233	2038.775	53008.15	1111.339	0.035503	0.1
4	3	2.059	2038.775	55046.92	1116.925	0.033825	0.1
3	3	1.834	2038.775	57085.7	1120.815	0.031137	0.1
2	5	2.387	2370.642	59456.34	1123.459	0.025265	0.1
1	5	1.092	2370.642	61826.98	1124.313	0.01201	0.1

Table 4.14 Y-Y direction second order calculation.

Story No.	h (m)	$(\Delta_i^{(x)})_{av.}$	$W_{K,i}(t)$	$\sum_{k=i}^{N} W_K$	$V_i(t)$	$\theta_{I,II}$	$\leq 0.12\frac{D}{C_h R}$
30	3	3.981	2038.775	2038.775	71.2798	0.037955	0.1
29	3	4.012	2038.775	4077.55	148.7558	0.036658	0.1
28	3	4.04	2038.775	6116.325	223.4159	0.036867	0.1
27	3	4.082	2038.775	8155.099	295.2285	0.037586	0.1
26	3	4.117	2038.775	10193.87	364.156	0.038416	0.1
25	3	4.15	2038.775	12232.65	430.1579	0.039339	0.1
24	3	4.182	2038.775	14271.42	493.1926	0.040338	0.1
23	3	4.209	2038.775	16310.2	553.2182	0.041364	0.1
22	3	4.23	2038.775	18348.97	610.1951	0.0424	0.1
21	3	4.245	2038.775	20387.75	664.0868	0.043441	0.1
20	3	4.251	2038.775	22426.52	714.8621	0.044454	0.1
19	3	4.249	2038.775	24465.3	762.4959	0.045444	0.1
18	3	4.235	2038.775	26504.07	806.9709	0.046365	0.1
17	3	4.209	2038.775	28542.85	848.2791	0.047208	0.1
16	3	4.17	2038.775	30581.62	886.4222	0.047955	0.1
15	3	4.116	2038.775	32620.4	921.4138	0.048572	0.1
14	3	4.048	2038.775	34659.17	953.2799	0.049059	0.1
13	3	3.961	2038.775	36697.95	982.0601	0.049339	0.1
12	3	3.857	2038.775	38736.72	1007.809	0.049417	0.1
11	3	3.732	2038.775	40775.5	1030.596	0.049219	0.1
10	3	3.587	2038.775	42814.27	1050.51	0.04873	0.1
9	3	3.419	2038.775	44853.05	1067.655	0.047878	0.1
8	3	3.227	2038.775	46891.82	1082.154	0.046611	0.1
7	3	3.009	2038.775	48930.6	1094.154	0.044854	0.1
6	3	2.763	2038.775	50969.37	1103.819	0.042528	0.1
5	3	2.487	2038.775	53008.15	1111.339	0.039541	0.1
4	3	2.181	2038.775	55046.92	1116.925	0.03583	0.1
3	3	1.837	2038.775	57085.7	1120.815	0.031188	0.1
2	5	2.207	2370.642	59456.34	1123.459	0.02336	0.1
1	5	0.959	2370.642	61826.98	1124.313	0.010547	0.1

If $\theta_{II,max}^{(X)}$ value is below the limit value in both earthquake directions. For this reason, second-order effects will not be taken into account in the internal force calculations.

If the maximum value of $\theta_{II,max}^{(X)}$ calculated for all stories does not satisfy the condition given in Eq. (4.11), all internal forces for the earthquake direction (X) considered are calculated with the second order amplification factor $\beta_{\mathbb{I}}^{(x)}$ defined by Eq. (3.12) below will be mmultiplied by

$$\beta_{\mathbb{I}}^{(x)} = 0.88 + \frac{C_h R}{D}\theta_{II,max}^{(X)} \geq 1 \qquad (4.12)$$

4.4.12 Reinforcement of Structural System

Due to the large number of structural system elements, the minimum reinforcement tables for beam, colon and shear wall in Figure 4.12 are presented in the appendices. Rules in TS 500 and TBDY2018 are applied by using Doğangün (2019) resource for element reinforcement calculations.

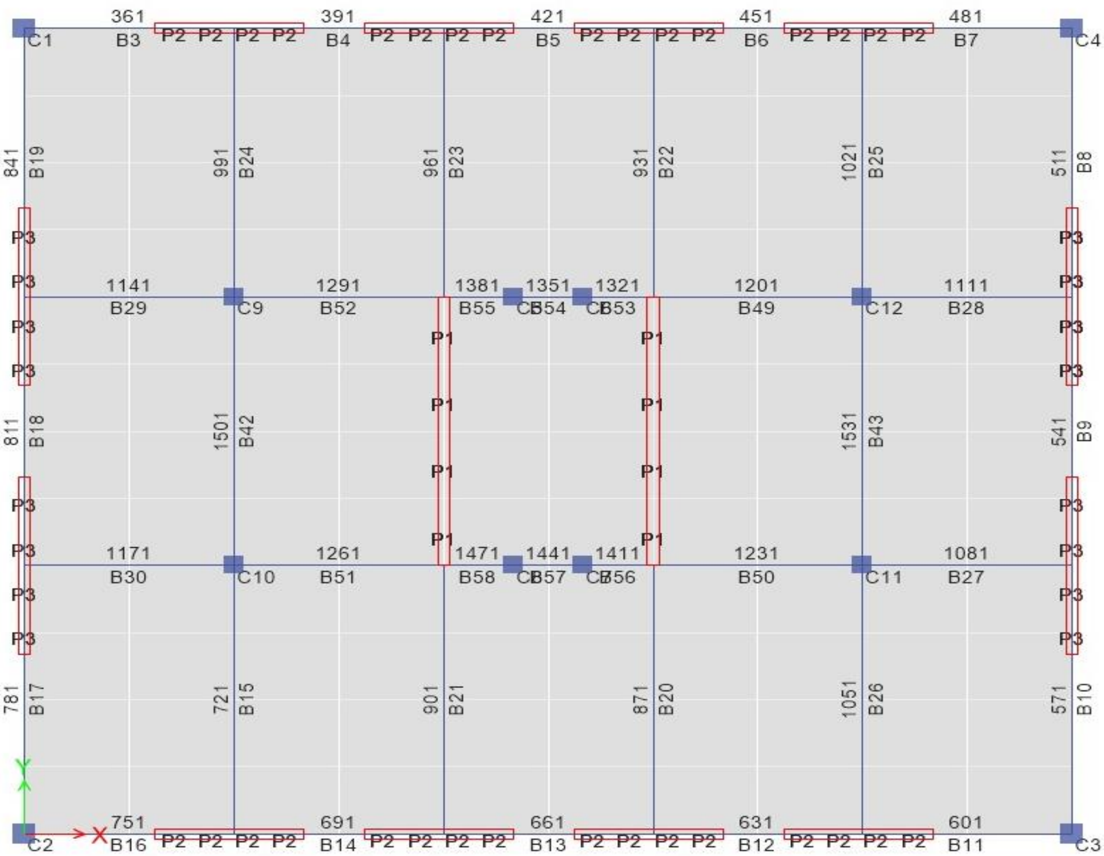


Figure 4.12 Reinforced concrete elements.

4.4.13 Column Reinforcements

For the design of the columns, first of all, the maximum axial force that will occur in the columns under the earthquake effect will be determined. It will be checked that this force provides the $Ac \geq N_{dm}/(0.40f_{ck})$ condition that the section can bear. Figure 4.13 shows that the limit value of 0.40 for maximum axial forces is not exceeded.

The longitudinal reinforcement area in columns shall not be less than 1% and greater than 4% of the gross section. Reinforcement thinner than φ14 in columns and less than 6 in circular columns shall not be used.

We examine whether some columns in the Structure to the *TBDY2018-7.3.1.2* Condition. The maximum column axial force is obtained from the Etabs Program and it is seen that the columns carried the loads safely.

It is found from the Sap2000 Program that the longitudinal reinforcement areas of all columns are 9025 mm2. In other words, when choosing the column longitudinal reinforcement, the reinforcement with an area of at least 90.25 cm2 should be selected. Now it finds the column longitudinal reinforcements in % from the Sap2000 program.

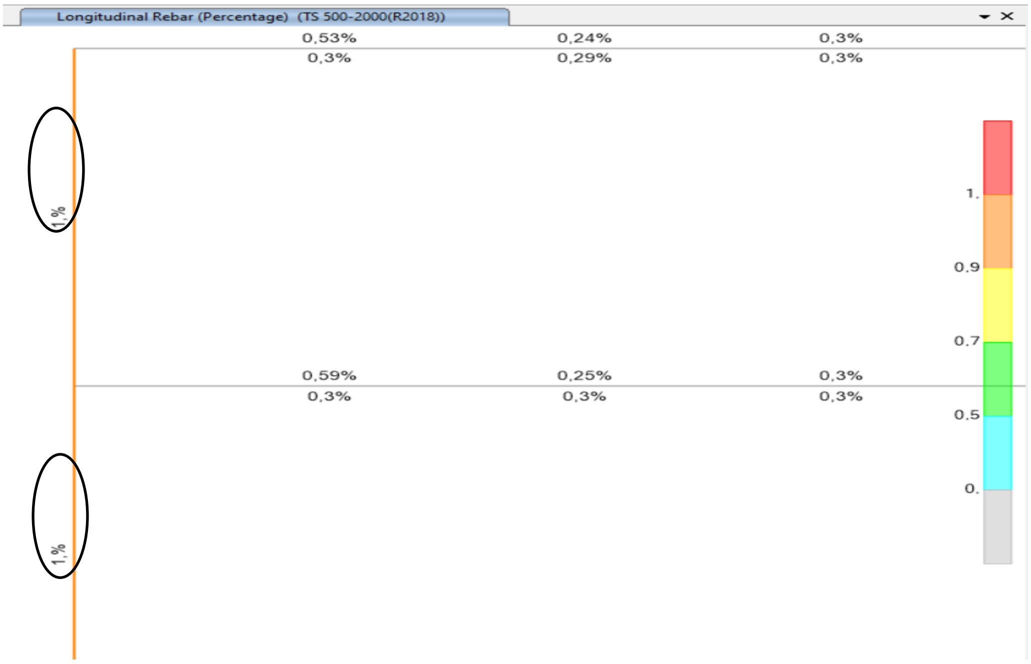


Figure 4.13 Reinforcement percentages of columns from the Sap2000 program

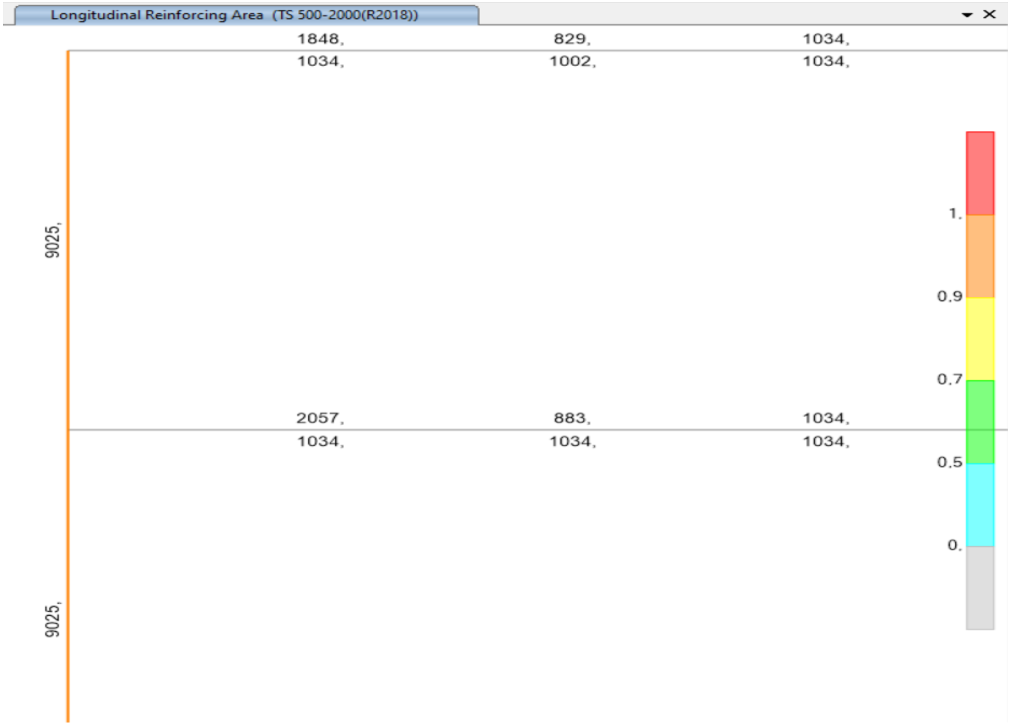


Figure 4.14 Reinforcement area of columns from the Sap2000 program

Reinforcement percentages of all columns from the Sap2000 program are 1% as seen in the figure:4.13 above. In other words, the columns in the structure meet the *TBDY2018-7.3.2* Longitudinal Reinforcement Conditions.

a) Reinforcing of 95cmx95cm Columns

The most suitable reinforcement diameter is determined according to the reinforcement table. While reinforcing the columns, we will use 22 mm reinforcement, ie *φ22* reinforcement, as the reinforcement diameter. 1 ϕ22 reinforcement area 3.80 cm2 ⇒ 24Ø22. Since 24x3.80 = 91.2 cm2 ⇒ 91.2 cm2 ≥ 90.25 cm2, the selected reinforcement is appropriate. Selected reinforcement is 24Ø22.

In this case, the longitudinal reinforcement ratio = $\frac{91,2}{95*95} = 0,01010526$. The reinforcement rate is 1.01% ≥ 1% ⇒ Reinforcement rate is between the limit values of 4% and 1%. In the calculation of column transverse reinforcement, that is, in the calculation of column stirrups, the required shear reinforcement area is determined as 1300 mm²/m. Column stirrup is to be chosen as 4-armed. Column stirrup diameter is determined as Ø8.

Column stirrup is to be determined as 4-armed Ø8/15/10. In other words, stirrups are placed at the junction of columns and beams, in other words, with 10 cm intervals in the compaction regions, and at 15 cm intervals in the middle regions. Figure 4.15 shows 95 cm x 95 cm Column Reinforcements and Stirrups

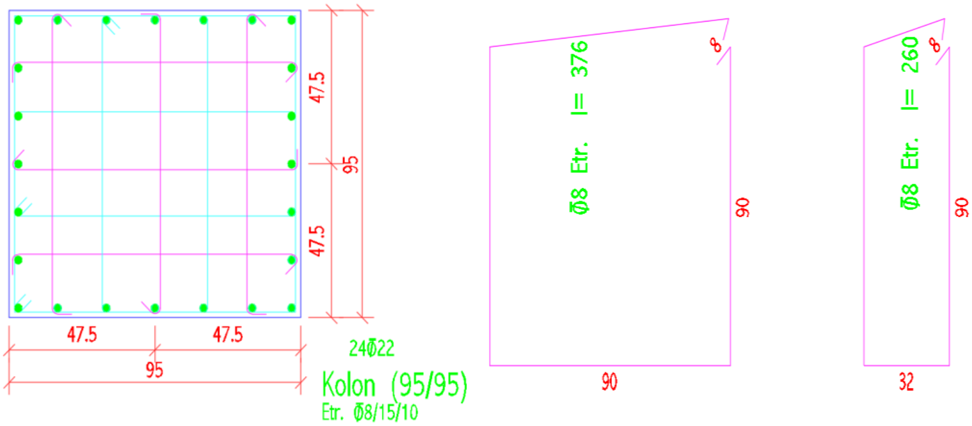


Figure 4.15 95 cm x 95 cm Column Reinforcements and Stirrups.

b) Reinforcing 85cmx85cm Columns

1 ϕ22 reinforcement area 3.80 cm2 ⇒ 24Ø22. Since 24x3,80 = 91.2 cm2 ⇒ 91.2 cm2 ≥ 90.25 cm2, the selected reinforcement is appropriate. Selected reinforcement is 24Ø22.

In this case, the longitudinal reinforcement ratio = $91.2/85 * 85 = 0.01262284$. The reinforcement rate is %1.262284 ≥ %1 ⇒ Reinforcement rate is between the limit values of 4% and 1%. In the calculation of column transverse reinforcement, that is, in the calculation of column stirrups, the required shear reinforcement area is determined as 1300 mm²/m. Column stirrup is to be chosen as 4-armed. Column stirrup diameter is determined as Ø8.

Column stirrup is to be determined as 4-armed Ø8/15/10. In other words, stirrups are placed at the junction of columns and beams, in other words, with 10 cm intervals in the compaction regions, and at 15 cm intervals in the middle regions Figure 4.16 shows 85 cm x 85 cm Column Reinforcements and Stirrups.

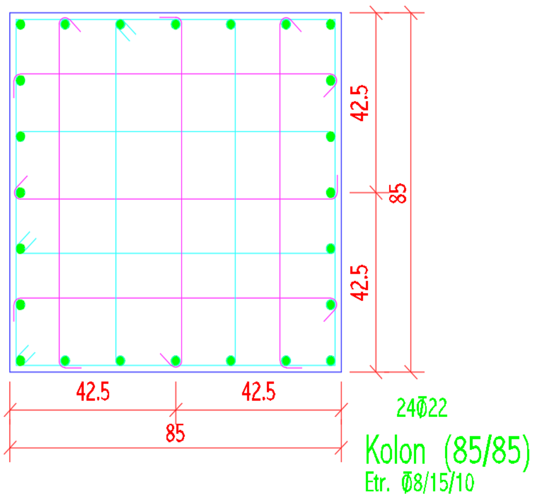


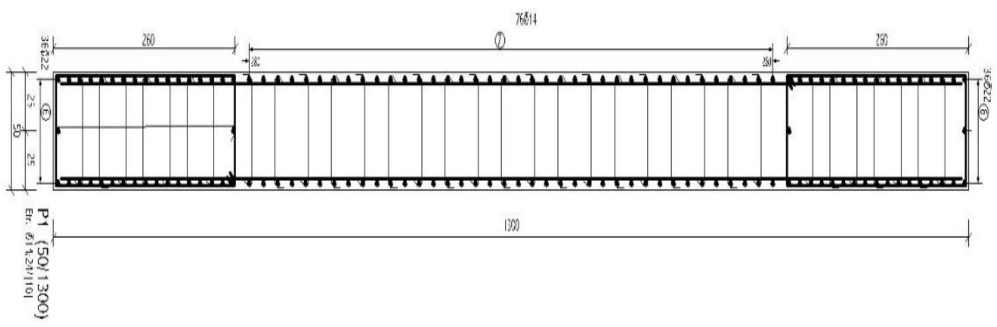
Figure 4.16 85 cm x 85 cm Column Reinforcements and Stirrups.

4.4.14 Shear Walls Reinforcement

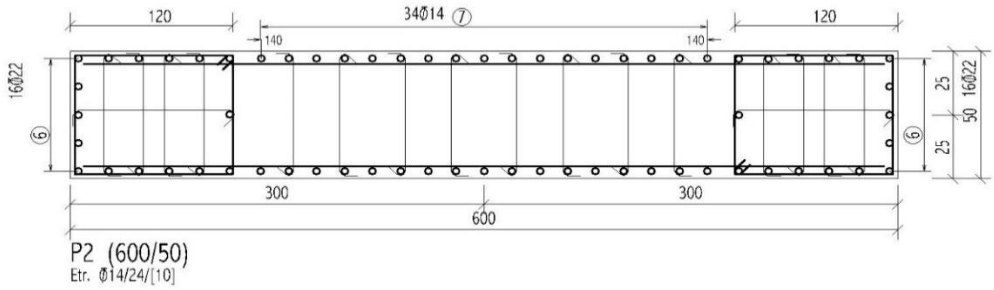
For the design of the walls, first of all, the maximum axial force that will occur on the walls under the earthquake effect will be determined. This force can be carried by the section $\geq N_{dm}/(0.35 f_{ck})$. It is given in Table 4.15. All the reinforcements shown will be formed on the shear walls.

TBDY-2018 for shear walls provides the conditions given. There are 3 types of shear walls in the building:

a) Reinforcements of shear walls in 50 cm X 1300 cm Dimensions



b) Reinforcements of shear walls in 50 cm X 1300 cm Dimensions



c) **Reinforcements of shear walls in 50 cm X 1300 cm Dimensions**

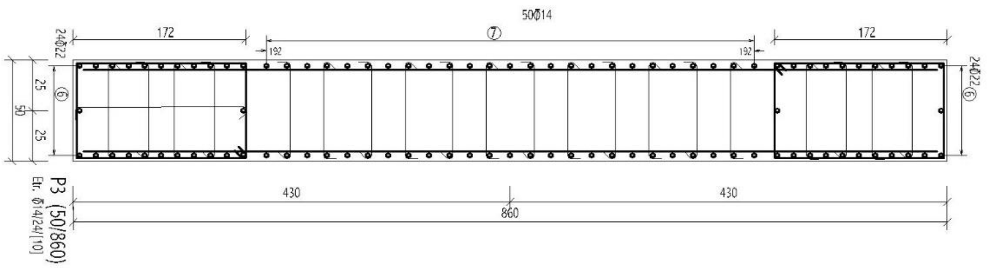


Table 4.15 Shear wall longitudinal reinforcements.

Shear wall dimensions (cm)	Shear wall end area		Shear wall body area		Reinforcement Area (As)
	Length (cm)	Reinforcement	Length(cm)	Reinforcement	
50×600	120	16Ø22	360	34Ø14	5231.24
50×860	172	24Ø22	516	50Ø14	7693
50×1300	260	36Ø22	780	76Ø14	11693.36

4.4.15 Beams Reinforcement

There are a total of 1560 beam elements in the building. Due to the excess of beam elements, typing is preferred while reinforcing. The names and dimensions of the beam elements are shown in the formwork plan given in the figure below. The longitudinal reinforcements defined to these elements are shown in Table 4.16 in detail. There are 3 types of beam elements in the structure. Separate reinforcement will be made for all 3 types of beams.

Table 4.16 All selected beam reinforcements.

Story No		Beam Dimensions	50 x 70 cm	Reinforcement Area (As)	55 x 70 cm	Reinforcement Area (As)	45 x 65 cm	Reinforcement Area(As)
30	Top	Left	6Ø20	1884	8Ø20	2512	8Ø18	2034.72
		Middle	6Ø14	923.16	5Ø14	769.3	7Ø14	1077.02
		Right	7Ø14	1077.02	7Ø20	2198	6Ø32	4823.04
	Bottom	Left	7Ø14	1077.02	6Ø14	923.16	7Ø16	1406.72
		Middle	7Ø14	1077.02	9Ø18	2289.06	8Ø20	2512
		Right	7Ø14	1077.02	8Ø14	1230.88	8Ø14	1230.88
20	Top	Left	6Ø22	2279.64	8Ø20	2512	8Ø20	2512
		Middle	7Ø14	1077.02	8Ø12	904.32	6Ø16	1205.76
		Right	7Ø16	1406.72	5Ø26	2653.3	7Ø32	5626.88
	Bottom	Left	6Ø16	1205.76	5Ø16	1004.8	7Ø16	1406.72
		Middle	7Ø14	1077.02	8Ø20	2512	5Ø26	2653.3
		Right	4Ø20	1256	8Ø14	1230.88	7Ø16	1406.72
10	Top	Left	7Ø20	2198	4Ø26	2122.64	8Ø22	3039.52
		Middle	5Ø16	1004.8	5Ø16	1004.8	5Ø16	1004.8
		Right	7Ø18	1780.38	6Ø26	3183.96	6Ø32	4823.04
	Bottom	Left	4Ø20	1256	5Ø16	1004.8	6Ø16	1205.76
		Middle	7Ø14	1077.02	8Ø20	2512	8Ø20	2512
		Right	7Ø16	1406.72	6Ø16	1205.76	6Ø16	1205.76
1	Top	Left	7Ø14	1077.02	7Ø16	1406.72	6Ø28	3692.64
		Middle	2Ø16	401.92	6Ø16	1205.76	6Ø14	923.16
		Right	6Ø16	1205.76	8Ø26	4245.28	6Ø28	3692.64
	Bottom	Left	5Ø12	565.2	6Ø16	1205.76	5Ø16	1004.8
		Middle	4Ø16	803.84	6Ø22	2279.64	8Ø20	2512
		Right	3Ø16	602.88	6Ø16	1205.76	5Ø16	1004.8

All selected beam reinforcements are given in the table above. All selected beam reinforcements meet the beam reinforcement areas taken from the Sap2000 Program. Selected beam reinforcements are suitable.

4.5 Design Phase II

In the design phase I, the internal force capacities of the load-bearing system, of which the pre-sizing under the impact of DD-2 level earthquake and designed, will be checked by using linear calculation methods under the effect of DD-4 earthquake, in order to achieve the uninterrupted use (IO-KK) performance target. The rules to be applied at this phase are presented below:

No additional eccentricity will be applied at this phase.

The damping ratio will be calculated as 2.5%.

R/I =1 and D =1 will be taken since the mode combination method will be used in the earthquake calculation. The minimum base shear force applied in phase I will not be considered at this phase.

At this phase, the average material strengths given in Table 4.17 will be taken in the calculation of internal force capacities.

Table 4.17 Average strength of materials.

Material	**Predicted Strengths**
Concrete	$f_{ce} = 1.3f_{ck}$
Reinforcement Steel	$f_{ye} = 1.2f_{yk}$
Structural Steel (S235)	$fye = 1.5fyk$
Structural Steel (S275)	$fye = 1.3fyk$
Structural Steel (S355)	$fye = 1.1fyk$
Structural Steel (S460)	$fye = 1.1fyk$

Since the calculations will be made with the design approach according to the strength based on the linear calculation, the effective section stiffness of the reinforced concrete elements will be determined from Table 4.18.

Table 4.18 Effective section rigidities to be applied in the Design Phase II.

Concrete Member	**Effective Stiffness Multiplier**	
Wall – Slab (In-Plane)	**Axial**	**Shear**
Shear Wall	0.75	1.00
Basement Shear wall	1.00	1.00
Slab	0.50	0.80
Wall – Slab (Out Of Plane)	**Flexure**	**Shear**
Shear Wall	1.00	1.00
Basement Shear wall	1.00	1.00
Slab	0.50	1.00
Frame member	**Flexure**	**shear**
Coupling beam	0.30	1.00
Frame beam	0.70	1.00
Frame column	0.90	1.00
Wall (equivalent strut)	0.80	1.00

4.5.1 Material Properties

C40/50 class concrete and B420C class reinforcing steel are used in all structural elements of the building**.**

- Characteristic strengths for C40/50;

  $f_{ck} = 40\ MPa$ (Characteristic compressive strength of concrete)

  $f_{ce} = 40 \times 1.3 = 52\ MPa$ (Average compressive strength of concrete)

  $E_c = 34\ GPa$ (Module of elasticity of concrete)

  $f_{ctk} = 0.35\sqrt{f_{ck}} = 0.35\sqrt{40} = 2.2\ MPa$ (Characteristic tensile strength of concrete)

- Design strengths for C40/50;

  The material coefficient of concrete is $\gamma_{mc} = 1.50$

  $f_{cd} = \frac{40}{1.5} = 26.67\ MPa$ (Design compressive strength of concrete)

  $f_{ctd} = \frac{2.2}{1.5} = 1.47\ Mpa$ (Design tensile strength of concrete)

- Strengths for B420C reinforcing steel;

  $f_{yk} = R_e = 420\ MPa$ (Yield strength)

$f_{ye} = 420 \times 1.2 = 504\ MPa$ (Average yield strength)

Tensile Strength / Yield Strength $1.15 \leq f_{su}/f_{yk} \leq 1.35$

$f_{su} = R_m = 525\ MPa$ (Tensile strength)

$E_s = 200\ GPa$ (Module of elasticity)

The material coefficient of reinforcing steel is $\gamma_{mc} = 1.15$

$f_{yd} = 420/1.15 = 365.22\ MPa$

4.5.2 Earthquake Parameters

The earthquake level used for the secound design phase of tall buildings is DD-4. The structure is located in Istanbul Province at latitude 41.131012° and longitude 29.017682°. The S_S and S_1 values for the horizontal elastic spectrum are taken from the Turkey Earthquake Hazard Maps.

$S_s = 0.170$

$S_1 = 0.055$

The local ground class is assumed to be ZB. The short-period local ground effect coefficient is 0.9 and the 1.0 second period local ground effect coefficient is 0.8 from Table 4.1 and Table 4.2. From here, the design spectral acceleration coefficients

$S_{DS} = S_s\, F_s = 0.170\ x\, 0.900 = 0.153$

$S_{D1} = S_1\, F_1 = 0.055\, x\, 0.800 = 0.044$

To plot the horizontal design spectrum, the corner periods will be calculated:

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.044}{0.153} = 0.058\ \text{s}$$

$$T_B = \frac{S_{D1}}{S_{DS}} = \frac{0.159}{0.610} = 0.288\ \text{s}$$

$T_L = 6\ s$

The horizontal elastic design spectrum is defined in Figure 4.8, depending on the natural vibration period:

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

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

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

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

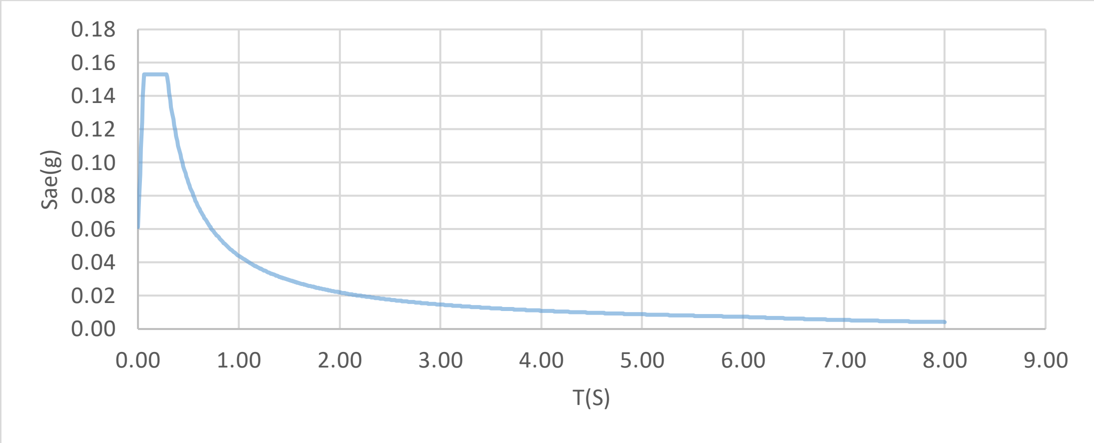


Figure 4.17 DD-4 earthquake level horizontal elastic design acceleration spectrum.

To create the vertical elastic design acceleration spectrum, SaeD (T) vertical elastic design spectral accelerations are determined depending on the period and the short period design spectral acceleration coefficient.

$$S_{aeD}(T) = \left(0.32 + 0.48\frac{T}{T_{AD}}\right)S_{DS} \qquad 0 \leq T \leq T_{AD}$$

$$S_{aeD}(T) = 0.8S_{DS} \qquad T_{AD} \leq T \leq T_{BD}$$

$$S_{aeD}(T) = 0.8S_{DS}\frac{T_{BD}}{T} \qquad T_{BD} \leq T \leq T_{LD}$$

Calculation ofT_{AD}, T_{BD} andT_{LD}, which are the corner periods of the vertical spectrum:

$$T_{AD} = \frac{T_A}{3} \qquad ; T_{BD} = \frac{T_B}{3} \qquad ; T_{LD} = \frac{T_L}{2}$$

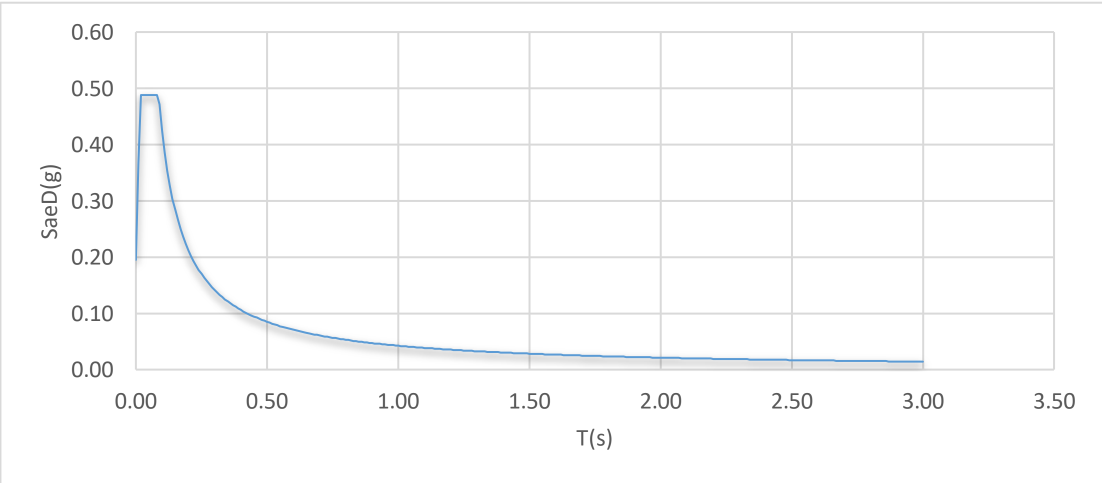


Figure 4.18 DD-4 earthquake level vertical elastic design acceleration spectrum.

4.5.3 Load and Load Combinations

- Loads to be used in the design:

  Fixed weight of reinforced concrete element: 25 KN/m^3

  Fixed pavement load: 2 KN/m^2

  Beam wall load: 6 KN/m

  Live load: 3.5 KN/m^2

  Snow load = 0.5 KN/m

- Load combinations to be used in the design:

  G: Dead Load.

  Q: Live Load.

  W: Wind Load

  E_X: Earthquake Load X Direction.

  E_Y: Earthquake Load Y Direction.

  EZ: Earthquake Load Z Direction.

1.4G+1.6Q

G+1.3Q+1.3WX

G+1.3Q-1.3WX

G+1.3Q+1.3WY

G+1.3Q-1.3WY

0.9G+1.3WX

0.9G-1.3WX

0.9G+1.3WY

0.9G-1.3WY

G+1.2Q

G+Q+EX

G+Q-EX

G+Q+EY

G+Q-EY

0.9G+EX

0.9G-EX

0.9G+EY

0.9G-EY

: Vertical Earthquake Effect Ez ⇒ (2/3)*Sds*G

EX+0.3EY

EX-0.3EY

-EX-0.3EY

-EX+0.3EY

EY+0.3EX

EY-0.3EX

-EY-0.3EX

-EY+0.3EX

G+Q+0.2S+EX+0.3EY+0.3EZ

G+Q+0.2S+EX-0.3EY+0.3EZ

G+Q+0.2S-EX-0.3EY+0.3EZ

G+Q+0.2S-EX+0.3EY+0.3EZ

G+Q+0.2S-EY-0.3EX+0.3EZ

G+Q+0.2S-EY+0.3EX+0.3EZ

G+Q+0.2S+E_Y-0.3E_X+0.3EZ

G+Q+0.2S+EY+0.3EX+0.3EZ

0.9G+EX+0.3EY-0.3EZ

0.9G+EX-0.3EY-0.3EZ

0.9G-EX-0.3EY-0.3EZ

0.9G-EX+0.3EY-0.3EZ

0.9G+EY+0.3EX-0.3EZ

0.9G+EY-0.3EX-0.3EZ

0.9G-EY+0.3EX-0.3EZ

0.9G-EY-0.3EX-0.3EZ

4.5.4 Capacity Calculation of Carrier System Elements

Capacity calculations of structural system elements are calculated using the average material strengths in the Response-2000 (2019) program. As an example, moment and shear force capacity of 50/80 size beam is drawn in Figure 4.19 and Figure 4.20. The capacity values of all other elements are shown in Table 4.19.

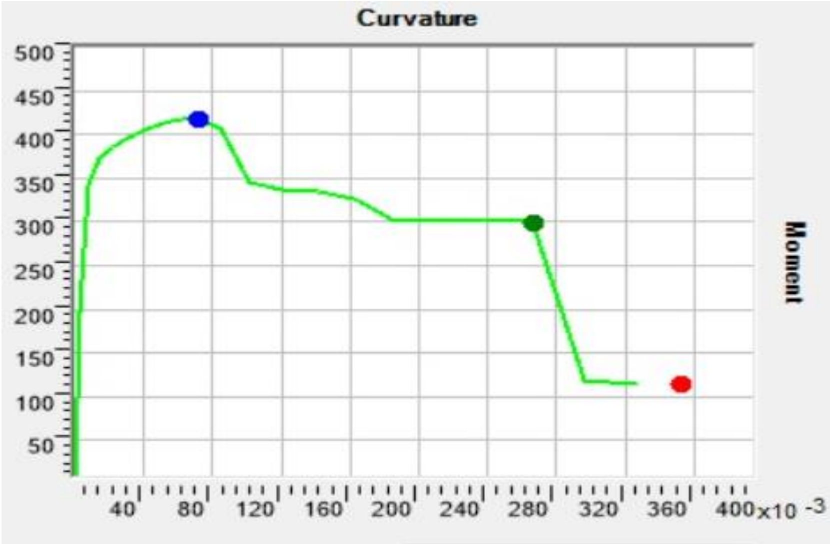


Figure 4.19 50/80 beam moment capacity curve.

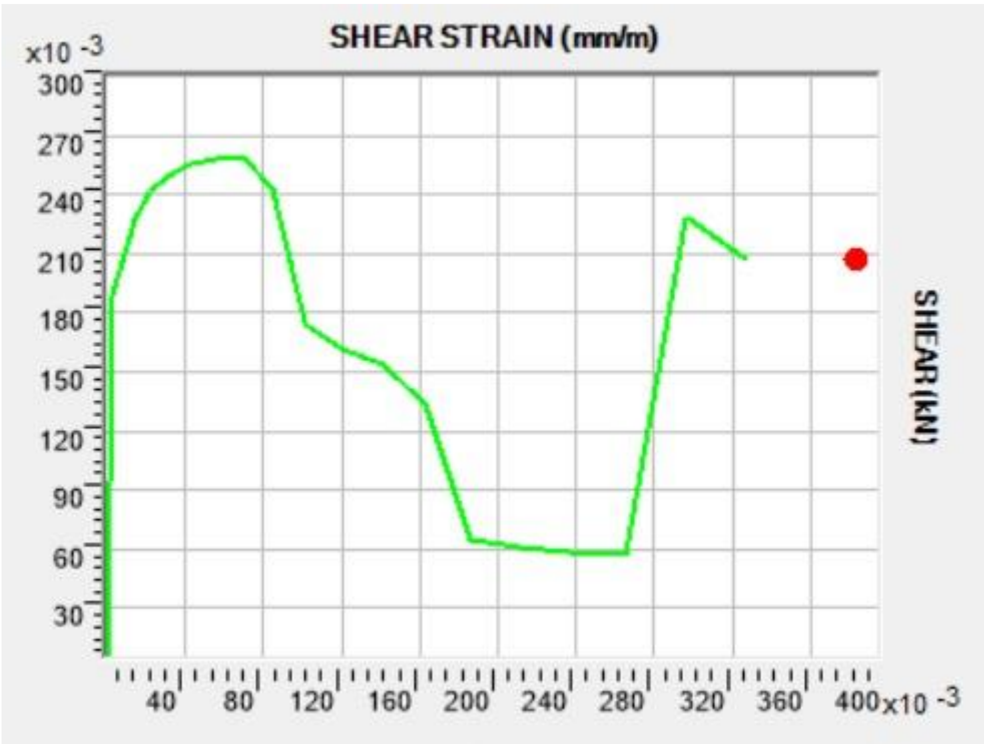


Figure 4.20 45/65 beam shear force capacity curve

Table 4.19 Beam capacity values.

Story	Element No.	Dimensions cm*cm	Vr (kN)	Mr (kN.m)
All stories	B3	B (70*50)	888.1847	1440.375
	B4	B (70*50)	793.5145	1018.375
	B5	B (70*50)	932.3441	1018.375
	B6	B (70*50)	1066.597	1357.427
	B7	B (70*50)	1196.273	1357.427
	B8	B (70*50)	1321.372	1482.629
	B9	B (70*50)	1441.895	1482.629
	B10	B (70*50)	1557.84	1143.576
	B11	B (70*50)	1669.209	1143.576
	B12	B (70*50)	1776.001	1357.427
	B13	B (70*50)	1878.217	1357.427
	B14	B (70*50)	1975.855	1143.576
	B15	B (70*50)	2068.917	1143.576
	B16	B (70*50)	2157.401	1628.698
	B17	B (70*50)	2241.31	1628.698
	B18	B (70*50)	2320.641	872.3059
	B19	B (70*50)	2395.395	872.3059
	B20	B (70*50)	2465.573	1211.359
	B21	B (70*50)	2531.174	1211.359
	B22	B (70*50)	2592.198	1628.698
	B23	B (70*50)	2648.645	1628.698
	B24	B (70*50)	2700.515	1289.645
	B25	B (70*50)	2747.809	1289.645
	B26	B (70*50)	2790.526	2190.292
	B27	B (70*55)	2828.666	2190.292
	B28	B (70*55)	2862.229	1433.9
	B29	B (70*55)	2891.215	1433.9
	B30	B (70*55)	2915.625	1772.952
	B42	B (70*50)	2935.458	1772.952
	B43	B (70*50)	2951.851	2190.292
	B49	B (45*65)	2960.616	2190.292
	B50	B (45*65)	793.5145	1851.239
	B51	B (45*65)	932.3441	1851.239
	B52	B (45*65)	1066.597	1772.952
	B53	B (45*65)	1196.273	1772.952
	B54	B (45*65)	1321.372	1851.239
	B55	B (45*65)	1441.895	1851.239
	B56	B (45*65)	1557.84	1579.968
	B57	B (45*65)	1669.209	1579.968
	B58	B (45*65)	1776.001	1919.021

The D/C ratios of the columns and walls are calculated in the ETABS (2019) program, which is not manually calculated since separate moments would occur for each axial force value and the capacity would change. Shear capacities of walls and columns are given in Table 4.20 and Table 4.21

Table 4.20 Columns capacity values

Story	Element no.	Dimensions cm*cm	Vr (kN)	Mr (kN.m)
All Story	C1	C95/95	480.8296	1440.375
	C1	C95/95	480.8296	1440.375
	C3	C95/95	631.6439	728.9481
	C4	C95/95	631.6439	728.9481
	C5	C85/85	631.6439	1440.375
	C6	C85/85	631.6439	1440.375
	C7	C85/85	480.8296	1023.036
	C8	C85/85	480.8296	1023.036
	C9	C85/85	881.292	1146.287
	C10	C85/85	881.292	1146.287
	C11	C85/85	730.4777	1023.036
	C12	C85/85	730.4777	1023.036

Table 4.21 Shear wall capacity values

Story No.	Element No.	Dimensions cm*cm	Vr (kN)	Mr (kN.m)
30	P1	50 x 130	2960.62	14826.29
30	P2	50 x 600	2960.62	14826.29
30	P3	50 x 860	2960.616	10183.75
29	P1	50 x 130	2960.616	10183.75
29	P2	50 x 600	2960.62	13574.27
29	P3	50 x 860	2960.62	13574.27
28	P1	50 x 130	2960.616	14826.29
28	P2	50 x 600	2960.616	14826.29
28	P3	50 x 860	2960.62	11435.76
27	P1	50 x 130	2960.62	11435.76
27	P2	50 x 600	2960.616	13574.27
27	P3	50 x 860	2960.616	13574.27
26	P1	50 x 130	2960.616	11435.76
26	P2	50 x 600	2960.616	11435.76
26	P3	50 x 860	888.185	16286.98
25	P1	50 x 130	888.185	16286.98
25	P2	50 x 600	888.1847	8723.059
25	P3	50 x 860	888.1847	8723.059
24	P1	50 x 130	888.185	12113.59
24	P2	50 x 600	888.185	12113.59
24	P3	50 x 860	888.1847	16286.98
23	P1	50 x 130	888.1847	16286.98
23	P2	50 x 600	888.185	12896.45
23	P3	50 x 860	888.185	12896.45
22	P1	50 x 130	888.185	21902.92
22	P2	50 x 600	888.185	21902.92
22	P3	50 x 860	888.1847	14339
21	P1	50 x 130	888.1847	14339
21	P2	50 x 600	888.185	17729.52
21	P3	50 x 860	888.185	17729.52
20	P1	50 x 130	888.1847	21902.92
20	P2	50 x 600	888.1847	21902.92
20	P3	50 x 860	888.185	18512.39
19	P1	50 x 130	888.185	18512.39
19	P2	50 x 600	888.1847	17729.52
19	P3	50 x 860	888.1847	17729.52
18	P1	50 x 130	888.1847	18512.39
18	P2	50 x 600	888.1847	18512.39
18	P3	50 x 860	2960.62	15799.68
17	P1	50 x 130	2960.62	15799.68

Table 4.21 (continuing-1)

Story No.	Element No.	Dimensions cm*cm	Vr (kN)	Mr (kN.m)
17	P2	50 x 600	2960.62	19190.21
17	P3	50 x 860	2960.62	19190.21
16	P1	50 x 130	2960.62	17051.7
16	P2	50 x 600	2960.62	17051.7
16	P3	50 x 860	2960.62	20442.23
15	P1	50 x 130	2960.62	20442.23
15	P2	50 x 600	2960.616	15799.68
15	P3	50 x 860	2960.616	15799.68
14	P1	50 x 130	2960.616	19190.21
14	P2	50 x 600	2960.616	19190.21
14	P3	50 x 860	2960.616	20442.23
13	P1	50 x 130	2960.616	20442.23
13	P2	50 x 600	2010.74	13633.4
13	P3	50 x 860	2010.74	13633.4
12	P1	50 x 130	2010.741	10242.87
12	P2	50 x 600	2010.741	10242.87
12	P3	50 x 860	2010.741	13633.4
11	P1	50 x 130	2010.741	13633.4
11	P2	50 x 600	2010.74	11376.64
11	P3	50 x 860	2010.74	11376.64
10	P1	50 x 130	2010.74	14767.17
10	P2	50 x 600	2010.74	14767.17
10	P3	50 x 860	2010.74	9487.029
9	P1	50 x 130	2010.74	9487.029
9	P2	50 x 600	2010.741	14767.17
9	P3	50 x 860	2010.741	14767.17
8	P1	50 x 130	603.2224	16089.89
8	P2	50 x 600	603.2224	16089.89
8	P3	50 x 860	603.222	12699.36
7	P1	50 x 130	603.222	12699.36
7	P2	50 x 600	603.222	16089.89
7	P3	50 x 860	603.222	16089.89
6	P1	50 x 130	603.2224	8920.146
6	P2	50 x 600	603.2224	8920.146
6	P3	50 x 860	603.2224	12310.67
5	P1	50 x 130	603.2224	12310.67
5	P2	50 x 600	603.2224	15858.81
5	P3	50 x 860	603.2224	15858.81
4	P1	50 x 130	603.222	16992.57
4	P2	50 x 600	603.222	16992.57

Table 4.21 (continuing-2)

Story No.	Element No.	Dimensions cm*cm	Vr (kN)	Mr (kN.m)
4	P3	50 x 860	603.222	20383.1
3	P1	50 x 130	603.222	20383.1
3	P2	50 x 600	2010.741	19249.34
3	P3	50 x 860	2010.741	19249.34
2	P1	50 x 130	2010.741	18315.3
2	P2	50 x 600	2010.741	18315.3
2	P3	50 x 860	2010.741	21705.83
1	P1	50 x 130	2010.741	21705.83
1	P2	50 x 600	603.222	18315.3
1	P3	50 x 860	603.222	18315.3

4.5.5 Demand/Capacity Ratios of Structural System Elements

According to TBDY, the D/C ratio of the internal forces of the elements with ductile behavior should not exceed 1.50 in the case that phase- II analyses are performed using linear calculation methods. The D/C ratio of internal forces that do not have ductile behavior should not exceed 0.70.

Internal forces with ductile behavior: Bending in two directions and yielding under the effect of axial force on the shear walls. Flow in beams under bending effect. Bending in two directions and yielding under the influence of axial force in columns. Shear in cross-reinforced tie beams. Internal forces that do not have ductile behavior: Shear forces in shear walls, columns, and beams.

Due to a large number of structural system elements, the results are given from floors (1-11-20-30) where the element cross-sections change. The naming of the carrier system elements to be used in the D/C diagrams is given in Figure 4.21.

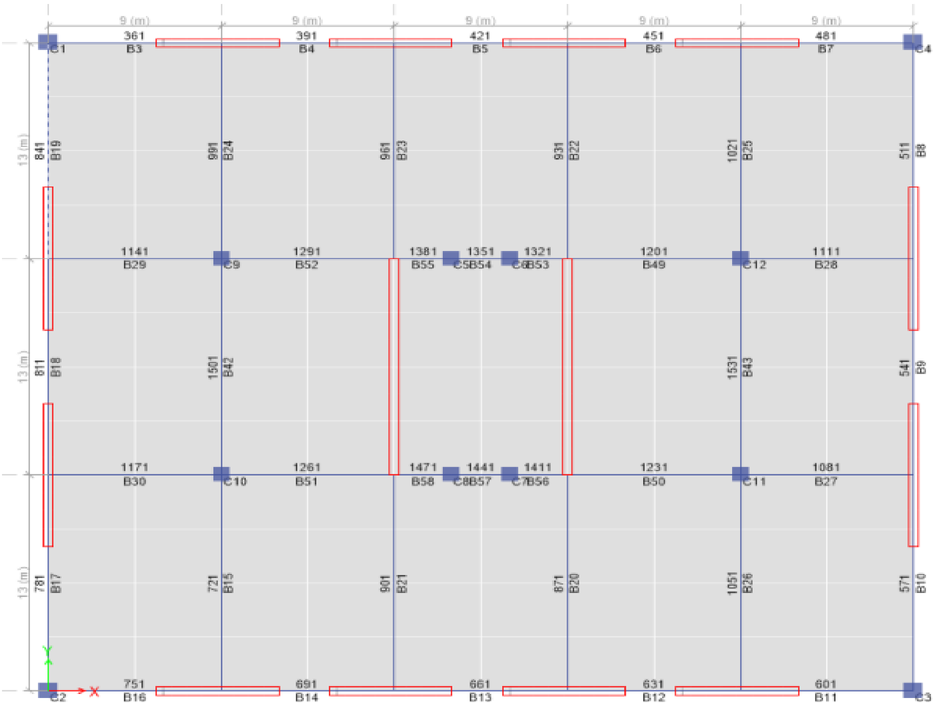


Figure 4.21 Carrier system element nomenclature.

Shear Force D/C Ratios of Beams:

The shear force values of the beams under the effect of DD-4 earthquake ground motion are proportional to the calculated capacity values taken from the Etabs program. The first-floor results are given in Figure 4.22, the 11th-floor results are given in Figure 4.23, the 20th-floor results are given in Figure 4.24 and the 30th-floor results are given in Figure 4.25. When the shear force values of the beams are examined, it is seen that the greatest effects occur in the beams connecting the core system to the frame columns. The ratios found are calculated as a maximum of 0.35 for first-floor beams, maximum of 0.28 for11thstory beams, a maximum of 0.092 fo20th-floor beams and 0.039 a maximum for 30th-floor beams. In TBDY, the beam shear force D/C ratio of 0.70 is provided on all floors.

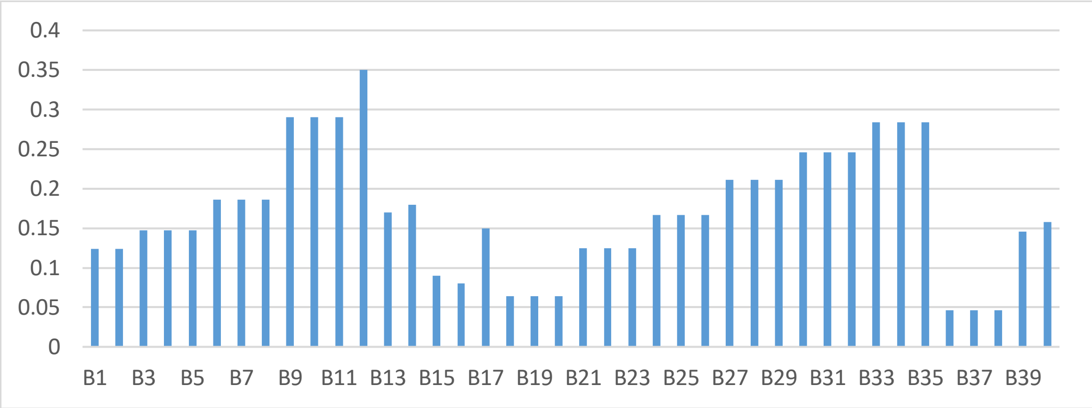


Figure 4.22 Shear force D/C ratios of 1st floor beams.

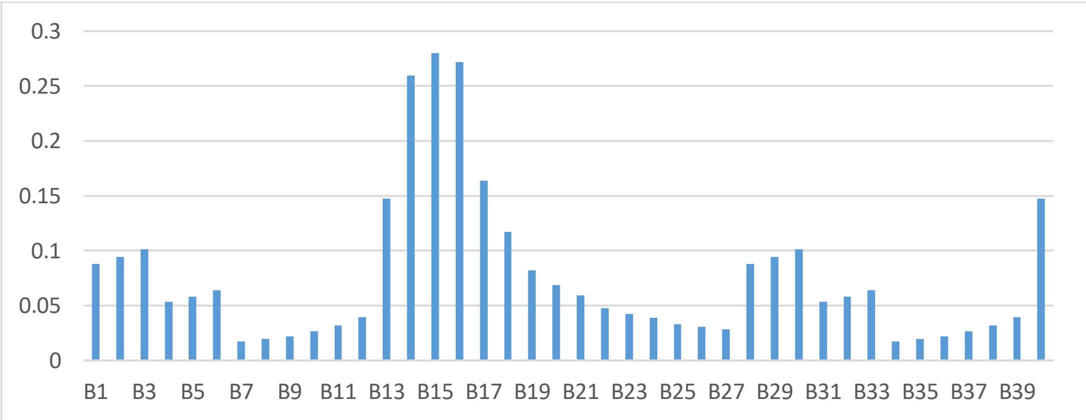


Figure 4.23 Shear force D/C ratios of 11th floor beams.

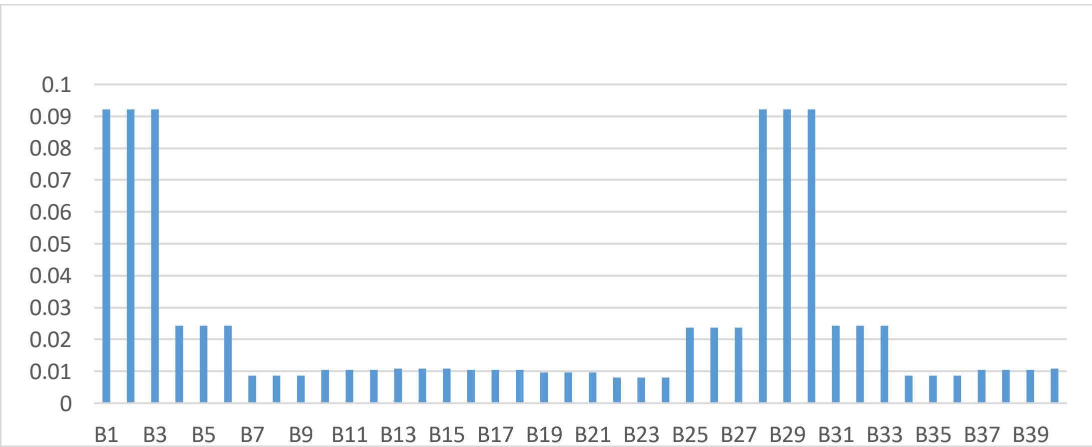


Figure 4.24 Shear force D/C ratios of 20th floor beams.

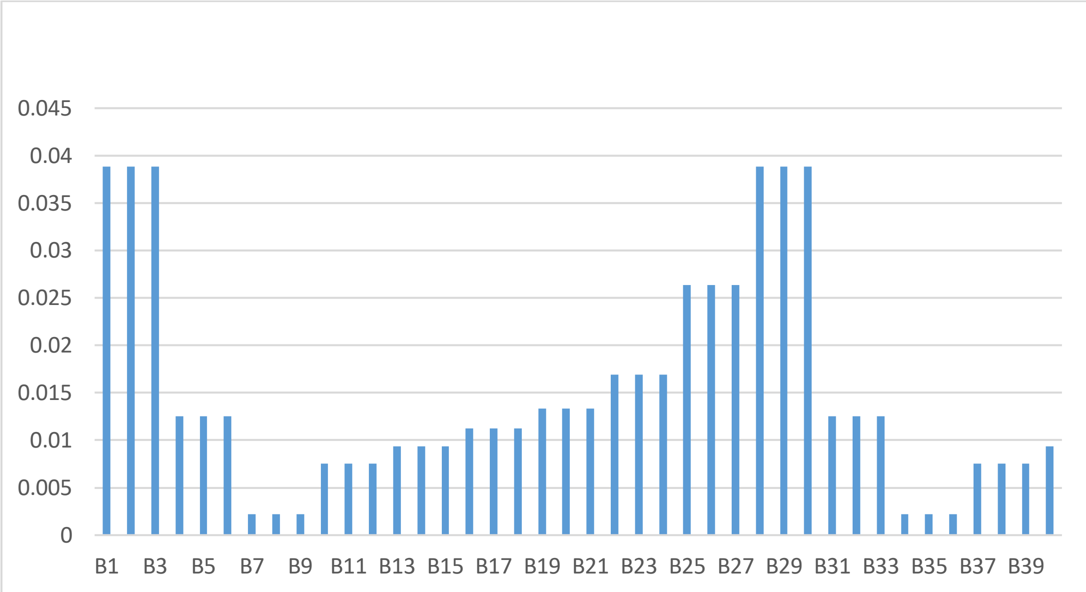


Figure 4.25 Shear force D/C ratios of 30th floor beams.

Moment D/C Ratios of Beams:

The moment values of the beams under the effect of DD-4 earthquake ground motion are proportional to the calculated capacity values taken from the Etabs program. 1st floor results are given in Figure 4.26, 11th floor results are given in Figure 4.27, 20th floor results are given in Figure 4.28 and 30th floor results are given in Figure 4.29. When the moment values of the beams are examined, it is seen that the greatest effects occur in the beams connecting the core system to the frame columns. The ratios found are calculated as maximum 0.030 for 1st floor beams, 0.15 maximum for 11th floor beams, 0.06 maximum for 20th floor beams and 0.63 maximum for 30th floor beams. In TBDY, the beam moment D/C ratio of 1.5 is provided on all floors.

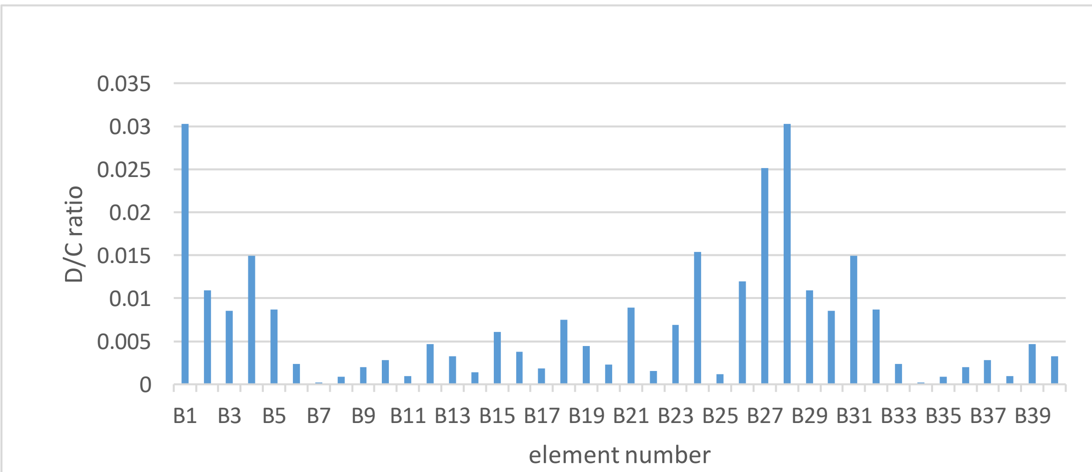


Figure 4.26 Moment D/C ratios of 1ststory beams.

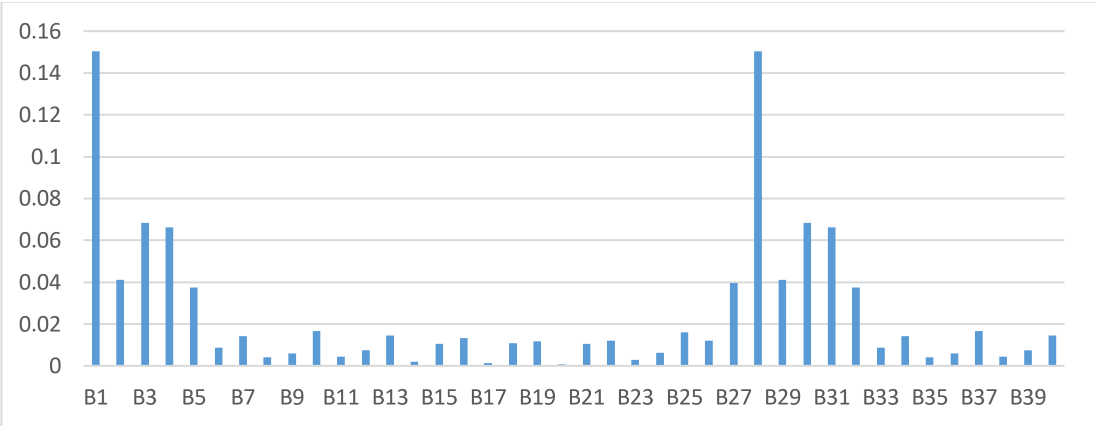


Figure 4.27 Moment D/C ratios of 11th story beams.

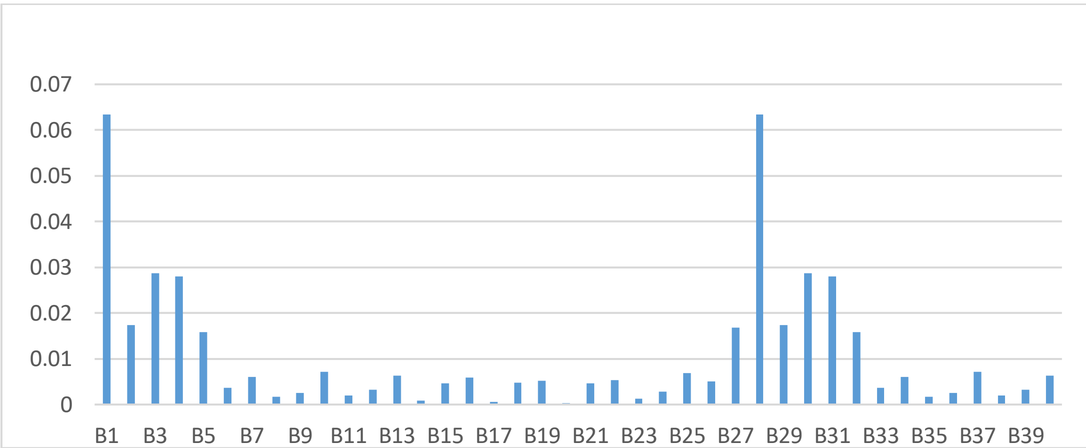


Figure 4.28 Moment D/C ratios of 20th story beams.

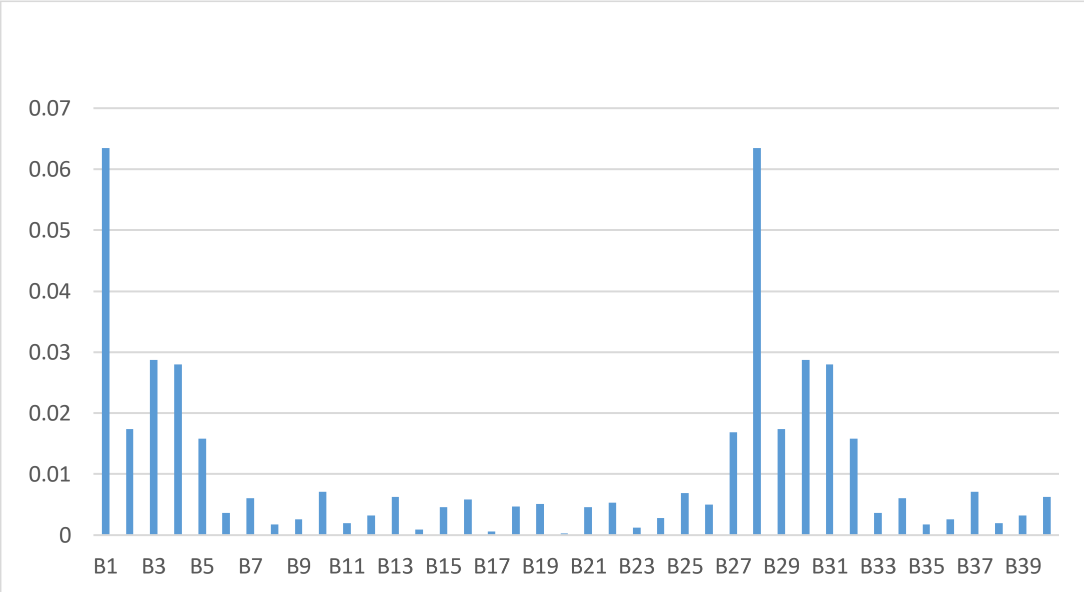


Figure 4.29 Moment D/C ratios of 30th story beams

D/C Ratios of Shear walls:

Shear force values of the shear walls under the effect of DD-4 earthquake ground motion are proportional to the calculated capacity values taken from the Etabs program. Shear force D/C ratios are given in Figure 4.30. The graph shows that the largest value is 0.152. The D/C ratio here is calculated as 01.52. In TBDY, it is seen that the shear capacity ratio of the shear walls is 0.7. Although very strong wall sections are defined in our structure, where all the earthquake loads are met by the walls, it is seen that it approaches the limit value. Calculations for the bending of the shear walls in two directions and yielding under the effect of axial force are completely made in the Etabs program. Separate moment capacity values are calculated for each changing axial force condition. D/C ratios are given in Figure 4.31. The graph shows that the largest value is 0.76. Since the limit value for the shear walls is defined as 1.50 in TBDY, it has been concluded that the Shear wall capacities are sufficient.

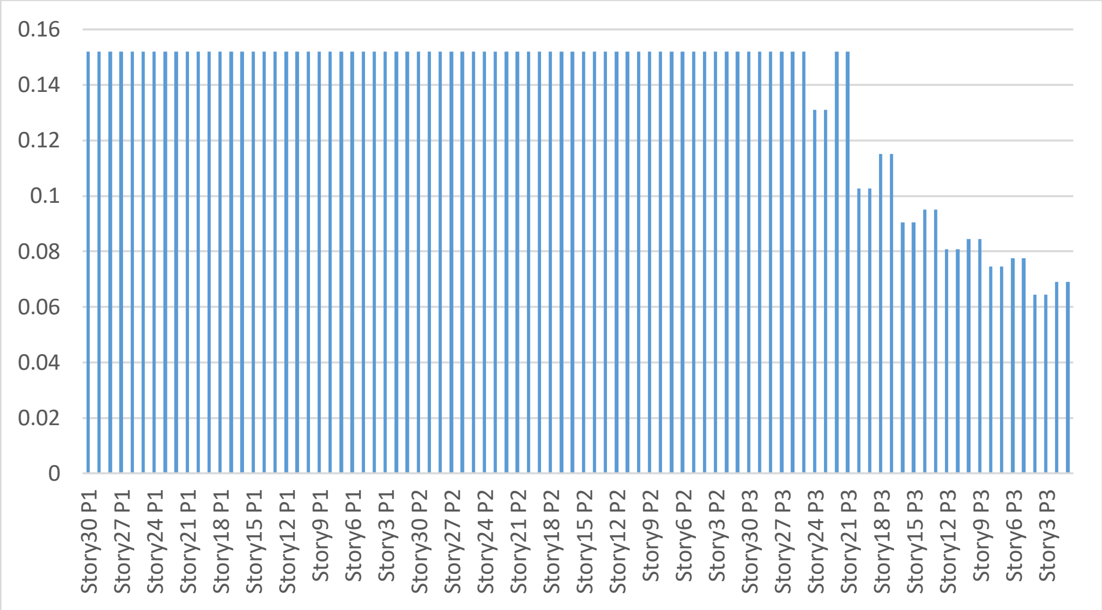


Figure 4.30 Shear walls shear force D/C ratios.

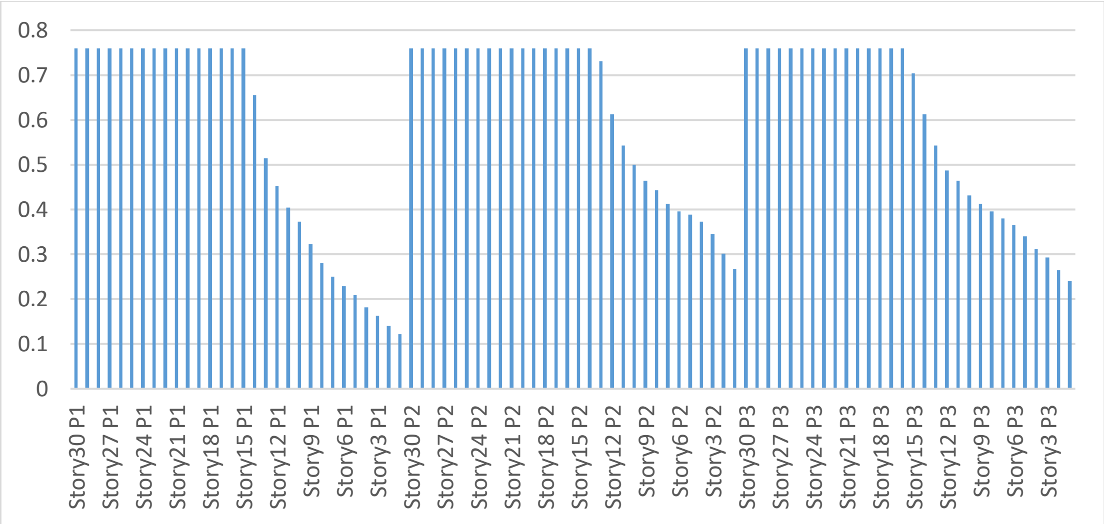


Figure 4.31 Shear walls P-M-M D/C ratios.

D/C Ratios of Columns:

The shear force values of the columns under the effect of DD-4 earthquake ground motion are proportional to the calculated capacity values taken from the Etabs program. The 1st floor results are given in Figure 4.32, the 11th floor results are given in Figure 4.33, the 20th floor results are given in Figure 4.34 and the 30th floor results are given in Figure 4.35. The largest D/C ratio is calculated as 0.69. Shear D/C ratio in TBDY is defined as 0.70 for columns. Calculations for the bending of the columns in two directions and flowing under the effect of axial force are completely made in the Etabs program. Separate moment capacity values are calculated for each changing axial force condition. 1st floor results of D/C ratios are given in Figure 4.36, 11th floor results in Figure 4.37, 20th floor results in Figure 4.38 and 30th floor results in Figure 4.39. The largest of the values is calculated as 0.70 on the 20th floor, where the axial force is maximum. Since the boundary condition given in TBDY is 1.50, column capacities are sufficient.

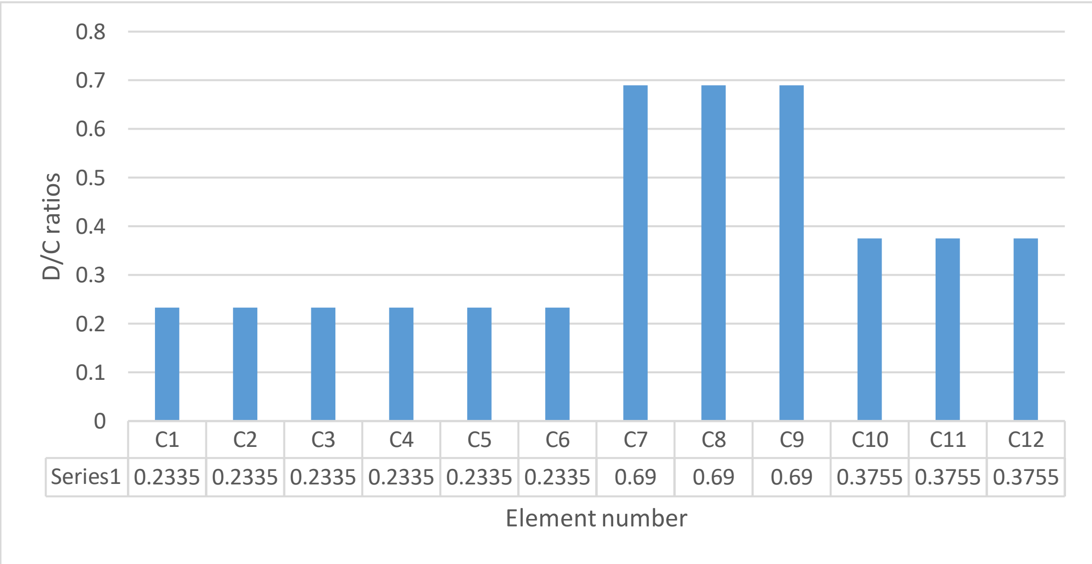


Figure 4.32 1st floor column shear force D/C ratios.

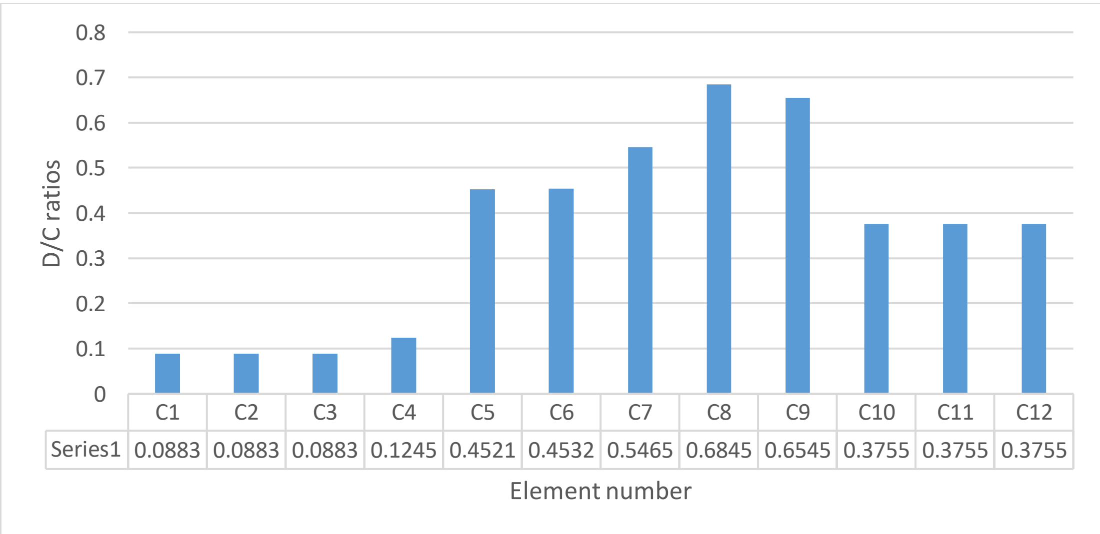


Figure 4.33 11th floor column shear force D/C ratios

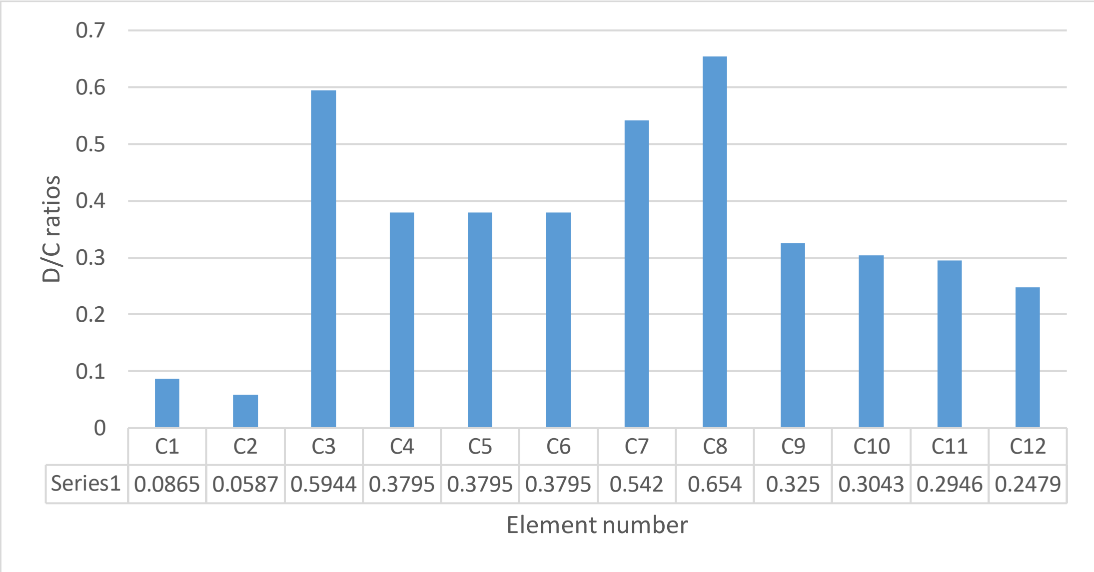


Figure 4.34 20thstory column shear force D/C ratios.

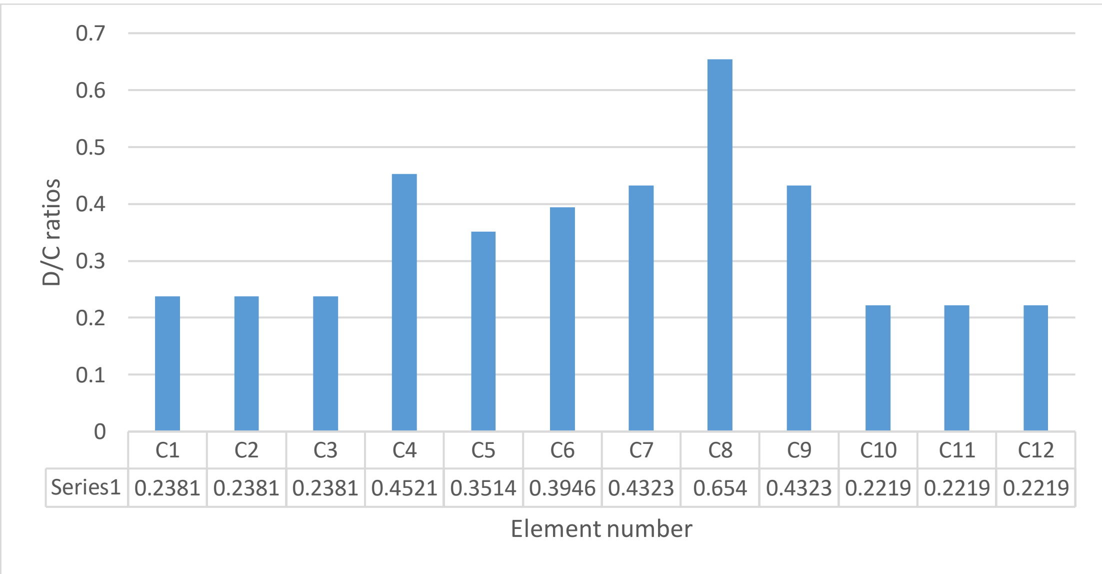


Figure 4.35 30th floor column shear force D/C ratios.

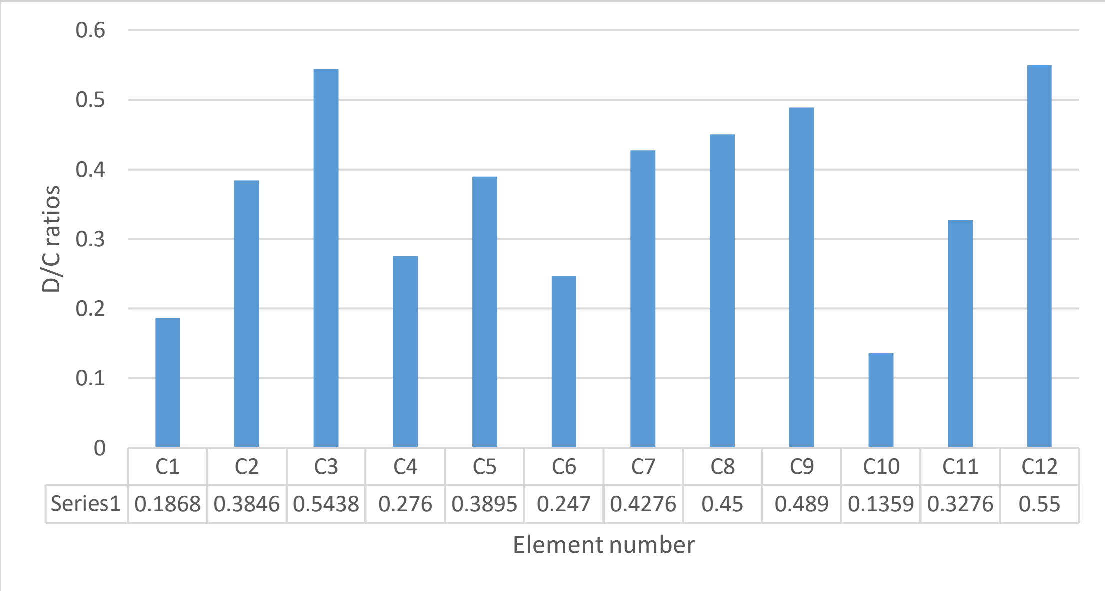


Figure 4.36 1st floor column P-M-M D/C ratios

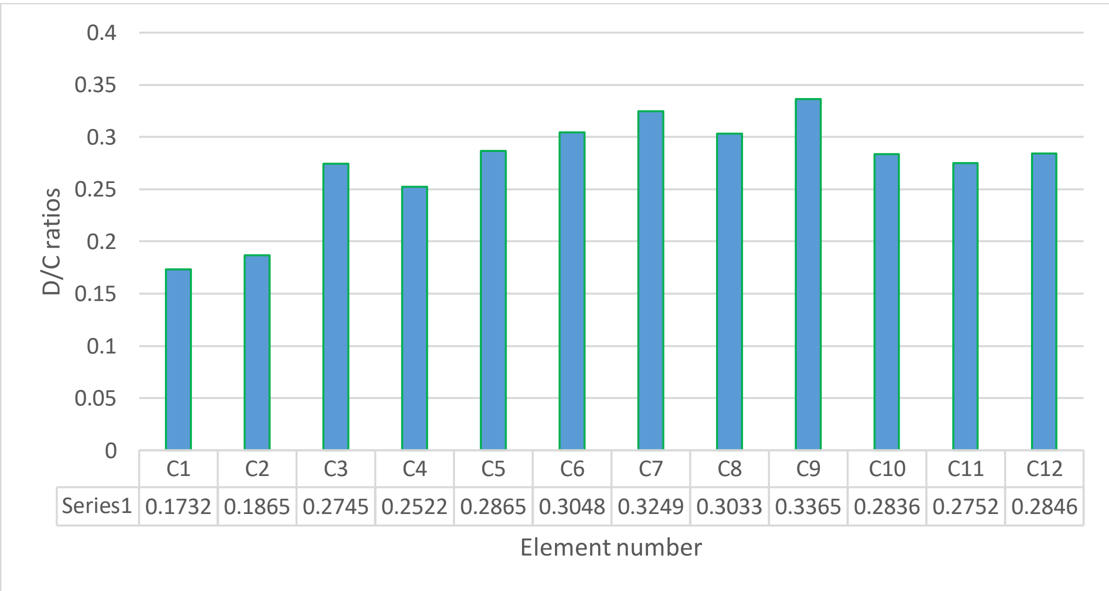


Figure 4.37 11th floor column P-M-M D/C ratios.

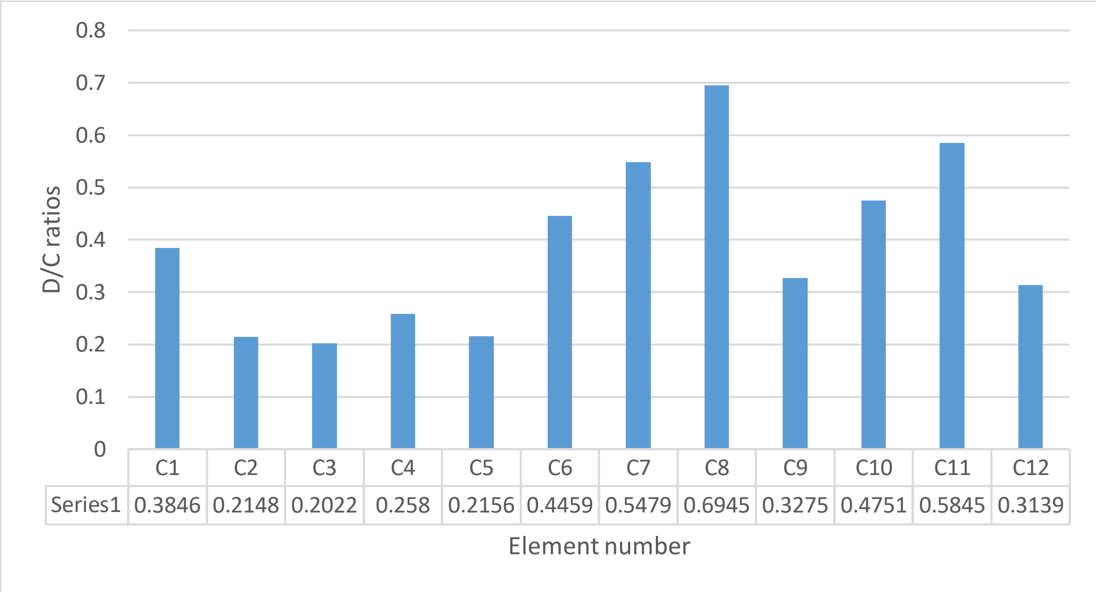


Figure 4.38 20th floor column P-M-M D/C ratios.

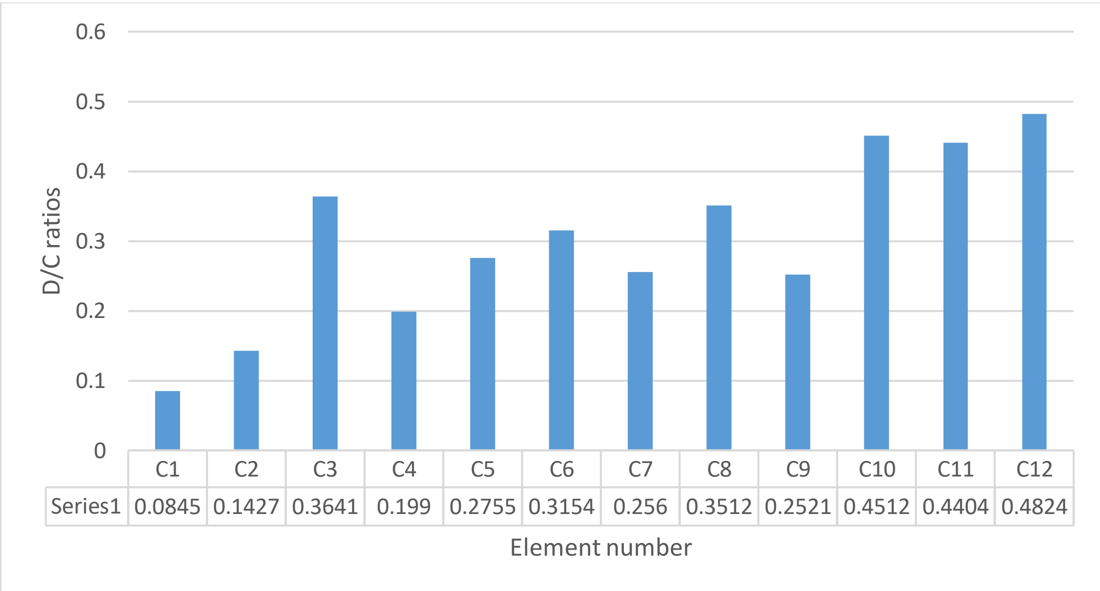


Figure 4.39 30th floor column P-M-M D/C ratios.

4.6 Comparison of TBDY 2018 and DBYBHY 2007 Analysis

Assuming the structural system and material properties are the same, the structure will be analyzed by accepting it as Z2 soil class and Second Degree earthquake zone within the scope of DBYBHY 2007 regulation (Pakoglu, 2009).

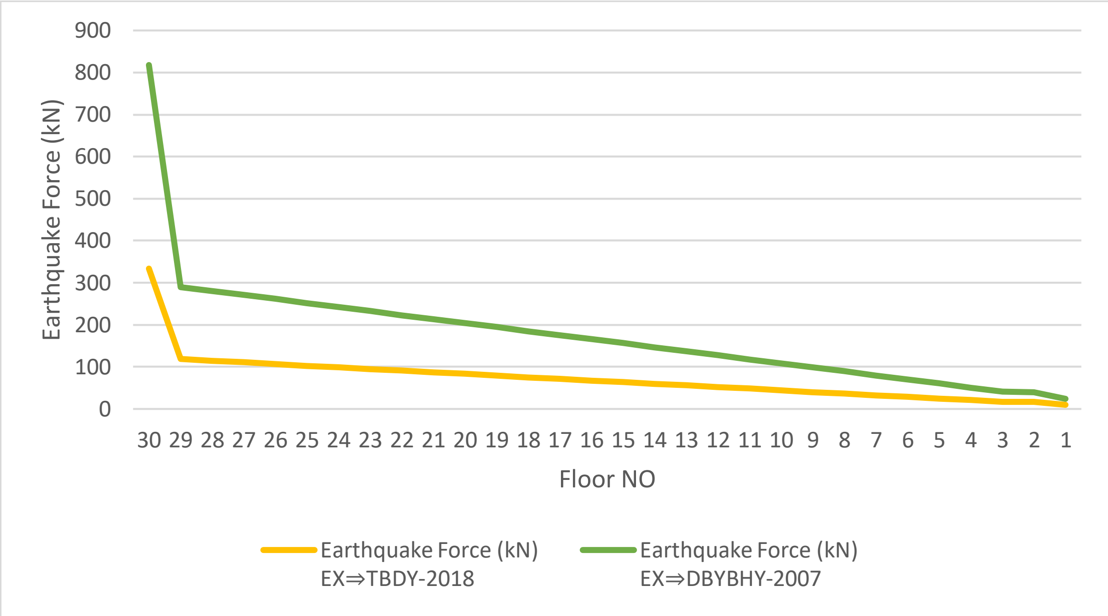


Figure 4.40 Comparison of 2007 and 2018 X-Directional Seismic Forces.

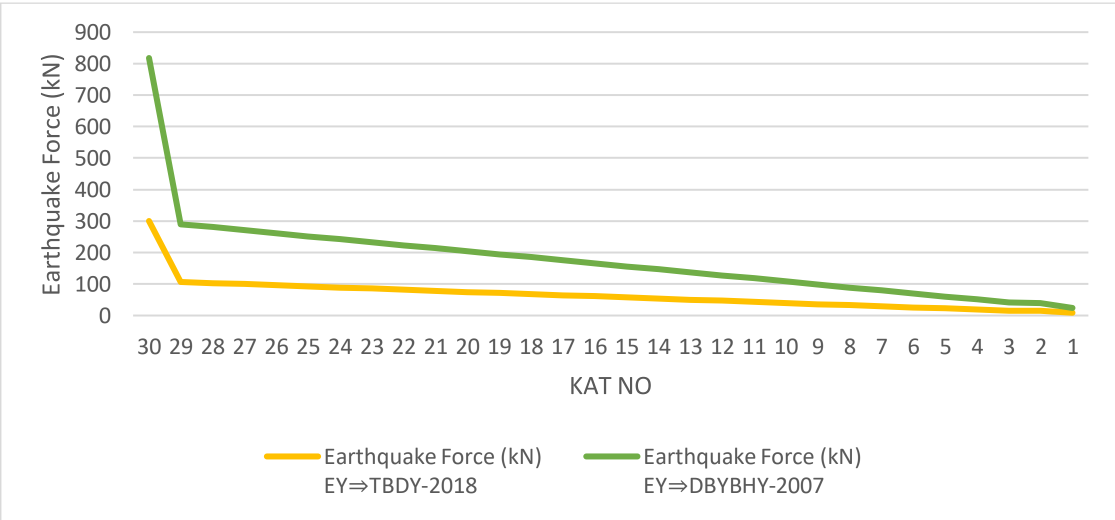


Figure 4.41 Comparison of 2007 and 2018 Y Direction Seismic Forces.

4.6.1 DBYBHY 2007 Earthquake Parameters

In order to determine the earthquake loads of the building, the spectral acceleration coefficient A(T) should be determined. As it is known, an earthquake has an acceleration. Due to the ground motion, an acceleration occurs in the building. However, the acceleration in the building and the value of the earthquake acceleration are not the same. The earthquake force that will occur in the structure determines the acceleration in the structure. The ratio of the acceleration in the structure to the acceleration of gravity is called the spectral acceleration coefficient. A(T) As given in Eq. (4.12), the effective ground acceleration coefficient A_0, is equal to the product of the building importance factor I and the spectrum coefficient S(T).

$$A(T) = A_0 \times I \times S(T)$$

Effective ground acceleration coefficient (A_0):

The effective ground acceleration coefficient (A_0) should be selected from Table 4.22 according to the earthquake zone in the Turkey Earthquake Zones Map of the region where the building will be built.

Table 4.22 Effective ground acceleration coefficients

Earthquake Zone	A_0
1	0.40
2	0.30
3	0.20
4	0.10

Spectrum coefficient S (T):

Spectrum coefficient S (T) will be calculated by Eq. (4.13) based on local ground conditions and building natural period.

$$S(T) = 1 + 1.5(T/T_A) \qquad 0 \le T \le T_A$$

$$S(T) = 2.5 \qquad T_A \le T \le T_B$$

$$S(T) = 2.5 \times (T_B/T)^{0.8} \qquad T_B \le T$$

Spectrum characteristic periods T_A and T_B used in Eq. (4.13) are taken from Table 4.23 depending on local soil classes.

Table 4.23 Spectrum characteristic periods.

Local Ground Grade	T_A (s)	T_B (s)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Earthquake load reduction coefficient:

The seismic load reduction coefficient is used to consider the intrinsic inelastic behavior of the structural system. R_a, the earthquake load reduction coefficient will be determined by Eq. (4.14) based on the natural vibration period structural system response coefficient R.

$$R_a(T) = 1.5 + (R - 1.5) \times (T/T_A) \qquad 0 \leq T \leq T_A$$

$$R_a(T) = R \qquad T_A \leq T$$

4.6.2 Comparison of Horizontal Elastic Spectra

Horizontal elastic design spectra have been calculated and plotted in Figure 4.42 for the earthquake at local soil class Z2 for the 2007 regulation and for the DD-2 level earthquake for the 2018 code. When the spectra are examined, the dominant natural vibration period of our structure is 2.749 sec. The horizontal elastic design acceleration value corresponding to the value is 0.0266 g. for the 2007 regulation; in the 2018 regulation, this value is calculated as 0.015 g. In other words, in the new regulation, the force that the structure will be exposed to has been reduced by 1.8 times (Koçer et al., 2018).

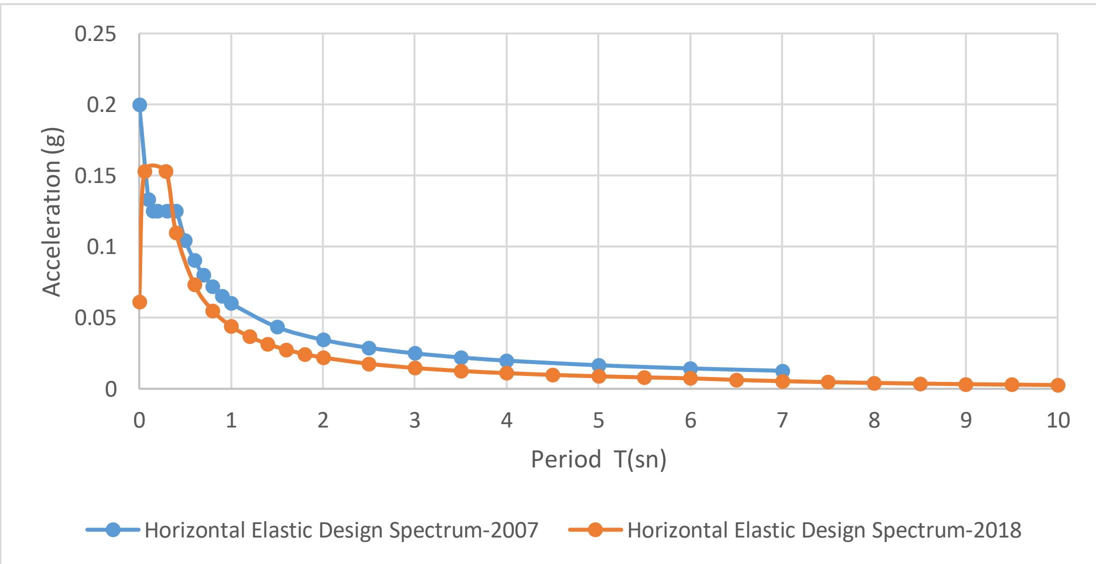


Figure 4.42 Horizontal elastic spectrum comparison.

The reduced horizontal elastic design acceleration spectrum of the same earthquake levels is obtained in Figure 4.43. Here, the dominant natural vibration period of the structure is 2.749 s. The reduced horizontal elastic design acceleration value corresponding to the value is 0.004 g for the 2007 regulation; in the 2018 regulation, this value is calculated as 0.003 g. In both examinations, it is obvious that the loads to be affected on the structure will decrease in the 2018 regulation.

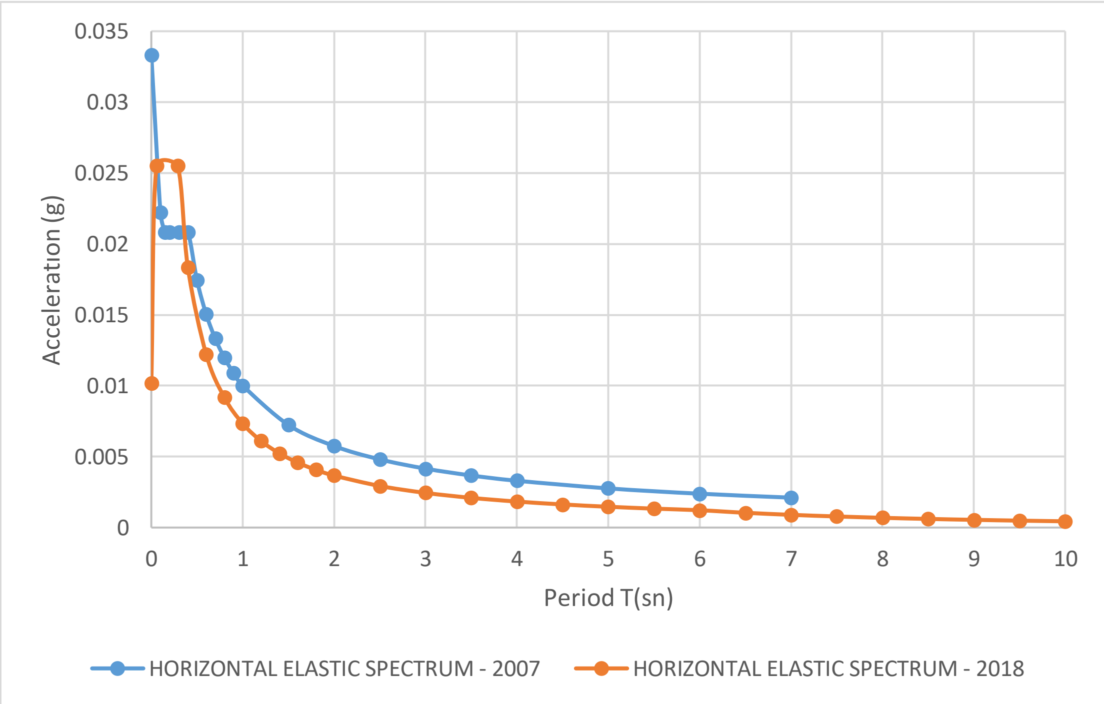


Figure 4.43 Reduced horizontal elastic spectrum comparison

4.6.3 Comparison of Relative Story Offsets

The effective relative story drifts of the building in both earthquake directions are calculated and presented in Figure 4.44 and Figure 4.45. The maximum effective relative story drift in the X direction occurred at the 10st story. While this value is 4.2 mm according to the 2007 regulation, it is calculated as 4.7 mm on the same story in the 2018 regulation. When looking at the Y direction, the maximum values occur on the 20th story, and while these values are 5.1 mm for the 2007 regulations, they are 7.5 mm for the 2018 regulations. It is understood that the horizontal loads that the structure will be exposed to will be 12% higher in the X direction and 45% in the Y direction in the 2018 regulation. When the boundary condition is examined, the effective relative story drift limit in the 2007 regulation is fixed as 0.02 in both directions. In the 2018 regulation, this limit value is determined by calculating the λ

coefficient. For our example structure, the previously calculated X-direction λ value is 0.415, and the Y-direction λ value is 0.41. Effective relative story drift limit values calculated with these values in Eq. 4.15 are 0.414for the X direction and 4.42 for the Y direction.

$$\lambda \frac{\delta_{i,max}^{X}}{h_i} \leq 0.008\kappa \tag{4.15}$$

While the limit values for our example structure do not differ much in the two regulations, it is allowed to double this limit value for the buildings with flexible joints in the 2018 regulation. Since the λ coefficient is the ratio of the elastic spectral accelerations of DD2 and DD3, it has been determined that this ratio will not change much under different ground conditions.

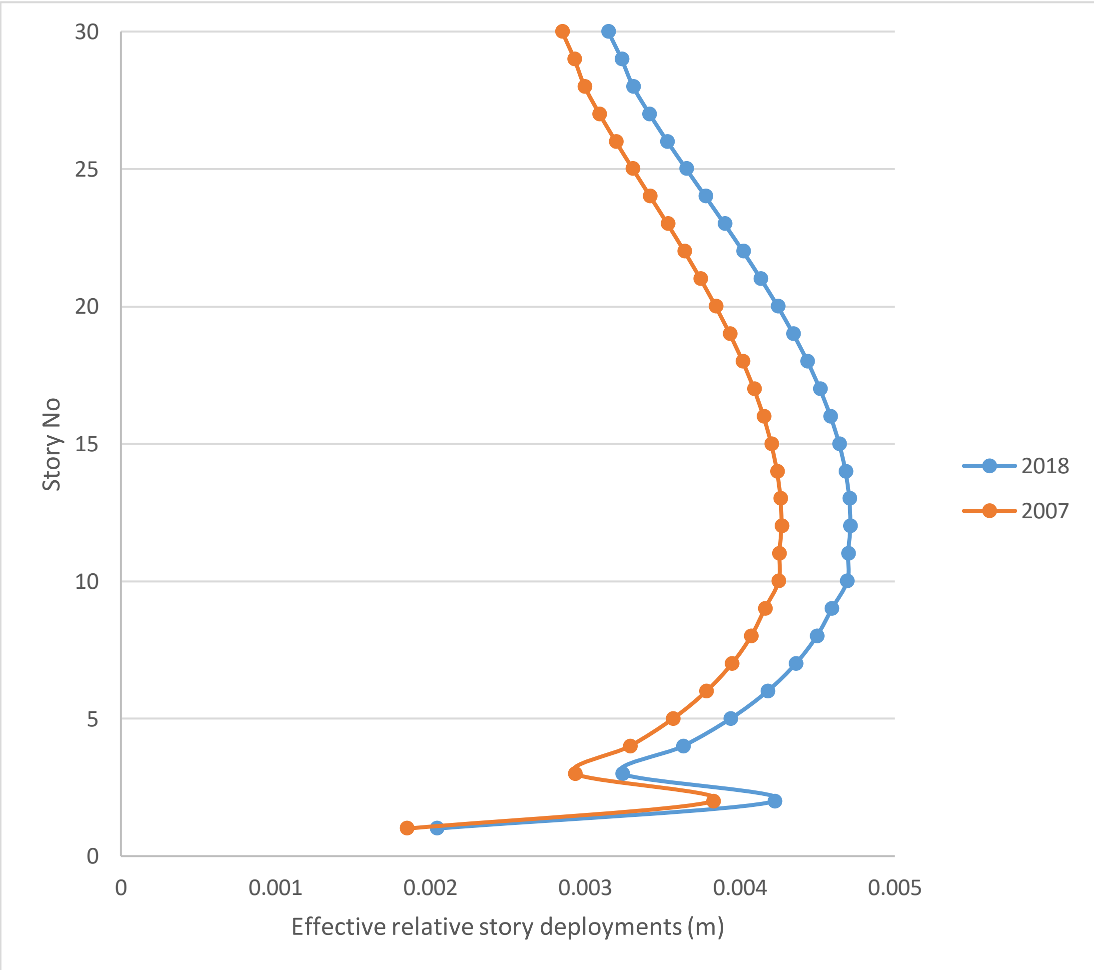


Figure 4.44 Comparison of X-direction-enabled relative story offsets.

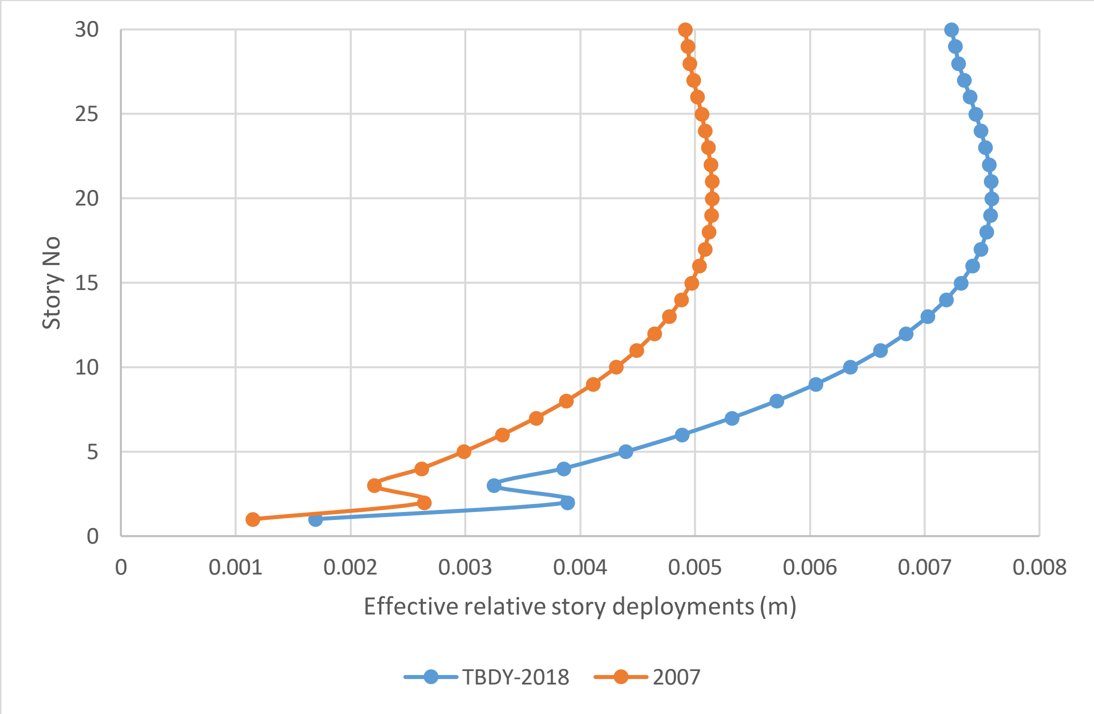


Figure 4.45 Comparison of y-direction-enabled relative story offsets

4.6.4 Comparison of Internal Forces

As a result of the linear analysis using the mode combination method, the story shear forces of the building are presented in Figure 4.46 for the X direction and in Figure 4.47 for the Y direction. While the X-direction base shear force calculated for the 2018 regulation is 29043 kN, this value reached 26300 kN in the 2007 regulation. For the Y direction, base shear forces of 29043 kN for 2018 regulation and 19725 kN for 2007 regulation are found. As a result of these values, it is understood that the horizontal loads that the structure will be exposed to will be 11% higher in the X direction and 48% in the Y direction in the 2018 regulation.

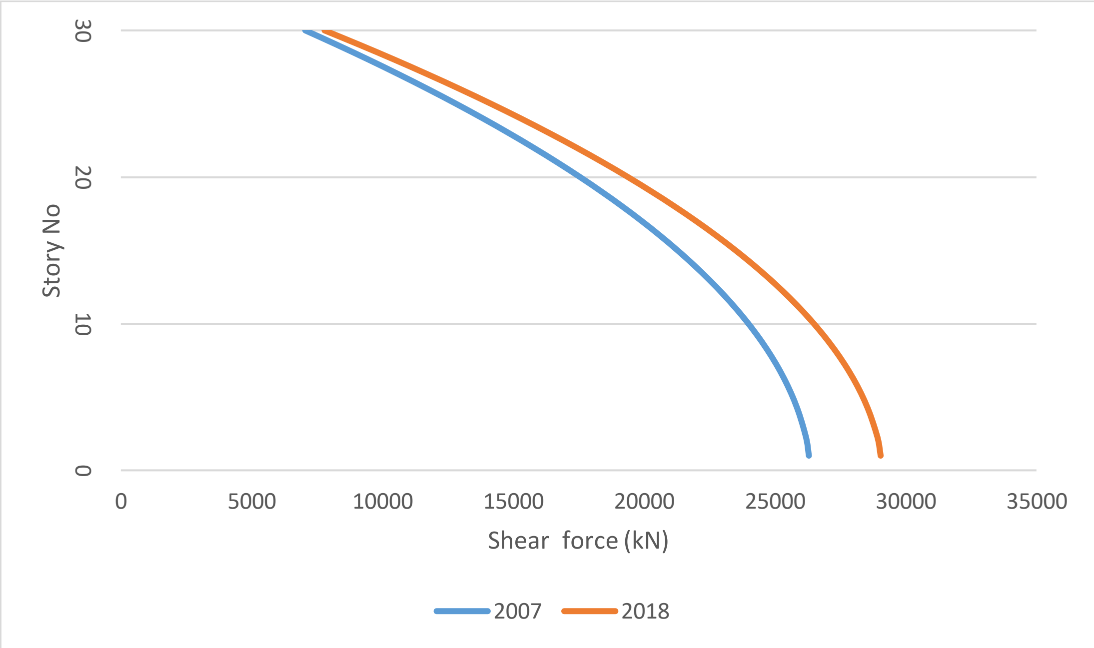


Figure 4.46 Comparison of X-direction story shear forces.

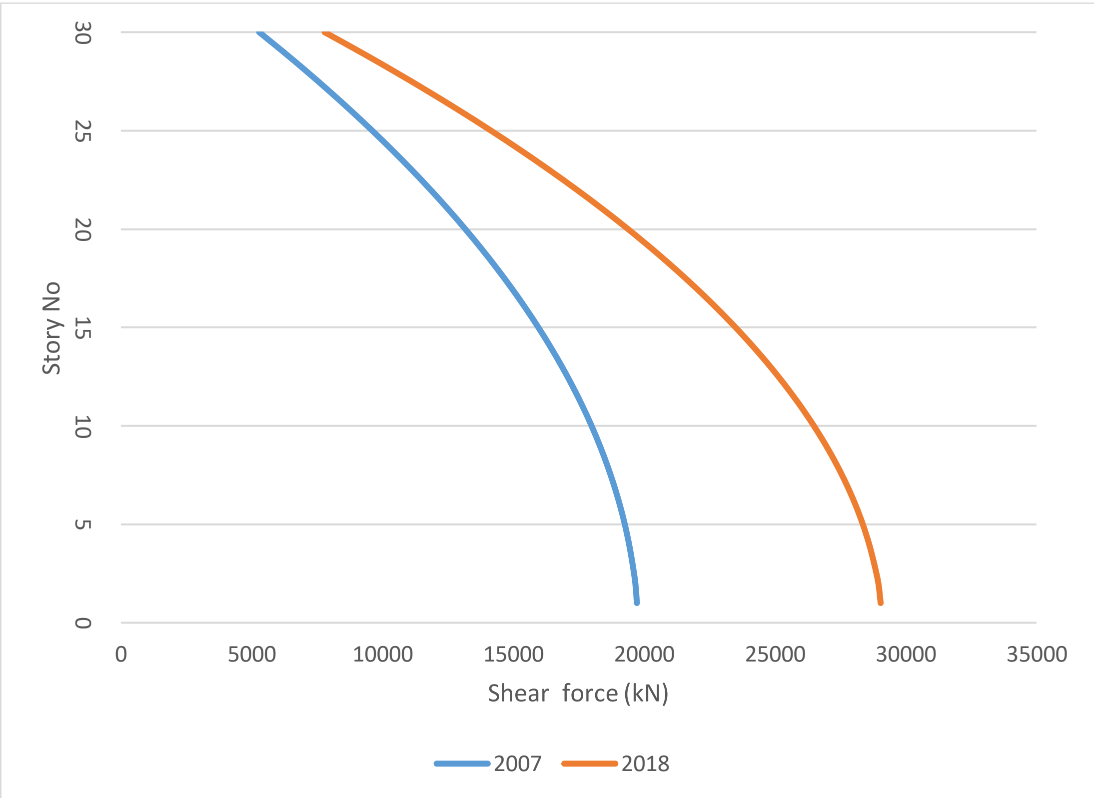


Figure 4.47 Y direction story shear forces

4.7 Design Phase III

In the design phase I, the internal force capacities are checked in order to ensure the performance target of uninterrupted use (IO-KK) by using linear calculation methods under the influence of DD-4 earthquake in the design phase II of the load-bearing system, which is pre-sizing under the impact of DD-2 level earthquake and in design phase II. In the last phase, design phase III, it will be shown that the performance target of preventing immigrants under the influence of the DD-1 earthquake, the largest earthquake considered, has been achieved. In the calculation of performance targets, non-linear three-dimensional analysis of the high-rise structural system will be made in the time domain. It will be shown that the strain and internal force values obtained from this analysis are below the limit conditions in the regulation.

4.7.1 Determination of Elastic Spectrum

The earthquake level to be used for the design phase-III of tall buildings is DD1. The structure is located in Istanbul Province at latitude 41.131012° and longitude 29.017682°. The SS and S1 values for the horizontal elastic spectrum are taken from the Turkey Earthquake Hazard Maps.

$$S_S = 1.227$$

$$S_1 = 0.341$$

The local ground class is assumed to be ZB. The short period local ground effect coefficient is 0.9 and the 1.0 second-period local ground effect coefficient is 0.8 from Table 4.1 and Table 4.2. From here, the design spectral acceleration coefficients.

$$S_{DS} = S_s\, F_s = 1.277\ x\, 0.900 = 1.104$$

$$S_{D1} = S_1\, F_1 = 0.371\, x\, 0.800 = 0.237$$

In order to plot the horizontal design spectrum, the corner periods will be calculated:

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.397}{1.739} = 0.046 \text{ s}$$

$$T_B = \frac{S_{D1}}{S_{DS}} = \frac{0.397}{1.739} = 0.228 \text{ s}$$

$$T_L = 6\ s$$

The horizontal elastic design spectrum is defined in Figure4.48, depending on the natural vibration period:

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

$$S_{ae}(T) = S_{DS} \qquad (T_A \le T \le T_B)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad (T_B \le T \le T_L)$$

$$S_{ae}(T) = \frac{S_{D1}T_L}{T^2} \qquad (T_L \le T)$$

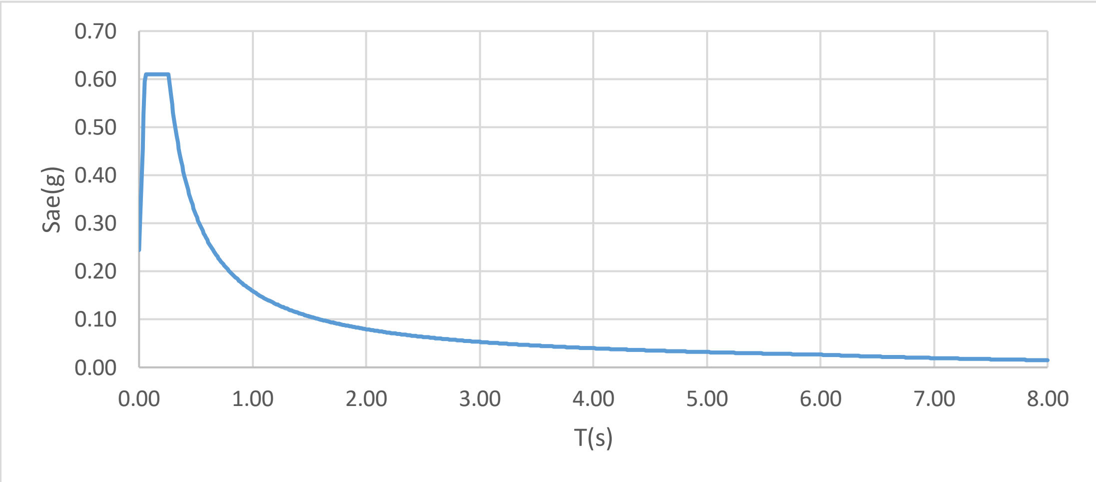


Figure 4.48 DD-1 earthquake level horizontal elastic design acceleration spectrum.

To create the vertical elastic design acceleration spectrum, SaeD (T) vertical elastic design spectral accelerations are determined depending on the period and the short period design spectral acceleration coefficient. The vertical elastic design spectrum is defined in Figure 4.49

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

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

$$S_{aeD}(T) = 0.8S_{DS}\frac{T_{BD}}{T} \qquad T_{BD} \le T \le T_{LD}$$

Calculation ofT_{AD}, T_{BD} andT_{LD}, which are the corner periods of the vertical spectrum:

$$T_{AD} = \frac{T_A}{3} \qquad ; T_{BD} = \frac{T_B}{3} \qquad ; T_{LD} = \frac{T_L}{2}$$

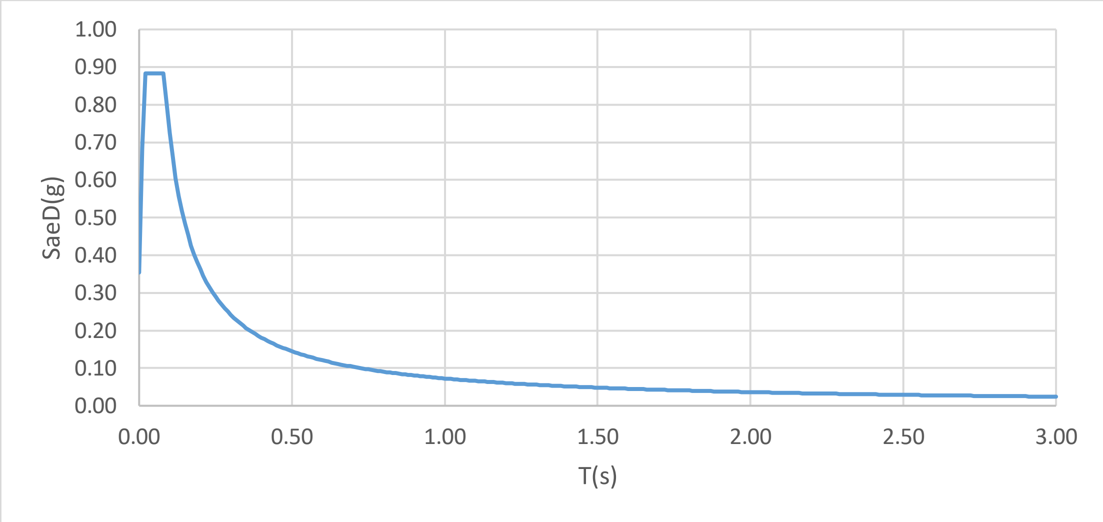


Figure 4.49 DD-1 earthquake level vertical elastic design acceleration spectrum

4.7.2 Rules for Design Phase III

Additional eccentricity will not be applied at this phase. The damping ratio will be calculated as 2.5%. Load components to be used in Phase-III Eq. It is defined in eq. (4.16).

$$G + Q_e + 0.2S + E_d^{(H)} + 0.3E_d^{(Z)} \quad , \quad Q_e = nQ \tag{4.16}$$

Before starting the non-linear earthquake calculation, a non-linear static calculation will be made in which static vertical loads other than $E_d^{(H)}$ are applied incrementally to the carrier system in Eq. 4.16, and the internal force and strain values found here will be taken as the initial values of the horizontal earthquake calculation.

Two horizontal components of earthquake effects will always be taken perpendicular to each other. Second-order effects will be considered.

Average material strengths given in Table 4.24 shall be used for material strengths. The effective section stiffnesses of linearly modeled walls and stories will be taken according to Table 4.25.

$$(EI)_e = \frac{M_y}{\theta_y}\frac{L_s}{3}$$

Table 4.24 Average strength of materials.

Material	Predicted Strengths
Concrete	$f_{ce} = 1.3 f_{ck}$
Reinforcement Steel	$f_{ye} = 1.2 f_{yk}$
Structural Steel (S235)	$fye = 1.5 fyk$
Structural Steel (S275)	$fye = 1.3 fyk$
Structural Steel (S355)	$fye = 1.1 fyk$
Structural Steel (S460)	$fye = 1.1 fyk$

Table 4.25 Rigidity factors for Strength-base design (DGT).

Concrete Member	Effective Stiffness Multiplier	
Wall – Slab (In-Plane)	**Axial**	**Shear**
Shear Wall	0.50	0.50
Basement Shear wall	0.80	0.50
Slab	0.25	0.25
Wall – Slab (Out Of Plane)	**Flexure**	**Shear**
Shear Wall	0.50	1.00
Basement Shear wall	0.25	1.00
Slab	0.50	1.00
Frame member	**Flexure**	**shear**
Coupling beam	0.15	1.00
Frame beam	0.35	1.00
Frame column	0.70	1.00
Wall (equivalent strut)	0.50	0.50

4.7.3 Selecting Earthquake Records

Earthquake magnitudes compatible with DD1 earthquake level, phase mechanisms, distances to the fault and ground conditions are taken into account in the selection of earthquake records to be used while calculating in the time domain. For the three-dimensional earthquake calculation, eleven earthquake sets stipulated by the regulation are selected. The maximum set of records selected from the same earthquake is limited to three. Earthquake records to be used and their properties are presented in Table 4.26. (Kuzu 2018).

Table 4.26 Selected earthquake records.

Time history	RSN	Earthquake Name	Station Name	Year	Rup (km)	Vs30 (m/sn)	Moment Mag.
Time history-1	1944	Anza-02	Idyllwild - Kenworthy Fire Sta.	2001	20.34	382.44	4.92
Time history-2	3887	Tottori, Japan	HRSH03	2000	73.92	486.78	6.61
Time history-3	4223	Niigata, Japan	NIGH06	2004	31.6	493.27	6.63
Time history-4	5654	Iwate	IWTH22	2008	29.83	532.13	6.90
Time history-5	4873	Chuetsu-oki	Kashiwazaki City Takayanagicho	2007	20.03	561.59	6.80
Time history-6	4874	Chuetsu-oki	Oguni Nagaoka	2007	20	561.59	6.80
Time history-7	4891	Chuetsu-oki	Iizuna Imokawa	2007	66.44	591.2	6.80
Time history-8	3884	Tottori, Japan	HRS021	2000	36.33	409.29	6.61
Time history-9	1524	Chi-Chi, Taiwan	TCU095	1999	45.18	446.63	7.62
Time history-10	1487	Chi-Chi, Taiwan	TCU047	1999	35	520.37	7.62
Time history-11	1642	Sierra Madre	Cogswell Dam - Right Abutment	1991	22	680.37	5.61

4.7.4 Scaling Earthquake Records

Both components of the selected eleven earthquake records are downloaded from the Peer (2019) Database and transferred to the Etab 20 program. Apart from the records, the reference DD1 earthquake design spectrum to be scaled is also defined in the program. The definition of all records to the program is presented in Figure 4.50.

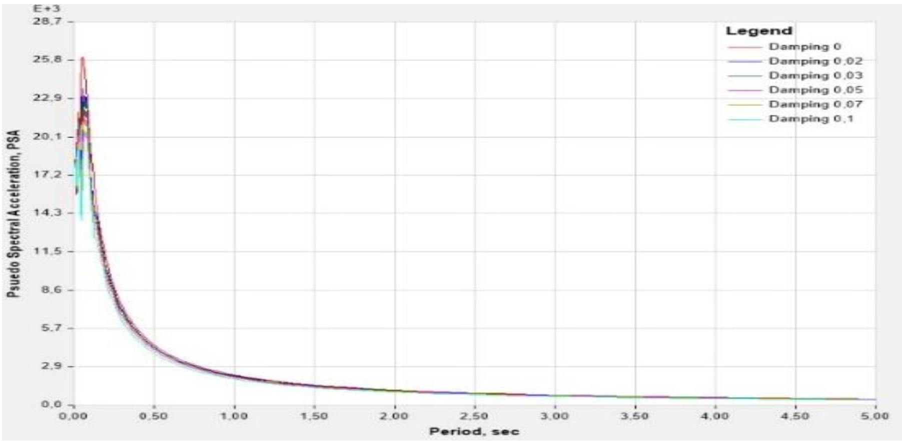


Figure 4.50 Earthquake records are defined in the program

Both components of the selected eleven earthquake records are downloaded from the Peer (2019) Database. The selected earthquake records are scaled according to the DD1 earthquake design spectrum and converted into earthquake ground motions to be used in the calculation in the time history (Kayhan 2012). Earthquake records after scaling are presented in Figure 4.51. After the scaling process is completed, the resultant horizontal spectrum is formed by taking the square root of the sum of the squares of the two horizontal components of each earthquake record set selected to be used in the three-dimensional calculation. By taking the average of all the resulting horizontal spectra, it is shown in Figure 4.52 that the amplitudes between the 0.2Tp and 1.5Tp periods are 1.3 times the amplitudes in the same period range of the DD1 design spectrum. The Tp value for our build is 2.749s.

0.2Tp = 0.549s 1.5Tp = 4.12s

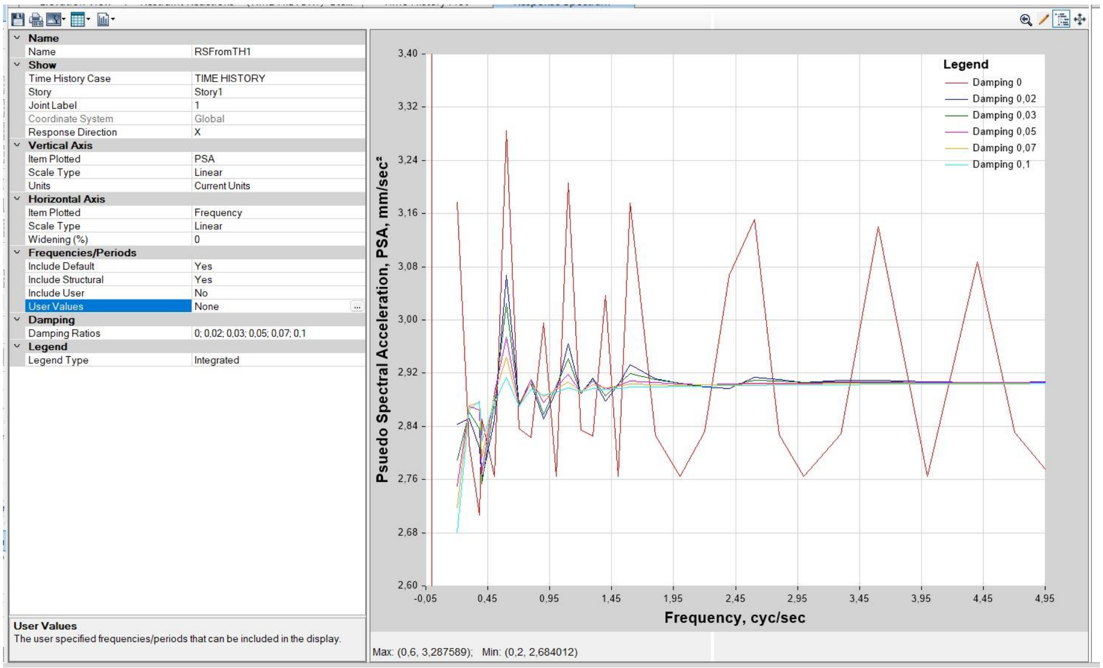


Figure 4.51 Scaled earthquake records.

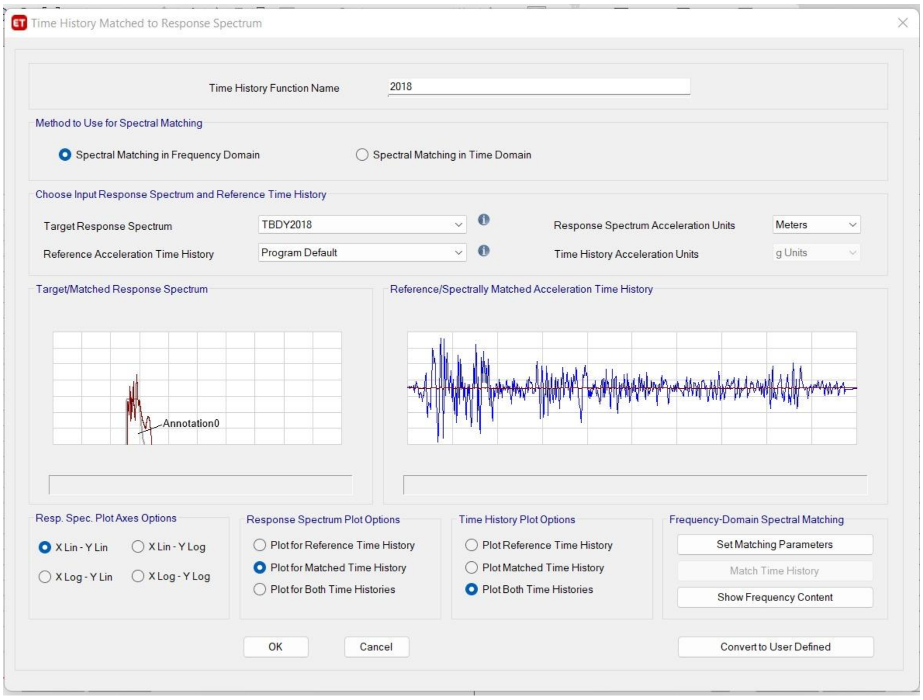


Figure 4.52 Average of resultant spectra.

4.7.5 Determination of Internal Force and Strain Limits

In order to determine the performance level of prevention of collapse, limit values for the concrete and reinforcing steel unit strains called ε_c^{CP} and $\varepsilon_s^{(cp)}$ should be determined for each section. Eq.(4.18) and Eq.(4.19) were used in the calculations of these values. (Celep 2017, Celep 2018, Foroughi and Yüksek 2019)

$$\varepsilon_c^{CP} = 0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018$$

$$\omega_{we} = \alpha_{se}\rho_{sh,min}\frac{f_{ywe}}{f_{ce}} \tag{4.18}$$

$$\alpha_{se} = \left(1 - \frac{\sum a_i^2}{6b_0h_0}\right)\left(1 - \frac{s}{2b_0}\right)\left(1 - \frac{s}{2h_0}\right) \quad ; \quad \rho_{sh} = \frac{A_{sh}}{b_k s}$$

Since there is no circular section element in our structure, only the formulas valid for rectangular section elements are defined.

$$\varepsilon_s^{CP} = 0.4\varepsilon_{su} \tag{4.19}$$

The plastic rotation limit values$\theta_P^{(CP)}$ in the stacked plastic behavior should be obtained from the curvature analysis to be made. Eq. (4.20) was used to calculate the plastic rotation limit values of the sections.

$$\theta_P^{(CP)} = \frac{2}{3}\left[(\emptyset_u - \emptyset_y)L_P\left(1 - 0.5\frac{L_P}{L_s}\right) + 4.5\emptyset_u d_b\right] \tag{4.20}$$

Deformation demands of structural elements with ductile behavior will be calculated as the average of the absolute values of 22 calculation results obtained from 11 earthquake recording sets. The values found should not exceed the limit values calculated by Eq. (4.18), Eq. (4.19) and Eq. (4.20).

Demands for the shear forces of shear walls, columns and beams will be calculated by adding one standard deviation to the average of the absolute values of the 22 calculation results obtained from the 11 earthquake record sets. However, it will not be less than 1.20 times the average or more than 1.50 times. The shear force values found should be lower than the shear force capacities calculated using the average material strengths.

The average of the relative story drifts obtained from 22 nonlinear calculations in the time history should not exceed 0.03, and the maximum relative story drift obtained from a single calculation should not exceed 0.045.

The cross-sectional properties and plastic rotational capacities of the beam and column groups, whose moment-curvature graphs were obtained, are calculated in Table 4.27

Table 4.27 Cross-sectional damage limits of structural elements

Section	Axial Force kN	ϕy	ϕu	Lp (m)	Ls (m)	dp (m)	Θp(CP)
B3(6 Ø20)	-475.7818	0.0066	0.1356	0.15	2	0.020	0.0347
B16 (8 Ø20)	-464.1229	0.024054	0.00076	0.15	2	0.020	0.01464
B26(8 Ø18)	-282.044	0.000826	0.03464	0.3	2	0.018	0.027783
B30 (8 Ø18)	-276.1327	0.000826	0.03464	0.3	2	0.018	0.027783
B58 (8 Ø18)	-322.521	0.001509	0.03464	0.3	2	0.018	0.027362
C3(24Ø22)	728.9481	0.016186	0.00035	0.15	2	0.022	0.028167
C7(24Ø22)	1023.356	0.024054	0.00076	0.15	2	0.022	0.016399

Fiber elements are assigned to the shear walls in the analysis program. The unit strain limit values of shear walls for concrete and reinforcement are presented in Table 4.28

Table 4.28 Shear wall strain limit values

Shear Walls Dimension (cm)	α_{se}	Reinforcemnt Area As(mm)	ρ_{sh}	ω_{we}	$\varepsilon_c^{(cp)}$	$\varepsilon_s^{(cp)}$
50x600	0.4870	5231.24	0.012	0.06337669	0.012	0.032
50x860	0.4740	7693	0.012	0. 06251228	0.012	0.032
50x1300	0.4680	11693.4	0.012	0.06251228	0.012	0.032

4.7.6 Evaluation of Phase III Analysis Results

By using the determined limit values, the plastic hinges stacked in columns and beams and fiber elements in the walls were defined, and the analysis was completed with 22 analyzes using 11 earthquake record sets according to the deformation in the time history.

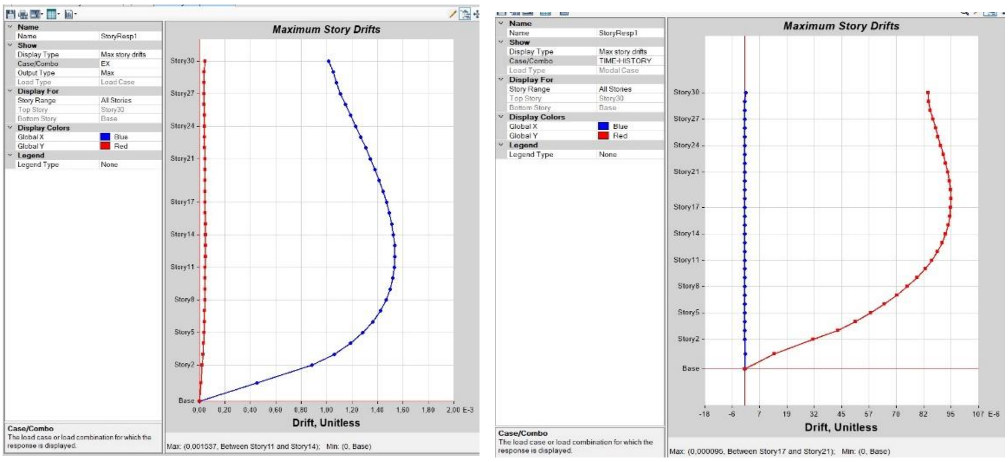


Figure 4.53 Relative story drifts (maximum of 22 analyses)

Maximum beam shear force values obtained from 22 analyzes in time history are presented in Table 4.29. Since our structure is symmetrical, only different beam groups are given in the chart. Values are the largest shear forces generated by 22 records on all floors. By taking the average of the obtained shear forces, a standard deviation is added and the total shear force values are calculated. In TBDY (2018), this value is required to be between 1.5 and 1.2 times the average. Since the values we found are less than 1.2 times, the comparison will be made over 1.20xVaverage.

Table 4.29 Beam maximum shear forces

Earthquake Record No	Element No				
	B3	B16	B26	B30	B58
1447	241.5717	230.3956	290.12	76.7158	237.2515
1524	106.5643	66.8307	114.6254	10.981	371.6048
1642	341.5717	330.4	390.1037	76.7158	237.2515
1944	176.5758	210.4651	171.9609	143.2583	102.8866
3887	169.516	198.9381	198.7362	151.4721	125.9438
4223	215.5539	338.518	110.4347	40.0635	358.0189
4873	217.1482	359.179	363.8326	101.6837	152.0832
4874	176.5758	210.4651	265.5783	143.2583	102.8866
4891	382.2114	400.2466	375.7049	71.0391	207.6449
5654	293.42	264.6196	198.3456	410.037	368.7374
3884	261.0371	293.2799	288.3479	50.3095	258.9876
1447-1	265.7289	253.4352	319.132	84.38738	260.9767
1524-1	117.2207	73.51377	126.0879	12.0791	408.7653
1642-1	375.7289	363.44	429.1141	84.38738	260.9767
1944-1	194.2334	231.5116	189.157	157.5841	113.1753
3887-1	186.4676	218.8319	218.6098	166.6193	138.5382
4223-1	237.1093	372.3698	121.4782	44.06985	393.8208
4873-1	238.863	395.0969	400.2159	111.8521	167.2915
4874-1	194.2334	231.5116	292.1361	157.5841	113.1753
4891-1	420.4325	440.2713	413.2754	78.14301	228.4094
5654-1	322.762	291.0816	218.1802	451.0407	405.6111
3884-1	287.1408	322.6079	317.1827	55.34045	284.8864
Vaverage.	246.4394	277.1368	264.1982	121.7555	240.8601
Standard Dev	23.68	27.089	43.86	11.657	33.648
Total	270.1194	304.2258	308.0582	133.4125	274.5081
1.2x Vave.	295.7273	332.5641	317.0378	146.1066	289.0322
Vr	1892.48	1892.48	1623.154	1432.832	1100.589

Shear capacities of all beam groups were calculated using the average material strengths and presented in the table. Among the beam groups. Since all the shear forces obtained from the analysis results were below the capacities, the brittle collapse of the beams under the shear force was prevented and ductile behavior was ensured.

Table 4.30 shows the ratios of the maximum plastic rotations obtained from 22 nonlinear analyzes in the time domain for the DD1 earthquake level of all beam groups to the maximum rotation limit calculated for the section failure limit.

Table 4.30 Beam plastic rotation rates

Earthquake Record No	Element No				
	B3	B16	B26	B30	B58
1447	0	0	0.002223	0.026103	0.000521
1524	0	0	0.002446	0.026154	0.000569
1642	0	0	0.002436	0.026147	0.000596
1944	0	0	0.002475	0.026142	0.000646
3887	0	0	0.002508	0.026138	0.000705
4223	0	0	0.002546	0.026132	0.000773
4873	0	0	0.002584	0.026125	0.000849
4874	0	0	0.002621	0.026117	0.000928
4891	0	0	0.002655	0.026107	0.001011
5654	0	0	0.002685	0.026096	0.001094
3884	0	0	0.00271	0.026084	0.001176
1447-1	0	0	0.002728	0.02607	0.001256
1524-1	0	0	0.002738	0.026055	0.001332
1642-1	0	0	0.002739	0.026038	0.001403
1944-1	0	0	0.002731	0.026019	0.001468
3887-1	0	0	0.002711	0.025999	0.001525
4223-1	0	0	0.002679	0.025977	0.001574
4873-1	0	0	0.002633	0.025954	0.001613
4874-1	0	0	0.002574	0.025929	0.001641
4891-1	0	0	0.0025	0.025902	0.001656
5654-1	0	0	0.002409	0.025873	0.001657
3884-1	0	0	0.002301	0.025842	0.001643

As a result of 22 non-linear analyzes in the time history for the DD1 earthquake level, the plastic rotation values in B3, B16 beam groups were calculated as zero. The highest plastic rotation rate is calculated as 2.65% in the 1524 record in the B30 beam. As a result, the plastic rotation values of all beams remained well below the limits.

Maximum column shear force values obtained from 22 analyzes in time history are presented in Table 4.31.

Table 4.31 Column maximum shear forces.

Earthquake Record No	Element No	
	C3	C7
1447	655.2844	265.1185
1524	308.842	228.2318
1642	655.2844	265.1185
1944	801.7235	486.1997
3887	701.374	607.4339
4223	104.8627	42.4259
4873	630.3389	263.7003
4874	601.7235	311.7069
4891	356.7487	144.3353
5654	636.9729	628.7146
3884	368.7538	149.1924
1447-1	589.756	238.6066
1524-1	268.6925	198.5616
1642-1	589.756	238.6066
1944-1	645.4995	422.9937
3887-1	551.2366	546.6905
4223-1	91.23053	36.91053
4873-1	567.305	237.3303
4874-1	485.4995	71.18497
4891-1	321.0738	129.9018
5654-1	2642.166	108.9817
3884-1	331.8784	134.2731
Vaverage.	586.6365	261.6463
Standard Dev	23.68	27.089
Total	610.3165	288.7353
1.2x Vave.	703.9638	313.9756
Vr	3593.14	3014.324

The maximum shear force values of the C3 and C7 columns, where the greatest shear force occurs in the existing structure, are calculated at all floors of the 22 analysis. By taking the average of the obtained shear forces, a standard deviation is added and the total shear force values are calculated. In TBDY (2018), this value is required to be between 1.5 and 1.2 times the average. Since the values we found are less than 1.2 times, the comparison will be made over 1.20xVort. Brittle collapse of the columns under the effect of shear force is prevented.

Table 4.32 shows the ratios of the maximum plastic rotations obtained from 22 nonlinear analyzes in the time history for the DD1 earthquake level of all columns to the maximum rotation limit calculated for the section failure limit.

Table 4.32 Column plastic rotation rates

Earthquake Record No	Element No	
	C3	C7
1447	0.005141	0.022217
1524	0.003984	0.02534
1642	3.26E-07	0.000007
1944	0.004181	0.01894
3887	0.000001	0.02534
4223	0.00392	0.000013
4873	0.000001	0.020915
4874	0.003736	0.02534
4891	0.000001	0.000018
5654	0.002418	0.01593
3884	0.000002	0.000023
1447-1	0.003334	0.025342
1524-1	0.000002	0.01715
1642-1	0.003032	0.000029
1944-1	0.000002	0.025343
3887-1	0.001533	-0.01821
4223-1	0.000002	0.000034
4873-1	0.002646	0.025345
4874-1	0.000001	0.01891
4891-1	0.002393	0.000038
5654-1	0.000001	0.02535
3884-1	0.002121	0.000041

In the C7 column, where the greatest rotation rate occurs, it can reach 2.5% in the 1447 record. It is obvious that the columns do not exceed the plastic rotation limits calculated according to TBDY (2018).

The largest shear force values obtained from 22 analyzes for shear wall elements are presented in Table 4.33. By taking the average of the maximum cutting forces obtained, a standard deviation is added and the cutting force values defined as the total was calculated. In TBDY (2018), this value is required to be between 1.5 and 1.2 times the average. Since the values we found are less than 1.2 times, the comparison will be made over 1.20xVort.

Table 4.33 Shear wall maximum shear forces.

Earthquake Record No	Element No		
	P1	P2	P3
1447	27417.27	1496.236	21576.09
1524	4553.976	248.523	3583.764
1642	27417.27	1496.236	21576.09
1944	50280.41	2743.941	39568.31
3887	62817.86	3428.144	49434.69
4223	4387.481	239.4369	3452.74
4873	26373.54	1439.277	20754.73
4874	50280.41	2743.941	39568.31
4891	14926.46	814.5778	11746.42
5654	127067.7	6934.435	99996.29
3884	12628.74	1628.253	3260.94
1447-1	24401.37	1331.65	19202.72
1524-1	5464.771	298.2276	4300.517
1642-1	30158.99	1645.859	23733.7
1944-1	44749.57	2442.107	35215.8
3887-1	75381.44	4113.773	59321.63
4223-1	4826.229	263.3806	3798.014
4873-1	23472.45	1280.956	18471.71
4874-1	60336.5	3292.729	47481.97
4891-1	16419.1	896.0356	12921.06
5654-1	113090.3	6171.647	88996.7
3884-1	15154.49	1953.904	3913.128
Vaverage.	37345.74	2131.967	28721.61
Standard Dev	23.68	27.089	28.089
Total	37369.42	2159.056	28749.69
1.2x Vave.	44814.89	2558.36	34465.93
Vr	58541.36	9485.15	46542.65

The highest shear force values for all shear wall groups occur at the bottom floor of the building. Shear wall shear capacity values are calculated using average material strengths.

Since the fiber definition is made in the non-linear analytical model for shear walls, the unit strains in concrete and reinforcement will be determined. In Table 4.34, the ratio of the unit strains of the concrete in the wall groups to the previously calculated unit strain limit value is given.

Table 4.34 Concrete strain rates of Shear wall.

Earthquake Record No	Element No		
	P1	P2	P3
1447	0.015769	0.025962	0.0375
1524	0.021154	0.024038	0.019231
1642	0.018365	0.040385	0.036538
1944	0.016442	0.046154	0.053846
3887	0.022308	0.063462	0.053846
4223	0.025481	0.075	0.076923
4873	0.025481	0.085577	0.145192
4874	0.02875	0.118269	0.102885
4891	0.02875	0.108654	0.156731
5654	0.037308	0.170192	0.216346
3884	0.0525	0.138462	0.281731
1447-1	0.0205	0.024663	0.04125
1524-1	0.0275	0.022837	0.021154
1642-1	0.023875	0.038365	0.040192
1944-1	0.021375	0.043846	0.059231
3887-1	0.029	0.060288	0.059231
4223-1	0.033125	0.07125	0.084615
4873-1	0.033125	0.081298	0.159712
4874-1	0.037375	0.112356	0.113173
4891-1	0.037375	0.103221	0.172404
5654-1	0.0485	0.161683	0.237981
3884-1	0.06825	0.131538	0.309904

In Table 4.35, the ratio of the reinforcement unit strains in the wall groups to the previously calculated strain limit value is given.

Table 4.35 Reinforcing unit strain rates of Shear Walls.

Earthquake Record No	Element No		
	P1	P2	P3
1447	0.0164	0.027	0.039
1524	0.022	0.025	0.02
1642	0.0191	0.042	0.038
1944	0.0171	0.048	0.056
3887	0.0232	0.066	0.056
4223	0.0265	0.078	0.08
4873	0.0265	0.089	0.151
4874	0.0299	0.123	0.107
4891	0.0299	0.113	0.163
5654	0.0388	0.177	0.225
3884	0.0546	0.144	0.293
1447-1	0.02132	0.02565	0.0429
1524-1	0.0286	0.02375	0.022
1642-1	0.02483	0.0399	0.0418
1944-1	0.02223	0.0456	0.0616
3887-1	0.03016	0.0627	0.0616
4223-1	0.03445	0.0741	0.088
4873-1	0.03445	0.08455	0.1661
4874-1	0.03887	0.11685	0.1177
4891-1	0.03887	0.10735	0.1793
5654-1	0.05044	0.16815	0.2475
3884-1	0.07098	0.1368	0.3223

The largest unit strain rate of concrete and reinforcement in shear walls was obtained from the 3884 record. While the ε_s^{CP} ratio is 32.2%, the ε_c^{CP} ratio was calculated as 30.99% on the P3 fret. The deformation rates of the walls that meet the earthquake load the most are also found to be higher than the others.

Since the maximum plastic rotation and unit strain values found as a result of the calculations are below the limit conditions determined by the regulation for all element groups; There was no need to average the results from the 22 analyzes. Even the maximum values found are well below the limit values. As a result of these analyzes, it has been observed that there is a limited amount of nonlinear behavior in our structure.

CHAPTER 5

5. CONCLUSION

In accordance with the provisions of the Turkish Building Earthquake Code, which is described as the new earthquake regulation in Turkey and entered into force in January 2019, the design and performance analysis of a high-rise building where reinforced concrete core shear and frame system are used together are carried out. Since the building is 30 stories and 94 m high, it is considered a tall building in all regions of our country. The main conclusions drawn from the study are presented below:

The minimum base shear force is effective for both directions of the base shear force of the structure. When the linear analysis is done with the mode combination method, the structural system behavior coefficient of the structure doesn't need to reduce in the X direction and in the Y direction according to the minimum base shear force rule.

In the design phase I, seismic combinations are more effective than vertical load combinations in dimensioning the beams. The reason for this is the carrier system behavior coefficient, which decreases due to the effect of the minimum story shear force in high-rise buildings. The decrease in R increases the seismic forces acting on the building and is effective in element sizing.

It has been seen that the elements exposed to the greatest stress among the structural system elements whose D/C ratios are calculated in the design phase II are the shear walls. Since the shear walls meet almost all of the earthquake loads, the shear capacities of these elements are very close to the limit values. After the shear walls, the most critical values occur in the tie beams. In particular, the moment capacities the shear walls that meet the earthquake loads approached 0.8 values. Since earthquake loads are not effective on the columns, they are far below the capacity limits. For this reason, the core system should be chosen very well in the building of high-rise carrier systems. Shear wall dimensions and tie beam dimensions should never be compromised.

In the comparison of TBDY 2018 and DBYBHY 2007 regulations, when the spectral acceleration values for the region where our building is located are examined, the natural vibration period of our structure is 4.42 s. The new regulation spectral acceleration value for 0.01 g. While this value is 0.0182 g in the old regulation. İs calculated as. . The maximum effective relative story drift in the X direction occurred at the 10st story. While this value is 4.2 mm according to the 2007 regulation, it is calculated as 4.7 mm on the same story in the 2018 regulation. When looking at the Y direction, the maximum values occur on the 20th story, and while these values are 5.1 mm for the 2007 regulations, they are 7.5 mm for the 2018 regulations. It is understood that the horizontal loads that the structure will be exposed to will be 12% higher in the X direction and 45% in the Y direction in the 2018 regulation. While the X-direction base shear force calculated for the 2018 regulation is 29043 kN, this value reached 26300 kN in the 2007 regulation. For the Y direction, base shear forces of 29043 kN for 2018 regulation and 19725 kN for 2007 regulation are found. As a result of these values, it is understood that the horizontal loads that the structure will be exposed to will be 11% higher in the X direction and 48% in the Y direction in the 2018 regulation.

For design phase III, the earthquake records of the nonlinear analysis in the time domain are selected as eleven sets and scaled. The amplitudes of the mean of the resultant spectra between the 0.2Tp and 1.5Tp periods should be greater than 1.3 times the amplitudes in the same period range of the DD1 design spectrum. Due to the high period of our structure, the necessary amplitudes could be obtained. However, for buildings with shorter periods corresponding to the TA and TB range of the DD1 design spectrum of the 0.2Tp period, the number of real earthquake records that can be used is very small, considering the earthquake magnitudes, fault mechanisms, distances to the fault, and ground conditions.

Although the thesis subject is not very high quality, the modeling program's analysis, design phase, and results are very long. In addition, the total size of the 22 earthquake analyses of the modeled structure also covers an area of around 700 GB. For this reason, high-capacity computers are required in such high-rise solutions and take a lot of CPU time.

As a result of nonlinear analysis in the time history, the maximum shear force value is calculated at the base of the structure. Since the shear walls carry almost all the earthquake loads in the structural system of our structure, the shear force values acting on the walls have increased considerably. The shear body reinforcements selected in the first two analysis phases are insufficient and III. In this phase, the shear force capacity of the shear wall is increased by increasing the body reinforcements and brittle collapse is prevented.

Since the capacity limits of all plastic rotation, unit strain, and shear force calculated for the prevention of migration performance targets are not exceeded, our structure has met the performance target of preventing migration under the DD1 earthquake level.

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APPENDICES

APPENDIX A: B4 Beam Reinforcement Areas

Story No	Label	Section	Location	(-) Moment	As Top	(+) Moment	As Bot
				kN-m	mm²	kN-m	mm²
Story30	B4	B (70*50cm)-	End-I	-39.411	249	52.3806	343
Story30	B4	B (70*50cm)-	Middle	-218.405	1034	183.781	967
Story30	B4	B (70*50cm)-	End-J	-16.3247	172	14.541	239
Story29	B4	B (70*50cm)-	End-I	-17.6897	96	21.7759	109
Story29	B4	B (70*50cm)-	Middle	-308.571	1330	266.9356	1128
Story29	B4	B (70*50cm)-	End-J	-5.7908	88	2.3551	85
Story28	B4	B (70*50cm)-	End-I	-13.3039	63	15.3242	71
Story28	B4	B (70*50cm)-	Middle	-337.072	1441	295.2074	1259
Story28	B4	B (70*50cm)-	End-J	-4.9193	84	2.8032	72
Story27	B4	B (70*50cm)-	End-I	-13.9381	58	15.8756	74
Story27	B4	B (70*50cm)-	Middle	-353.889	1501	312.2462	1316
Story27	B4	B (70*50cm)-	End-J	-4.893	96	2.7219	100
Story26	B4	B (70*50cm)-	End-I	-5.067	62	16.8429	74
Story26	B4	B (70*50cm)-	Middle	-372.773	1571	331.4179	1388
Story26	B4	B (70*50cm)-	End-J	-4.9431	113	2.9789	123
Story25	B4	B (70*50cm)-	End-I	-5.374	71	17.9185	76
Story25	B4	B (70*50cm)-	Middle	-393.317	1650	352.2782	1469
Story25	B4	B (70*50cm)-	End-J	-5.3192	138	3.2525	145
Story24	B4	B (70*50cm)-	End-I	-5.6665	80	18.854	78
Story24	B4	B (70*50cm)-	Middle	-414.916	1735	374.2142	1556
Story24	B4	B (70*50cm)-	End-J	-5.6858	164	1.6165	166
Story23	B4	B (70*50cm)-	End-I	-5.9423	88	2.3452	84
Story23	B4	B (70*50cm)-	Middle	-437.052	1824	396.6969	1647
Story23	B4	B (70*50cm)-	End-J	-6.0404	189	1.4883	189
Story22	B4	B (70*50cm)-	End-I	-6.2029	96	2.5715	95
Story22	B4	B (70*50cm)-	Middle	-459.312	1915	419.3092	1740
Story22	B4	B (70*50cm)-	End-J	-6.3828	215	1.3723	211
Story21	B4	B (70*50cm)-	End-I	-6.4493	103	2.7882	107

Story21	B4	B (70*50cm)-	Middle	-481.366	2006	441.7165	1833
Story21	B4	B (70*50cm)-	End-J	-6.7129	240	1.2662	233
Story20	B4	B (70*50cm)-	End-I	-6.6823	111	2.996	118
Story20	B4	B (70*50cm)-	Middle	-502.947	2096	463.6466	1925
Story20	B4	B (70*50cm)-	End-J	-7.0307	266	1.1679	254
Story19	B4	B (70*50cm)-	End-I	-6.9029	118	3.1954	130
Story19	B4	B (70*50cm)-	Middle	-523.829	2183	484.872	2014
Story19	B4	B (70*50cm)-	End-J	-7.3362	292	1.0757	276
Story18	B4	B (70*50cm)-	End-I	0	125	3.3867	142
Story18	B4	B (70*50cm)-	Middle	-543.818	2267	505.1943	2100
Story18	B4	B (70*50cm)-	End-J	-7.6294	318	0.9879	297
Story17	B4	B (70*50cm)-	End-I	0	136	3.5704	155
Story17	B4	B (70*50cm)-	Middle	-562.732	2347	524.431	2181
Story17	B4	B (70*50cm)-	End-J	-7.9104	343	0.9033	318
Story16	B4	B (70*50cm)-	End-I	0	147	3.7468	167
Story16	B4	B (70*50cm)-	Middle	-580.397	2422	542.4032	2258
Story16	B4	B (70*50cm)-	End-J	-8.1793	369	0.8203	339
Story15	B4	B (70*50cm)-	End-I	0	158	3.9163	179
Story15	B4	B (70*50cm)-	Middle	-596.626	2490	558.9228	2327
Story15	B4	B (70*50cm)-	End-J	-8.4361	395	2.1532	360
Story14	B4	B (70*50cm)-	End-I	0	170	4.0792	192
Story14	B4	B (70*50cm)-	Middle	-611.212	2551	573.7803	2390
Story14	B4	B (70*50cm)-	End-J	-8.6808	421	2.1259	381
Story13	B4	B (70*50cm)-	End-I	0	181	4.2351	205
Story13	B4	B (70*50cm)-	Middle	-623.914	2602	586.7298	2443
Story13	B4	B (70*50cm)-	End-J	-8.9137	447	2.0966	402
Story12	B4	B (70*50cm)-	End-I	0	193	4.3829	217
Story12	B4	B (70*50cm)-	Middle	-634.436	2643	597.4709	2486
Story12	B4	B (70*50cm)-	End-J	-9.1358	473	2.0642	424
Story11	B4	B (70*50cm)-	End-I	0	206	4.535	231
Story11	B4	B (70*50cm)-	Middle	-642.416	2672	605.6363	2516
Story11	B4	B (70*50cm)-	End-J	-9.3467	499	2.0284	444
Story10	B4	B (70*50cm)-	End-I	0	219	4.6973	245
Story10	B4	B (70*50cm)-	Middle	-647.433	2686	610.8058	2532
Story10	B4	B (70*50cm)-	End-J	-9.5259	526	0	469
Story9	B4	B (70*50cm)-	End-I	0	233	4.8362	261
Story9	B4	B (70*50cm)-	Middle	-648.848	2683	612.3218	2532
Story9	B4	B (70*50cm)-	End-J	-9.7122	547	0	489
Story8	B4	B (70*50cm)-	End-I	0	247	4.957	275
Story8	B4	B (70*50cm)-	Middle	-645.927	2656	609.4413	2507
Story8	B4	B (70*50cm)-	End-J	-9.9	575	0	515
Story7	B4	B (70*50cm)-	End-I	0	260	5.0936	290
Story7	B4	B (70*50cm)-	Middle	-635.828	2606	599.6629	2458
Story7	B4	B (70*50cm)-	End-J	-10.0552	601	0	540
Story6	B4	B (70*50cm)-	End-I	0	273	5.2384	304

Story6	B4	B (70*50cm)-	Middle	-621.369	2523	585.0626	2376
Story6	B4	B (70*50cm)-	End-J	-10.2025	628	1.6611	567
Story5	B4	B (70*50cm)-	End-I	0	287	5.3865	319
Story5	B4	B (70*50cm)-	Middle	-598.851	2401	562.2877	2255
Story5	B4	B (70*50cm)-	End-J	-10.3345	655	1.8872	595
Story4	B4	B (70*50cm)-	End-I	0	300	5.5284	333
Story4	B4	B (70*50cm)-	Middle	-566.3	2231	529.3457	2085
Story4	B4	B (70*50cm)-	End-J	-10.4407	683	2.2054	624
Story3	B4	B (70*50cm)-	End-I	0	301	5.0987	331
Story3	B4	B (70*50cm)-	Middle	-521.891	2007	484.3368	1862
Story3	B4	B (70*50cm)-	End-J	-10.058	712	2.9455	660
Story2	B4	B (70*50cm)-	End-I	0	319	2.647	333
Story2	B4	B (70*50cm)-	Middle	-483.759	1801	443.9278	1641
Story2	B4	B (70*50cm)-	End-J	-3.5786	742	3.2471	732
Story1	B4	B (70*50cm)-	End-I	0	399	2.35	408
Story1	B4	B (70*50cm)-	Middle	-316.648	1034	275.3758	1034
Story1	B4	B (70*50cm)-	End-J	-4.5877	840	2.7595	822

APPENDIX B: B30 Beam Reinforcement Areas

Story No	Label	Section	Location	(-) Moment	As Top	(+) Moment	As Bot
				kN-m	mm²	kN-m	mm²
Story30	B30	B (70*55cm)	End-I	-591.522	2714	0	1322
Story30	B30	B (70*55cm)	Middle	-187.999	1096	227.3798	1138
Story30	B30	B (70*55cm)	End-J	0	878	212.7395	1138
Story29	B30	B (70*55cm)	End-I	-635.544	2879	0	1368
Story29	B30	B (70*55cm)	Middle	-202.019	1138	231.1371	1138
Story29	B30	B (70*55cm)	End-J	0	947	269.54	1199
Story28	B30	B (70*55cm)	End-I	-625.113	2864	0	1383
Story28	B30	B (70*55cm)	Middle	-197.5	1138	229.6373	1138
Story28	B30	B (70*55cm)	End-J	0	910	265.7588	1169
Story27	B30	B (70*55cm)	End-I	-626.41	2859	0	1374
Story27	B30	B (70*55cm)	Middle	-198.319	1138	228.8174	1138
Story27	B30	B (70*55cm)	End-J	0	917	266.3005	1175
Story26	B30	B (70*55cm)	End-I	-625.374	2856	0	1373
Story26	B30	B (70*55cm)	Middle	-197.811	1138	227.5189	1138
Story26	B30	B (70*55cm)	End-J	0	914	265.635	1171
Story25	B30	B (70*55cm)	End-I	-624.212	2849	0	1369
Story25	B30	B (70*55cm)	Middle	-197.278	1138	225.9685	1138
Story25	B30	B (70*55cm)	End-J	0	912	264.7884	1167
Story24	B30	B (70*55cm)	End-I	-622.383	2839	0	1365
Story24	B30	B (70*55cm)	Middle	-196.391	1138	224.1269	1138
Story24	B30	B (70*55cm)	End-J	0	909	263.4888	1161
Story23	B30	B (70*55cm)	End-I	-619.925	2826	0	1358
Story23	B30	B (70*55cm)	Middle	-200.607	1138	222.0035	1138
Story23	B30	B (70*55cm)	End-J	0	906	261.7393	1153
Story22	B30	B (70*55cm)	End-I	-616.768	2810	0	1350
Story22	B30	B (70*55cm)	Middle	-199.008	1138	219.5969	1138
Story22	B30	B (70*55cm)	End-J	0	901	259.6805	1144
Story21	B30	B (70*55cm)	End-I	-612.872	2790	0	1341
Story21	B30	B (70*55cm)	Middle	-197.016	1138	216.9057	1138
Story21	B30	B (70*55cm)	End-J	0	895	225.9723	1138
Story20	B30	B (70*55cm)	End-I	-608.195	2767	0	1329
Story20	B30	B (70*55cm)	Middle	-194.611	1127	213.9275	1138
Story20	B30	B (70*55cm)	End-J	0	888	221.3378	1138
Story19	B30	B (70*55cm)	End-I	-602.701	2739	0	1316
Story19	B30	B (70*55cm)	Middle	-191.771	1110	210.6587	1138
Story19	B30	B (70*55cm)	End-J	0	880	215.8361	1138
Story18	B30	B (70*55cm)	End-I	-596.353	2707	0	1301
Story18	B30	B (70*55cm)	Middle	-188.478	1091	207.0951	1138
Story18	B30	B (70*55cm)	End-J	0	871	209.4276	1138
Story17	B30	B (70*55cm)	End-I	-589.118	2671	0	1283
Story17	B30	B (70*55cm)	Middle	-184.712	1068	203.2317	1138
Story17	B30	B (70*55cm)	End-J	0	860	202.0731	1138
Story16	B30	B (70*55cm)	End-I	-580.958	2630	0	1264
Story16	B30	B (70*55cm)	Middle	-180.453	1043	199.0628	1138

Story16	B30	B (70*55cm)	End-J	0	848	211.2425	1138
Story15	B30	B (70*55cm)	End-I	-571.835	2585	0	1242
Story15	B30	B (70*55cm)	Middle	-175.681	1014	210.0474	1138
Story15	B30	B (70*55cm)	End-J	0	835	202.8364	1138
Story14	B30	B (70*55cm)	End-I	-561.707	2535	0	1218
Story14	B30	B (70*55cm)	Middle	-170.373	983	205.4706	1138
Story14	B30	B (70*55cm)	End-J	0	820	210.6084	1138
Story13	B30	B (70*55cm)	End-I	-550.528	2480	0	1192
Story13	B30	B (70*55cm)	Middle	-164.505	948	200.5175	1138
Story13	B30	B (70*55cm)	End-J	0	803	201.48	1138
Story12	B30	B (70*55cm)	End-I	-538.246	2420	0	1164
Story12	B30	B (70*55cm)	Middle	-158.047	910	195.1749	1138
Story12	B30	B (70*55cm)	End-J	0	786	203.7849	1138
Story11	B30	B (70*55cm)	End-I	-524.803	2355	0	1138
Story11	B30	B (70*55cm)	Middle	-150.969	868	205.477	1138
Story11	B30	B (70*55cm)	End-J	0	766	203.346	1138
Story10	B30	B (70*55cm)	End-I	-510.114	2283	0	1138
Story10	B30	B (70*55cm)	Middle	-143.224	822	199.4023	1138
Story10	B30	B (70*55cm)	End-J	0	745	202.1534	1138
Story9	B30	B (70*55cm)	End-I	-494.123	2206	0	1138
Story9	B30	B (70*55cm)	Middle	-134.782	773	192.804	1127
Story9	B30	B (70*55cm)	End-J	0	721	194.3773	1136
Story8	B30	B (70*55cm)	End-I	-476.728	2122	0	1138
Story8	B30	B (70*55cm)	Middle	-125.587	719	185.6427	1085
Story8	B30	B (70*55cm)	End-J	0	696	185.6784	1085
Story7	B30	B (70*55cm)	End-I	-457.786	2032	0	1138
Story7	B30	B (70*55cm)	Middle	0	667	177.8647	1039
Story7	B30	B (70*55cm)	End-J	0	669	176.2243	1029
Story6	B30	B (70*55cm)	End-I	-437.168	1934	0	1138
Story6	B30	B (70*55cm)	Middle	0	638	170.0914	993
Story6	B30	B (70*55cm)	End-J	-118.041	680	165.9525	969
Story5	B30	B (70*55cm)	End-I	-414.709	1827	0	1138
Story5	B30	B (70*55cm)	Middle	0	605	162.9085	951
Story5	B30	B (70*55cm)	End-J	-134.331	775	154.7865	903
Story4	B30	B (70*55cm)	End-I	-390.084	1711	0	1097
Story4	B30	B (70*55cm)	Middle	0	570	157.1248	917
Story4	B30	B (70*55cm)	End-J	-149.665	871	142.5747	832
Story3	B30	B (70*55cm)	End-I	-363.482	1589	0	1021
Story3	B30	B (70*55cm)	Middle	0	531	150.9878	881
Story3	B30	B (70*55cm)	End-J	-165.788	965	129.4366	754
Story2	B30	B (70*55cm)	End-I	-330.531	1424	0	905
Story2	B30	B (70*55cm)	Middle	0	487	144.211	846
Story2	B30	B (70*55cm)	End-J	-181.223	1061	113.9542	668
Story1	B30	B (70*55cm)	End-I	-218.238	1138	0	690
Story1	B30	B (70*55cm)	Middle	0	396	134.5727	794
Story1	B30	B (70*55cm)	End-J	-204.976	1138	0	593

APPENDIX C: B45 Beam Reinforcement Areas

Story	Label	Section	Location	(-) Moment	As Top	(+) Moment	As Bot
				kN-m	mm²	kN-m	mm²
Story30	B49	B (45*65cm)	End-I	0	740	92.224	819
Story30	B49	B (45*65cm)	Middle	-39.6391	727	89.2043	819
Story30	B49	B (45*65cm)	End-J	-305.643	2318	0	1114
Story29	B49	B (45*65cm)	End-I	0	770	91.8613	819
Story29	B49	B (45*65cm)	Middle	0	769	87.2635	819
Story29	B49	B (45*65cm)	End-J	-320.928	2435	0	1162
Story28	B49	B (45*65cm)	End-I	0	757	91.8575	819
Story28	B49	B (45*65cm)	Middle	-44.7513	758	87.5264	819
Story28	B49	B (45*65cm)	End-J	-317.743	2418	0	1160
Story27	B49	B (45*65cm)	End-I	0	758	91.5763	819
Story27	B49	B (45*65cm)	Middle	-44.8382	759	87.3853	819
Story27	B49	B (45*65cm)	End-J	-318.138	2421	0	1161
Story26	B49	B (45*65cm)	End-I	0	757	91.2352	819
Story26	B49	B (45*65cm)	Middle	-44.6331	758	87.2926	819
Story26	B49	B (45*65cm)	End-J	-317.708	2418	0	1160
Story25	B49	B (45*65cm)	End-I	0	756	90.7973	819
Story25	B49	B (45*65cm)	Middle	-44.397	757	87.1549	819
Story25	B49	B (45*65cm)	End-J	-317.227	2415	0	1159
Story24	B49	B (45*65cm)	End-I	0	754	90.2649	819
Story24	B49	B (45*65cm)	Middle	-44.0672	756	86.9888	819
Story24	B49	B (45*65cm)	End-J	-316.515	2410	0	1157
Story23	B49	B (45*65cm)	End-I	0	751	89.6331	819
Story23	B49	B (45*65cm)	Middle	-43.6451	754	86.7893	819
Story23	B49	B (45*65cm)	End-J	-315.576	2403	0	1154
Story22	B49	B (45*65cm)	End-I	0	747	88.8974	819
Story22	B49	B (45*65cm)	Middle	-43.1199	752	86.5554	819
Story22	B49	B (45*65cm)	End-J	-314.381	2394	0	1151
Story21	B49	B (45*65cm)	End-I	0	743	88.053	819
Story21	B49	B (45*65cm)	Middle	-42.4838	749	86.2851	819
Story21	B49	B (45*65cm)	End-J	-312.905	2382	0	1146
Story20	B49	B (45*65cm)	End-I	0	738	87.0949	819
Story20	B49	B (45*65cm)	Middle	-41.7286	746	87.0503	819
Story20	B49	B (45*65cm)	End-J	-311.126	2369	0	1141
Story19	B49	B (45*65cm)	End-I	0	732	91.7196	819
Story19	B49	B (45*65cm)	Middle	-40.8467	742	86.7296	819
Story19	B49	B (45*65cm)	End-J	-309.021	2353	0	1134
Story18	B49	B (45*65cm)	End-I	0	726	90.6224	819
Story18	B49	B (45*65cm)	Middle	-39.8303	737	86.3648	819

Story18	B49	B (45*65cm)	End-J	-306.568	2334	0	1126
Story17	B49	B (45*65cm)	End-I	0	718	89.3819	819
Story17	B49	B (45*65cm)	Middle	-38.6714	731	87.9536	819
Story17	B49	B (45*65cm)	End-J	-303.744	2312	0	1117
Story16	B49	B (45*65cm)	End-I	0	710	87.9906	819
Story16	B49	B (45*65cm)	Middle	-37.3621	724	87.2504	819
Story16	B49	B (45*65cm)	End-J	-300.527	2287	0	1106
Story15	B49	B (45*65cm)	End-I	0	700	86.441	819
Story15	B49	B (45*65cm)	Middle	-35.8941	716	86.4777	819
Story15	B49	B (45*65cm)	End-J	-296.893	2258	0	1094
Story14	B49	B (45*65cm)	End-I	0	689	84.7251	801
Story14	B49	B (45*65cm)	Middle	-34.2587	708	87.878	819
Story14	B49	B (45*65cm)	End-J	-292.818	2227	0	1081
Story13	B49	B (45*65cm)	End-I	-75.4746	711	82.8344	782
Story13	B49	B (45*65cm)	Middle	-32.4466	698	86.9692	819
Story13	B49	B (45*65cm)	End-J	-288.276	2191	0	1065
Story12	B49	B (45*65cm)	End-I	-79.4972	751	80.3902	762
Story12	B49	B (45*65cm)	Middle	-30.4482	687	86.7703	819
Story12	B49	B (45*65cm)	End-J	-283.24	2152	0	1048
Story11	B49	B (45*65cm)	End-I	-83.625	791	78.2961	741
Story11	B49	B (45*65cm)	Middle	-28.2534	676	85.2661	804
Story11	B49	B (45*65cm)	End-J	-277.683	2109	0	1030
Story10	B49	B (45*65cm)	End-I	-87.8502	819	76.0052	718
Story10	B49	B (45*65cm)	Middle	-25.8483	662	83.7016	790
Story10	B49	B (45*65cm)	End-J	-271.567	2062	0	1009
Story9	B49	B (45*65cm)	End-I	-92.1476	819	73.5101	693
Story9	B49	B (45*65cm)	Middle	-23.2249	648	82.4169	777
Story9	B49	B (45*65cm)	End-J	-264.871	2010	0	986
Story8	B49	B (45*65cm)	End-I	-88.7977	819	70.7955	667
Story8	B49	B (45*65cm)	Middle	-20.3689	632	81.0261	763
Story8	B49	B (45*65cm)	End-J	-257.555	1954	0	961
Story7	B49	B (45*65cm)	End-I	-92.9815	819	67.8406	638
Story7	B49	B (45*65cm)	Middle	-17.2616	615	79.5222	748
Story7	B49	B (45*65cm)	End-J	-249.57	1893	0	934
Story6	B49	B (45*65cm)	End-I	-97.2156	819	64.6278	606
Story6	B49	B (45*65cm)	Middle	-13.8883	596	77.8995	731
Story6	B49	B (45*65cm)	End-J	-240.879	1826	0	904
Story5	B49	B (45*65cm)	End-I	-101.482	819	61.1344	572
Story5	B49	B (45*65cm)	Middle	-10.2327	575	76.443	716
Story5	B49	B (45*65cm)	End-J	-231.438	1754	0	872
Story4	B49	B (45*65cm)	End-I	-105.743	819	57.3221	535
Story4	B49	B (45*65cm)	Middle	-6.261	552	75.171	703
Story4	B49	B (45*65cm)	End-J	-221.135	1676	0	836
Story3	B49	B (45*65cm)	End-I	-117.695	845	0	553
Story3	B49	B (45*65cm)	Middle	-2.0892	527	73.3406	691

Story3	B49	B (45*65cm)	End-J	-210.093	1591	0	819
Story2	B49	B (45*65cm)	End-I	-120.332	865	0	565
Story2	B49	B (45*65cm)	Middle	0	492	71.7961	675
Story2	B49	B (45*65cm)	End-J	-194.126	1472	0	819
Story1	B49	B (45*65cm)	End-I	-125.361	893	0	584
Story1	B49	B (45*65cm)	Middle	0	430	70.2218	664
Story1	B49	B (45*65cm)	End-J	-167.772	1271	0	819

CURRICULUM VITAE