

**ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF
SCIENCE, ENGINEERING, AND TECHNOLOGY**

**DESIGN AND SEISMIC
PERFORMANCE EVALUATION OF RC HIGH-RISE
BUILDING ACCORDING TO TURKISH SEISMIC CODE 2018**

M.Sc. THESIS

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**Department of Civil Engineering
Structure Engineering Programme**

MAY 2020

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**BETONARME YÜKSEK
BİR BİNANIN TBDY 2018'E GÖRE
TASARIMI VE DEPREM PERFORMANSININ İNCELENMESİ**

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To My Family and My Spouse



FOREWORD

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ABBREVIATIONS

DBYBHY	: The 2007 seismic design code of Turkey
SAP	: Integrated Software for Structural Analysis and Design
TBDY	: Turkey New Seismic Design Code 2018 (TBSC)
IYBDY	: Istanbul Province Design Code for High Rise Building
SRSS	: Square Root of Sum of Squares
ETABS	: Extended 3D Analysis of Building Systems
FEMA	: Federal Emergency Management Agency
SEAOC	: Structural Engineers Association of California
MH	: Minimum Damage
KK	: Immediate occupancy Performance Level
KH	: Controlled Damage
CG	: Life Safety Performance Level
IH	: Advanced Damage
GO	: Collapse Prevention Performance Level
ICC PC	: International Code Council Performance Code



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DESIGN AND SEISMIC PERFORMANCE EVALUATION OF RC HIGH-RISE BUILDING ACCORDING TO TURKISH SEISMIC CODE 2018

SUMMARY

As the human population increases day by day, the need for tall buildings (high-rise) also increases, because to have more rentable areas and accommodation facility for the city population. The desire to build tall structures came from centuries. In the past, the tall buildings were so challenging because of the lack of earthquake information and computer technologies. Nevertheless, today developments on computer technologies, facilities such as lifts and being familiar with engineers with earthquake motions helped most to build tall buildings. Today tall buildings become a challenge among developed and developing countries.

Today engineers use regulation and rules for tall buildings that were prepared for conventional buildings. These rules put a restriction for engineers to select specific and easy structural systems for tall buildings because these rules are given according to the strength based design and linear elastic analysis of the structures. The development of non-linear behavior of structural systems enables engineers to select earthquake resistant designs and more sophisticated analysis.

The lack of information about non-linear behavior of structures and using codes made according to strength, which was made under seismic hazard, forced engineers to build low-rise and mid-rise buildings.

The rapid increase of population and urbanization and the need to increase the density of population in cities forced engineers to develop new approaches to be able to build high rise buildings. Engineers developed new design criteria based on the deformation of the structural systems. It is also called deformation-based design or performance – based earthquake engineering. In recent years some recommended documents developed and published on design of tall buildings, such as, Los Angeles Tall Building Structural Design Council (2005, 2008), San Francisco SEAONC – Structural Engineers Association of Northern California (2007), and CTBUH- Council on Tall Building and Urban Habitat (2008). These published documents are related to performance-based design. In this approach, some damage (deformation) is allowed in the structural elements of the structure which develops under the earthquake ground motion. These deformations are determined by numerical analysis of the structural system of structures under earthquake effects. Istanbul High Rise Buildings Earthquake Code (IHRBEC 2008) published by Istanbul metropolitan Municipality. Moreover, new seismic code of Turkey (Turkey Building Seismic Code 2018, TBSC 2018) has new chapter, chapter 13, for high-rise (tall) buildings.

The purpose of this study is to design and analyze a 24 story reinforced concrete tall building according to TBSC 2018, by using strength based design. After that, the performance evaluation is carried out of this tall building according to chapter 13 of (TBSC 2018) code.

This 24 stories tall building is designed by strength-based design method, which is linear elastic analysis is used. Moreover, its performance assessment is accomplished by using nonlinear behavior according to TBSC 2018. For tall buildings different performance levels are defined according to different earthquakes, namely DD-1, DD-2, DD-3 and DD-4, maximum, design, frequent and service earthquakes, respectively. The strength based design is achieved under DD-4 and DD-2 earthquakes, the deformation based analysis and performance assessment is carried out by using four level of earthquakes mentioned above. Normal and advanced performance targets are defined. The structural system should be satisfied Immediate Occupancy and Controlled damage under frequent and maximum credible earthquakes, respectively. In chapter 13 of TBSC 2018, there are different steps to complete the design of a tall building. The structural system of the tall building is designed by using strength based approach considering DD4 and DD-2 earthquakes, then its performance evaluation is carried out by using nonlinear analysis, in the last step the nonlinear deformations are compared whether the target performance level is satisfied or not under four earthquake levels. The desired performance levels are: Operational, Immediate, Life Safety and Collapse Prevention. In the nonlinear time history analysis of 3D structural model of the tall building is accomplished by using ETABS software.

To perform this, in first and second chapter, a piece of brief information is given about the TBSC 2018, IHRBEC 2018 and methods of design used. Then in the third chapter, the case study is explained, and the last chapter of this thesis contains conclusions and decisions given for future design and research.

The structural design of a RC tall building is accomplished according to the ACI code and the TBSC 2018 codes. First, the structure is designed according to TBSC 2018 by using strength based approach. All the design results are given in tables and figures in the thesis text body. And then, the earthquake performance of the structural system of the tall buildings is evaluated according to Chapter 13 of TBSC 2018. This chapter prescribes that there are three steps to check the performance of a tall building. These steps are; the strength based design, deformation based analysis and performance assessments. The design and performance assessment results are given in tables and figures, comparatively.

BETONARME YÜKSEK BİR BİNANIN TBDY 2018'E GÖRE TASARIMI VE DEPREM PERFORMANSININ İNCELENMESİ

ÖZET

Deprem tehlikesi altında mevcut yönetmeliklerin çoğunda yapıların tasarımı dayanım bazlı tasarım yaklaşımına göre hazırlanmıştır. Bu yaklaşım mühendisleri aynı kurallar ve içerikle az katlı ya da orta yükseklikte kısıtlı yapılar tasarlamaya yönlendirmektedir. Artan nüfus ve hızlı şehirleşme, barınma ve çalışma alanları ihtiyacı mühendisleri yüksek bina tasarlamaya zorlamaktadır. Bu nedenle son yıllarda mevcut standart ve yönetmeliklerin yeterlilikleri tasarım felsefesi bakımından sorgulanmıştır. Performansa göre tasarım yaklaşımını esas alan özel düzenlemelere ilişkin yoğun çalışmalar hızlanmış ve dünyada daha çok yüksek binaların sismik tasarımı için tasvsiye edilen belgeler adı altında yayımlanmıştır. Bu gelişmelere paralel olarak ülkemizde yüksek binalar için İstanbul İli için taslak halde bulunan İstanbul Yüksek Binalar Deprem Yönetmeliği 2008 (İYBDY, 2008) hazırlanmıştır.

Yüksek binalar günümüzde şehirleşmenin bir ihtiyacı olarak yapılmakla birlikte, etkileyici yapılar olarak da inşa edilmektedir. Bina yüksekliği veya kat adedine bağlı olarak genel kabul görmüş bir “yüksek bina” tanımı günümüzde mevcut değildir. Türkiye Bina Deprem Yönetmeliği 2018, depremselliği yüksek olan bölgelerde bina özelliklerine de bağlı olarak 70m’den daha yüksek binaları yüksek bina olarak kabul etmekte ve bu binalara daha ayrıntılı tasarım kuralları öngörmektedir. Yurdumuzdaki yönetmeliklerinin kapsamadığı konularda ve özel binaların tasarımında sıklıkla başvurulanan Amerikan Yönetmeliği ASCE 7-10 sık karşılaşılan yapı sistemleri için yaklaşık 50m’yi yüksek bina sınırı olarak kabul etmektedir. CTBUH kat adedi 14 ve üstü veya yüksekliği 50m ve üstü olan binaların yüksek bina olarak sayılabileceğini bildirmektedir. CTBUH ayrıca yüksekliği 300m’den fazla olan binaları “süper yüksek bina” ve yüksekliği 600m’den fazla olan binaları “mega yüksek bina” olarak sınıflandırmaktadır.

Yüksek bina, genellikle zemine gömülü bodrum katlarının üstünde yer alan az katlı zemin üstü baza katları ve onun üstünde yükselen bir kule bölümünden oluşur. Yüksek binalarda deprem ve rüzgar etkilerinin önemli bir bölümü planda orta bölümde yer alan bir çekirdek perde sistemi ve kolon ve kirişlerin oluşturduğu çerçeveler tarafından taşınır. Yüksek binalarda iki doğrultuda eğilme rijitliği yanında, düşey eksen etrafında yeterli burulma rijitliği bulunması önemlidir. Uygun bir rijitlik düzenlenmesinde, ilk iki titreşim periyodu, iki doğrultuda eğilme ve üçüncüsü ise, burulma titreşimi olarak ortaya çıkar. Simetrik olmayan sistemlerde eğilme titreşimlerinde burulma katkısı ve burulma titreşiminde eğilme etkisinin katkısı da bulunur. Çekirdek bölümünün dışında, kirişsiz döşemelerin seçildiği durumda, çekirdek perdeleri ile dışta bulunan kolonları döşeme şeritlerinin bağlayacağı ve bu şeritlerin düzlem içi kuvvet etkisinde kalacağı unutulmamalıdır. Bunun gibi orta bölümdeki birden fazla olan çekirdek perdelerini birbirine bağlayan kiriş ve döşeme şeritleri kısa olan açıklıklarından dolayı büyük zorlamalar etkisinde bulunur. Binanın zemin üstündeki ve bodrum katlarındaki kulenin taşıyıcı sistemi yanında, genişleyen bölümlerde ek kolonlar ve çevre perdesi bulunur. Özellikle

çevre perdesinin kesildiği kattaki kat döşemesi, geçiş (transfer) döşemesi olarak isimlendirilir ve çekirdek perdelerinin etkilerinin bir bölümü düzlemsel gerilmelerle çevre perdesine iletilir.

Depreme dayanıklı bina tasarımı gelişmiş ülkelerde olduğu gibi, yurdumuzda da ileri düzeyde uygulanmaktadır. Genellikle uygulanmakta olan deprem yönetmelikleri az katlı binaların tasarım kurallarını içerir. Yüksek binaların deprem tasarımına ilişkin bu yönetmeliklerin kapsamadığı özel konular aşağıda gibi özetlenebilir:

- Karmaşık dinamik davranış: Az katlı binalarda genelde birinci modun etkisi belirleyici olurken, yüksek binalarda yüksek modların etkisi belirgin olarak ortaya çıkar; bazı durumlarda tasarımı yönlendiren mod şekilleri olur.

- Eksenel kuvvetler: Kat sayısının fazla olmasından dolayı kolon ve perdelerin maruz kalacağı eksenel kuvvet büyük olur; buna bağlı olarak kolon ve perde kesitleri büyür, dolayısıyla rijitlikleri de artar. Kolonun artan rijitliği sayesinde kirişle meydana getireceği etkileşim, ilave eksenel kuvvetin ortaya çıkmasına sebep olur. Dolayısıyla, kolonların eksenel kuvvet tasarımına özel önem verilmesi gerekir.

- Boyut etkisi: Mevcut araştırmaların çoğu, küçük ölçekli numuneler üzerinde yapılan deney sonuçlarına dayanmaktadır. Bu tür araştırmalara dayanan yönetmelik kurallarının çok büyük boyutlu yüksek bina taşıyıcıları için geçerliliği konusunda yapılan araştırma sayısı sınırlıdır.

- Sönüm: Yüksek bina doğal sönüm oranının geleneksel %5'ten daha küçük, %2 ~ %3 mertebesinde olduğu yapılan araştırmalarla ortaya konmuştur. Yüksek bina sönüm oranının küçük olması, dinamik performansının daha düşük olması anlamına gelir. Günümüzde yüksek binaların deprem tasarımına ilişkin özel konular, deprem yönetmelikleri tasarım kurallarına ek olarak doğrusal olmayan zaman tanım alanında analiz ile yapılan performansa dayalı tasarım yöntemiyle çözülmektedir.

Bu çalışmanın amacı, 24 katlı betonarme bir perde çerçeve taşıyıcı sisteme sahip binanın yürürlükte bulunan Türkiye Bina Deprem Yönetmeliği 2018 (TBDY, 2018) 'e göre analiz aşamaları kullanarak tasarımının ve gerçekleştirilmesi ve TBDY 2018' de yer alan analiz aşamaları kullanarak binanın deprem performansının belirlenmesidir.

Bunun için depreme dayanıklı tasarım ile ilgili teorik kısa bilgiler çalışmanın ikinci bölümünde sunulmuştur. Ardından (TBDY 2018'in) tasarım ve performans felsefesi açıklanmıştır.

Üçüncü bölümde örnek olarak 24 katlı bir yüksek bina hakkında genel bilgiler verilmiş ve mevcut yönetmelikteki doğrusal analiz yöntemlerine göre bina taşıyıcı sistemi boyutlandırılmıştır. Örnek binanın doğrusal olmayan davranışı TBDY 2018, TS 500 ve ACI 318 standardına göre tasarım aşamaları uygulanarak göz önüne alınmıştır. Ayrıca bu kısımda performans analizi yapılması için doğrusal olmayan zaman tanım alanında analizler yapılmıştır. Elde edilen analiz sonuçları TBDY 2018' de öngörülen performans düzeyi sınır değerleri ile karşılaştırılmış ve deprem performans düzeyi belirlenmiştir.

Son bölümde binanın deprem performansı Kontrollü Hasar (KH) performans hedefi olarak değerlendirilmiştir. Bu bölümde bina DD-1 deprem yer hareketi için analiz edilmiş ve doğrusal olmayan elastik analiz yapılmıştır. İlk plastik mafsallık tüm elemanlar için atandı ve sonra bu performans hedefi için tüm eleman kesitleri için şekildeğiştirmeler hesaplanmıştır. Ardından, TBDY 2018 de verilen sınırlar değerlerle karşılaştırıldığında, bütün eleman kesitlerindeki şekildeğiştirmelerin sınırı değeri aşmadığı kontrol edilmiş, tüm elemanlarda sınırı değerlerin aşılmadığı tespit edilmiştir. Dolayısıyla tasarımı yapılan yüksek bina taşıyıcı sisteminin yeterli deprem performansına sahip olduğu belirlenmiştir. Sonuç olarak, mafsallarda oluşan plastik

dönmeler binanın güvenli olduğunu ve sınırlardan çok uzak olduğunu de göstermektedir. Buna göre, yüksek binanın hastane ve vb. amaçla kullanabileceği söylenebilir. Bu durum dayanım bazlı projelendirmenin önemini göstermektedir.

Çalışmanın son bölümünde analiz sonuçları yorumlanmış ve elde edilen sonuçlar TBDY 2018’ de verilen sınır değerleri ile karşılaştırılmış ve tartışılmıştır.





1. INTRODUCTION

In developed and developing countries, the need for high rise buildings is increasing day by day because of an increase in the human population and urbanization. Furthermore, the lack of enough area for people to build houses increased the demand of tall buildings. The rules and regulation which were made for the design of the building are for low and moderate-rise buildings. These regulations usually do not cover additional requirements for tall buildings. However, nowadays, as technology developed and engineers can have detailed information about ground motion, design rules etc. It enables engineers to design tall buildings. Some countries developed and published recommended design documents for high rise buildings, like Los Angeles LATBSDC Los Angeles Tall Buildings Structural Design Council (2005, 2008), San Francisco SEAONC- Structural Engineers Association of Northern California (2007) and CTBUH – Council on Tall building and Urban Habitat (2008). These design codes are all based on performance. In performance-based design, limited damage is allowed to occur in structural elements of a building, and that damage should be controllable or under the limits proposed and the ductile behavior of structural system is essential.

Till 2018 Turkey used DBYBHY 2017 code for the design of buildings, published in 2007. In 2018 Turkey developed a new code Turkish Building Seismic Code (TBSC 2018), which containing new chapters such as tall buildings, base isolation etc.

In Strength-based design philosophy, the inelastic behavior of the system is considered decreasing the elastic earthquake effects by a factor according to the ductility of the structural system. As a result, the coefficient that reduces the earthquake load is unrelated to plastic deformation and the distribution of the loads. With this method, real deformation of system cannot be estimated.

The damage caused by earthquake load is determined by the accurate deformation of the system. Because of this, the need for an analysis method to give us accurate

deformation is increased. Nowadays, inelastic behavior-based analysis is used to find accurate deformations developed by loads.

Developments in Earthquake Engineering and computer technologies bring facilities to perform an accurate analysis of buildings and find the real behavior of a building.

In this case study, structural design and performance evaluation of a 24 story tall building is accomplished according to TSBC 2018 and ACI 314 standards. For structural design of tall building according to TSBC 2018, first design parameters such as important factor, seismic design factor, building height factor, type of building, seismic load reduction factor, and other similar factors are found. For structural design of building strength based method (DGT) is used. Beams, columns and shear walls sections designed by ETABS software and then reinforcement detailing of sections carried out using AUTOCAD. According to TBSC 2018 code tall building structure is designed under DD-2 seismic level load. Different slab system assigned for the building and the appropriate one which is waffle slab system applied and designed. structural design of slab carried out using strip-based method in SAFE software and then it is reinforcement detailing accomplished in AUTOCAD software. At last seismic performance of building evaluated according to chapter 13 of TBSC 2018 standard. According to this standard, the performance of tall building should be evaluated for DD-4 (service seismic level load) and DD-1 (high seismic level load). The seismic performance of tall building first evaluated under DD-4 using liner modal analysis. The ratio of seismic load over capacity of structural element sections calculated and compared with the limits given in TBSC 2018 standard. All structural element sections should pass this IO (immediate occupancy) seismic performance level. In second, seismic performance of tall building evaluated under DD-1 using nonlinear time history analysis method. To accomplish this, 11x2=22 seismic ground motions selected from PEER NGA database and they scaled with DD-1 seismic level load using ETABS software. Hinges are defined for beams, columns and shear walls. Xtract software used to calculate the moment curvature and integration diagrams which is then used for hinge details of structural elements. Only horizontal ground motion is considered and near fault effect is also not considered. Plastic hinge rotations calculated for all structural elements of tall building and compared them with nonlinear time history analysis results taken out from ETABS software. All members should pass this performance level to do not collapse under large earthquakes and save lives.

2. EARTHQUAKE RESISTANT DESIGN PHILOSOPHY

2.1 Introduction

Engineers do not want to build completely Earthquake-resistant buildings; instead, they want to build Earthquake resistant buildings because completely earthquake-resisted building increases the cost and also an extensive element section will be taken out. Earthquake resistant design is to limit the damage of structural elements; even large ground motions happen.

Like other countries earthquake standards and codes, Turkey also developed standards for earthquake resistant designs. This standard design and performance sections aims that under the low-intensity earthquake there must not be any damage in structural and non-structural elements, under the medium intensity earthquake the damage in structural and non-structural elements should be limited and repairable, under the high level earthquake the damage in structural elements should be limited to do not collapse for the safety of people. According to this high level earthquake is an earthquake that its probability of exceedance should be 10% in 50 years. Moreover, some minimum performance limits are given in design codes for available building analysis and strengthening of them.

After the researches and analysis, the main causes of heavy damage and collapsing of buildings according to earthquakes happened in turkey in the past 30 years is like this (Journal of Selcuk Technic, 2018);

- a) The joint area of Beam-Column does not have required shear strength.
- b) Adherence failure
- c) Shear failure of beam joints
- d) Failure of columns according to axial force
- e) Shear cracking of shear walls

In 1994 the United States of America introduced the phenomena of Brittle failure after researching a steel tall building that was damaged by the Northridge earthquake. (FEMA -340,2000).

2.2 Design of Building Based on Earthquake Resistant Method

For a building to be designed according to Earthquake resistant method, there are two approaches;

- a) The structural system will be designed under the limit of elastic behavior or, in other words, do not let the loads pass the elastic capacity of element sections and do not let the plastic deformations happen. So the system deformation is elastic and will be under the limits. At the start, we can guess the elastic behavior of the system and its performance. Furthermore, precisely this approach is not economical, and we can apply it to only buildings with high importance factor.
- b) The other approach is non-linear behavior. In this approach, some elements of the structure will be allowed to pass the elastic limit and perform an allowable inelastic deformation. This approach gives us a desirable and economical result. First, we have to know about the capacity design principle because the inelastic design is based on this principle.

2.2.1 Basic principles of earthquake-resistant building design

To design an earthquake-proof building, engineers need to reinforce the structure and counteract an earthquake's forces. Since earthquakes release energy that pushes on a building from one direction, the strategy is to have the building push the opposite way. Here are some of the methods used to help buildings withstand earthquakes.

- a) The structural simplicity which helps the load path to transfer the seismic load to its foundation safely. On the contrary, the complicated load path makes the transfer of load hard and causes stress concentration.
- b) The structure should be symmetric and uniform because this helps the building to show a better seismic performance. The structure should be symmetrical in plan and elevation.

- c) The structure should be designed resistant to seismic loads in both directions. So that the system shows a bi-directional stiffness in both directions.
- d) To prevent lateral torsion, the mass and stiffness centers should be at the same coordination in a plan.
- e) The adequacy of Diaphragm at each story level should be maintained. It will prevent the considerable lateral movement of the vertical elements.
- f) Experience of uniform excitation during an earthquake in all structures, the foundation should be connected so well with a superstructure (Zekai Celep, 2007, Sixth National Conference on Earthquake Engineering).

2.2.2 Capacity design principle

The usage of a Capacity design principle for designing an earthquake-resistant concrete building is first used in the New Zealand Earthquake regulation by Prof. Pauley in 1970. And then it used with current regulations for designing concrete and steel buildings.

The objective of capacity design is to confirm a building undergoes controlled ductile behavior in order to avoid a collapse in a design-level earthquake. This involves designing the structure to allow a ductile failure at key predictable locations within the structure and to prevent other failure types occurring near these locations or elsewhere in the structure.

Capacity design principles let the engineers design a high strength column to increase the ductility of structure and to prevent floor mechanism and also to prevent shear failure in joints. There are some rules for buildings if it comes inside the circumstance of capacity design principle.

- a) The system should be designed such that, the joints that become plastic hinge after the load is applied should have the required ductility and shear failure must not happen at that joint.
- b) The joints which undergo plastic joint term after the seismic load should be designed such that it has enough ductility and shear capacity to prevent collapse.
- c) The system should be designed such that the elastoplastic deformations first must occur in beams or columns with high ductility performance. The moment-

curvature curve shows that a section with a lower axial load ratio has high ductility and vice versa.

- d) An essential principle of capacity design is avoiding soft story. To prevent this, columns in all stories should have enough strength and ductility.

In the design of beam and column sections should consider the rule that elastoplastic joint first must occur in beam rather than a column. Because if a plastic joint occurs first in beams, it helps the structure to show a ductile and better performance. The ductile performance of a structure will let the people escape and save their life.

- e) Incapacity design the characteristic of concrete, steel, and non-structural elements which increase the strength of the system should be considered in calculations. (Zekai Celep, 2007)

2.2.3 Performance-based design

The need for a new approach was formally recognized after the 1989 Loma Prieto Earthquake and became a nationally-funded effort after the 1994 Northridge Earthquake (Northridge). Those moderate earthquakes proved that the then-current design standards saved lives. However, the public declared on multiple fronts that the extent of damage was unexpected and too expensive to repair, and therefore unacceptable. (Chris D. Poland, 2019).

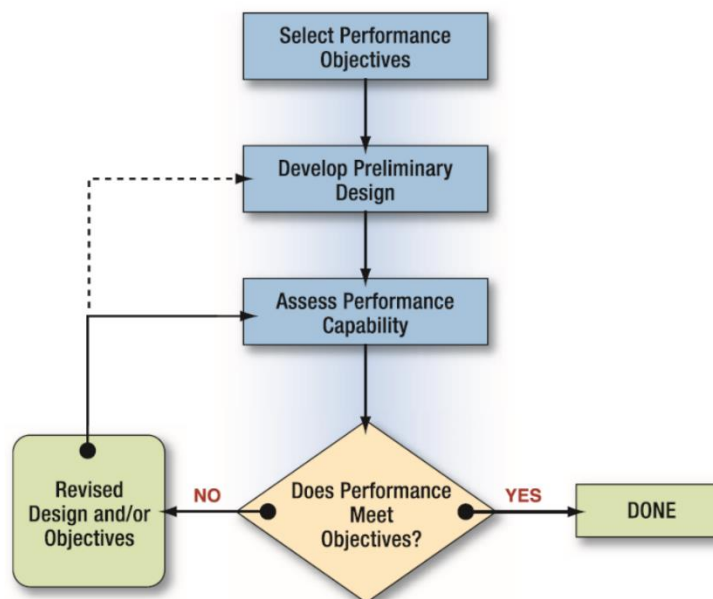


Figure 2.1 : Flowchart for performance selection in FEMA 749

Moving the profession from the prescriptive, non-committal, seismic design rules of the 1960s to PBD was, and continues to be, very controversial and contentious. Some believed that the simplicity of the existing codes was enough to guide the expert judgment of the designers. Others argued that there was too much uncertain in the entire Performance Based Design (PBD) process to declare the anticipated performance as we see in Figure 2.1 above. Protecting it from people was a way to hide under the building code's mandate to protect the public. Still, believes exist that while designing various performance levels was possible, the potential liability was too high. Many feared being sued out of business after the next major earthquake if expectations were not met. Fortunately, the earthquake design profession is protected by consensus-based codes and standards, including ASCE-41, that define the "state of the practice" the courts use to judge liability

2.2.4 Acceptable risk and performance level

In performance-based design, the design stage begins with selecting an acceptable risk and performance level for the system. The acceptable risk can be explained as maximum damage tolerated by the system, which comes from seismic hazards. The ICC PC categorized the level of damage into four performance levels. As we see in Table 2.2 and Table 2.3;

- a) Mild Impact: in this performance level, there is no damage in a structure, and non-structural elements should be fully functional. The injuries will be minor, and the building is safe to occupy.
- b) Moderate Impact: repairable structural damage is allowed to happen, and the building occupancy will be delayed for a short period. The non-structural elements and emergency systems remain fully functional. The probability of life loss is so low.
- c) High Impact: at this stage, the damage to the structure of the building is significant but in repairable level. Non-structural elements are non-functional. Injuries are moderate, but life loss is moderate. The building occupancy will be delayed for a significant period.

Table 2.1 : Performance level in increasing level in FEMA.

		INCREASING LEVEL OF PERFORMANCE			
		Performance Groups			
MAGNITUDE OF DESIGN EVENT Increasing Magnitude of Event	Very Large (Very rare)	Performance Group I Severe	Performance Group II Severe	Performance Group III High	Performance Group IV Moderate
	Large (Rare)	Severe	High	Moderate	Mild
	Medium (Less Frequent)	High	Moderate	Mild	Mild
	Small (Frequent)	Moderate	Mild	Mild	Mild

- d) Severe Impact: in this level, the structure is exposed to a high damage level, which is not repairable. The non-structural and emergency systems are entirely non-functional. The building re-occupancy is not allowed because it may cause collapse.

Today performance-based analysis is used only for analysis for available buildings performance. However, fortunately soon, it will be applied for the design of new buildings. Then investors and structural engineers will work together to design a building. The investor will explain their claim at the design stage.

2.3 High Rise Building Design

A building is defined as high-rise when it is considerably higher than the surrounding buildings, or its proportion is slender enough to give the appearance of a tall building. The construction of high-rise buildings started at the end of the 19th century in Chicago, with the evolution. It was made possible because of new inventions such as the safety elevator in 1853 and the telephone in 1876, which enabled the transport of building materials and the ability to communicate to higher levels. Besides, the building materials changed as they went from wood and masonry to using steel frames with lighter masonry walls. Earlier buildings that were built with heavy masonry walls were limited to certain heights by its self-weight. With steel frames, the masonry could

be thinner and act only as a facade for weather protection, and taller buildings could be constructed. (Hasgür and Gündüz, 1996)

During the industrial revolution in Europe, the need for warehouses, factories, and multi-story buildings were huge. Europe also played a significant role in developing new materials such as glass, reinforced concrete, and steel. Before 1945 the high –rise buildings in Europe were few and below the 100-meter limit, and it was not until after the Second World War the construction of high-rise buildings excelled. It had to do with the reconstruction of all destroyed cities and the expanded demand for offices and residential.

In the past, a five-story building was called high rise building; however, nowadays, Burj Dubai with 818 m height is called a high rise building.

In IYBDY regulation, a building with a height of 60 m and above is called a tall building.

The improvement in technology and machinery, producing high strength concrete and steel, new design methods all together helped the construction of tall buildings. Now we will talk a little bit about Istanbul High Rise Earthquake Resistant Buildings Regulation.

Table 2.2 : Structural performance level and range in FEMA 749.

Nonstructural Performance Levels	Structural Performance Levels and Ranges					
	S-1 Immediate Occupancy	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention	S-6 Not Considered
N-A Operational	Operational 1-A	2-A	Not Recommended	Not Recommended	Not Recommended	Not Recommended
N-B Immediate Occupancy	Immediate Occupancy 1-B	2-B	3-B	Not Recommended	Not Recommended	Not Recommended
N-C Life Safety	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
N-D Hazards Reduced	Not Recommended	2-D	3-D	4-D	5-D	6-D
N-E Not Considered	Not Recommended	Not Recommended	Not Recommended	4-E	Collapse Prevention 5-E	No Rehabilitation

2.3.1 Istanbul high rise earthquake-resistant building code

In this regulation, definition to high rise or tall building is; a building that has a height of greater than 60 meters, or the total number of stories without basement stories must be 20 or more.

IYBDY earthquake regulation is based on the performance of the system. It also allows the elastic analysis methods for strength-based design; however, the damage of the system should be limited to the desired performance of the system. The DBYBHY regulation and codes mostly use the elastic analysis method.

The performance-based design of a building, which is designed according to IYBDY, is controlled by independent company staff who has the knowledge of theoretical and practical in this field.

2.3.2 Definition of earthquake effect

Earthquake Levels

Earthquake levels in Istanbul high rise building earthquake-resistant regulation are defined such as;

a) D1 Earthquake Level

According to this regulation, a D1 earthquake level is an earthquake, which has a high occurrence probability but low intensity. This earthquake often occurs in the whole life cycle of a building. It is an earthquake that its occurrence probability is %50 in 50 years, and its return period is 72 years.

b) D2 Earthquake Level

According to this regulation, a D2 earthquake level is an earthquake, which has a low occurrence probability but high intensity. This earthquake rarely occurs in the whole life cycle of a building. It is an earthquake that its occurrence probability is %10 in 50 years, and its return period is 475 years.

c) D3 Earthquake Level

According to this regulation, a D3 earthquake level is an earthquake, which has a too low occurrence probability but so high intensity. This earthquake occurs so rarely in the whole life cycle of a building. It is an earthquake that its occurrence probability

is %2 in 50 years, and its return period is 2475 years. Its influence is so high and destroyable on tall buildings.

2.3.3 Earthquake design spectrum

In DBYBHY, the Design Spectrum is defined with five sections. However, in IYBDY and Turkish New Codes for Earthquake, which is published in 2018, it is defined with five sections. Elastic design spectrum curve is given in Figure 2.4 below.

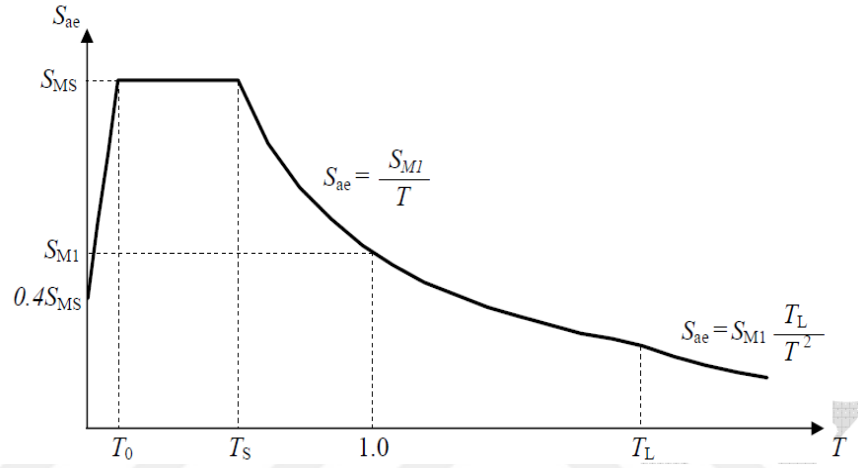


Figure 2.2 : Elastic design spectrum curve according to IYBDY

S_1 : Spectral Acceleration of 1 sec Period

S_{ms} : Spectral Acceleration of short period

T : Natural seismic period

T_0 : Period

T_L : Long Period

T_s : Short Period

In DBYBHY Spectral Acceleration is given as constant in tables. Nevertheless, in IYBDY, its found with these formulas. This formula is dependent on the natural ground period and ground effects (F_a , F_v).

For Istanbul Province, $T_L = 12$ sec is defined as an extended period. In DBYBHY, T_a and T_b are given according to ground type. The ground types are A, B, C, D, E, F in IYBDY, however it is classified in Z1, Z2, Z3, Z4 types. Formulas are given in equation (2.1) and (2.2).

$$\begin{aligned} S_{MS} &= F_a \times S_s \\ S_{M1} &= F_v \times S_1 \end{aligned} \quad (2.1)$$

$$T_s = \frac{S_{M1}}{S_{MS}} \quad ; \quad T_o = 0.2T_s \quad (2.2)$$

2.3.4 Time history analysis of earthquake

At least seven seismic ground motion is used for time history analysis. If there is not available any real earthquake record, an artificial seismic motion is created by using the similarities and design spectrum. For creating every spectrum, a real recorded seismic motion is used.

For Istanbul province to select recorded acceleration of ground, acceleration with the magnitude of between 7.0 – 7.5 and ground type of B, C is used. Furthermore, for the distance of the earthquake, the shortest distance from the Marmara fault is considered which is given in Figure 2.3.

The artificial seismic ground motion should be based on a physical model. For considering the direction effect, when the distance between building and Marmara fault is less than 15 Km, then three seismic ground motion is used for creating the spectrum.

For the T_0 period, the calculated average spectral amplitude, which is calculated from a group of ground motions, should not be less than the amplitude of the design spectrum at T_0 .

For every seismic recorded ground motion, the period between $\pm 0.05g$ should be five times of first natural vibration period of the building and should not be less than 15 sec.

2.3.5 Tall buildings performance levels

a) Mild Damage (MH-KK)

It is a performance level in which no damage happens on structural and non-structural elements of a structure, or the damage is at a level that can be repaired easily.

c) Controlled Damage (KH-CG)

In this performance level, a moderate level of damage is allowed to happen on structural and non-structural elements in a building. Furthermore, the damage is limited and can be repaired.

d) High damage (IH-GG)

It is a performance level that serious damage happens to the building structure. The structure will be no more usable because it may cause collapse. However, before collapsing, it will give enough time for people to leave the building.

2.4 Analysis and Design Codes of Tall Building

2.4.1 Analysis procedure for tall buildings

We have two ways of analysis for tall buildings within this regulation, Linear elastic and non-linear analysis. For linear elastic analysis of a system, a modal analysis method is used. In this method, the damping ratio is given %5, and the P- Δ effect is

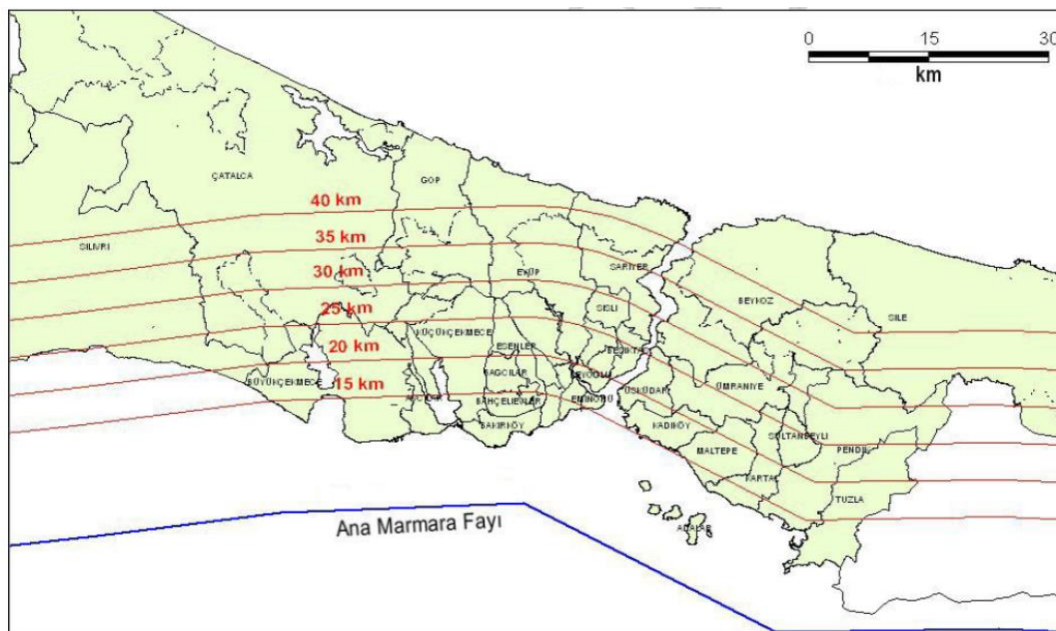


Figure 2.3 : Main Marmara fault lines for Istanbul area

considered in the rigidity matrix of the system.

For non-linear analysis of a system, time history analysis is used. Like linear analysis, in non-linear analysis, the damping ratio is %5, and the P-Δ effect is considered in the rigidity matrix.

For non-linear analysis, a group of 7 ground motion is used to prepare the time history analysis spectrum. Then this spectrum is applied on two main coordinate axis directions of the system individually. First, it is applied in one direction, and then the analysis is run. After this, the spectrum is rotated 90 degrees for the next direction then analysis runs for this direction. The results obtained from these analyses are then averaged.

In non-linear dynamic analysis, the live load contribution factor, n , is taken from table 2.7 of DBYBHY 2007. However, in any case, it is not taken less than 0.3. N for several stories can be calculated using equation (2.3) given below.

$$\begin{array}{ll} n = 0.01(50 - N) & N \leq 40 \\ n = 0.10 & N > 40 \end{array} \quad (2.3)$$

2.4.2 Rules and regulations for analysis model

For elements of a frame, inelastic analysis, they are all modelled as finite rigid bars. However, in non-linear analysis, these are all modelled as finite plastic bars, and their plasticity information is taken into the system.

If we come for another important structural model, which is shear walls, it is modelled as an inelastic shell analysis. However, this is modelled as a fiber element with limited plastic capacity. The regulations which are given in DBYBHY 2007 for shear walls section is not necessary for tall building shear walls. Moreover, the shear rigidity of a shear wall should be considered appropriately. For confined concrete and rebar, the appendix 7B of DBYBHY 2007 can be used for modelling. If a concrete with a strength of greater than C50 is used for tall buildings, the explicit permission of authorities is needed.

For the preliminary design of concrete sections, rebar, and steel, their material strength is divided by material safety factor (f_k). Nevertheless, in the design stage, the material strength is divided by the mean expected strength factor (f_e).

For

Concrete: $f_{ce}=1.3 f_{ck}$

Rebar: $f_{ye}=1.17 f_{yk}$

Steel (S235): $f_{ye}=1.5 f_{yk}$

Steel (S275): $f_{ye}=1.3 f_{yk}$

Steel (S355): $f_{ye}=1.1 f_{yk}$

For foundation and basement, soil specification and rigidity are calculated by the idealized method as specified by authorities.

2.6 Turkey New Earthquake Code for Buildings (TBDY)

This new regulation of turkey is the updated version of DBYBHY 2007. This is prepared for the design of new buildings under the earthquake and strengthening of the existing building against earthquakes. It is published in the formal magazine of Turkey with an act of 2018/11275 in 22/01/2018. After a long period of researches, this code is updated and published. There are so many important differences in TBDY. There are some changes which are mentioned below:

- a) Some differences in Building Importance factor
- b) Building classification according to usage
- c) Building classification according to height
- d) Effective section rigidity factor
- e) Minimum concrete strength C25
- f) Minimum column section 30cmX30cm
- g) Earthquake Hazard Map (Türkiye Deprem Haritasi) usage instead of Earthquake Regional Map (Deprem Bölge Haritasi)
- h) The different methods in finding the Design Acceleration Spectrum.
- i) Concrete strength greater than C80 is not included.
- j) The relative story displacement checking method is different.

The data needed for any region is taken from the Earthquake Hazard Map by entering the longitude and attitude of that region. Earthquake Hazard Map can be reached from the AFAD site.

2.6.1 Standard seismic ground motion spectrum

The typical earthquake ground motion response spectrum represents an envelope of the peak responses of many single-degree-of-freedom (SDOF) systems with different periods. The acceleration response spectrum of ground motion is a relationship between the natural period of vibration of an SDOF system and the maximum absolute acceleration that it experiences under the ground motion. Similarly, a displacement response spectrum typically represents the peak displacement, relative to the ground, of many SDOF systems with different periods. Hence, the construction of a response spectrum involves the analysis of many different SDOF systems. The value of each point on the spectrum is the peak response of a single degree of freedom system of a given period. This spectrum is constructed from 5 significant points. These points are calculated with the help of formulas given in TBDY. All of the formulas are related to S_s and S_1 as given in equation (2.4). These can be found from the Turkey Earthquake Hazard map by entering the latitude and longitude of building.

- a) Short period Spectral Acceleration factor
- b) 1.0 sec period Spectral Acceleration factor

$$S_{DS} = S_s \times F_s \quad S_{D1} = S_1 \times F_1 \quad (2.4)$$

F_s and F_1 are local ground effect factors that are taken from the Table 2.3 and Table 2.4 below.

Table 2.3 : F_s table is given in TBDY 2018

Soil Classification	Fs factor for short period area S_s					
	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \geq 1.5$
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 2.4 : F_1 table is given in TBDY 2018

Soil Classification	F1 factor for short period area 1.0					
	$S_s \leq 0.1$	$S_s = 0.2$	$S_s = 0.3$	$S_s = 0.4$	$S_s = 0.5$	$S_s \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Specific site soil behavior analysis should be done.					

Elastic design spectrum curve is given in Figure 2-4 according to TBDY 2018. And can be calculated using equations (2.5) up to (2.10) respectively.

$$S_{ae}(T) = \left[0.4 + 0.6 \times \frac{T}{T_A} \right] S_{DS} \quad 0 \leq T \leq T_A \quad (2.5)$$

$$S_{ae}(T) = S_{DS} \quad T_A \leq T \leq T_B \quad (2.6)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \quad T_B \leq T \leq T_L \quad (2.7)$$

$$S_{ae}(T) = \frac{S_{D1} T_L}{T^2} \quad T_L \leq T \quad (2.8)$$

$$T_A = 0.2 \frac{S_{D1}}{S_{DS}} \quad (2.9)$$

$$T_B = \frac{S_{D1}}{S_{DS}} \quad T_L = 6 \text{ s} \quad (2.10)$$

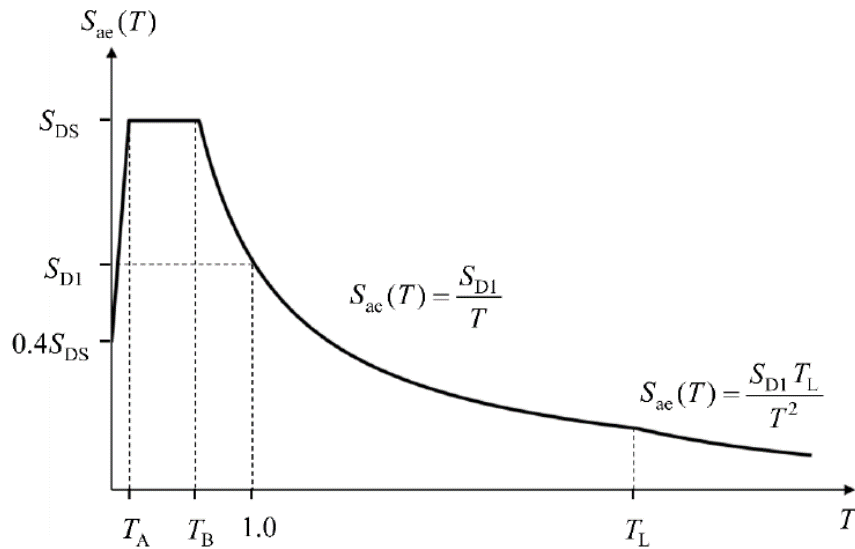


Figure 2.4 : Elastic design spectrum curve given in TBDY 2018

If one wants to do a linear elastic analysis of a structure under full earthquake loading and calculate forces acting on the structure, the resultant forces will be significantly large, and the design of the structure will be uneconomical and not feasible at all. Under such conditions, we are not taking advantage of the fundamental concept of ductility and inelastic energy dissipation of materials. Any structure is permanently loaded under dead and live loads. To make sure that the structure does not crack significantly under these gravity loads, we make sure that static demand is always less than the yield capacity of the members. We make sure that the deflection criteria are always satisfied, and floor vibrations are not exceeding certain limits. Nevertheless, earthquake loading is not something that is applied to a building permanently. Because a building is designed to make sure that it does not collapse under such an extreme event, we can let the building crack and let it move past the yield deformation. The push over curve for elastic and perfectly elastic displacement is given in Figure 2.5 and Figure 2.10 below. Designing a structure that is supposed to stay elastic under these seismic demands, which are generally significant, does not make sense. After all, what is the probability that the building will even see the seismic loading that we just designed for. The life expectancy of residential and office buildings is 100 years max, and an earthquake with significant shaking has a probability of occurring every 500 years (in most regions).

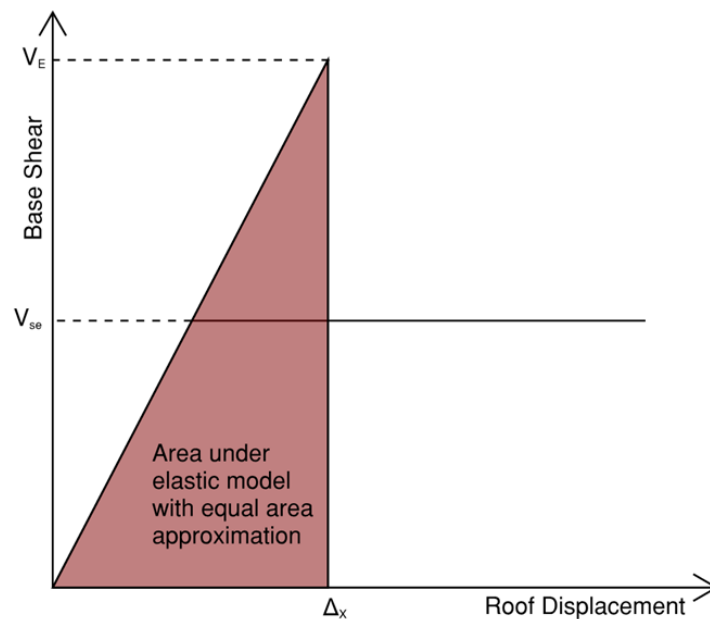


Figure 2.5 : Push over curve of elastic-perfectly plastic EPP

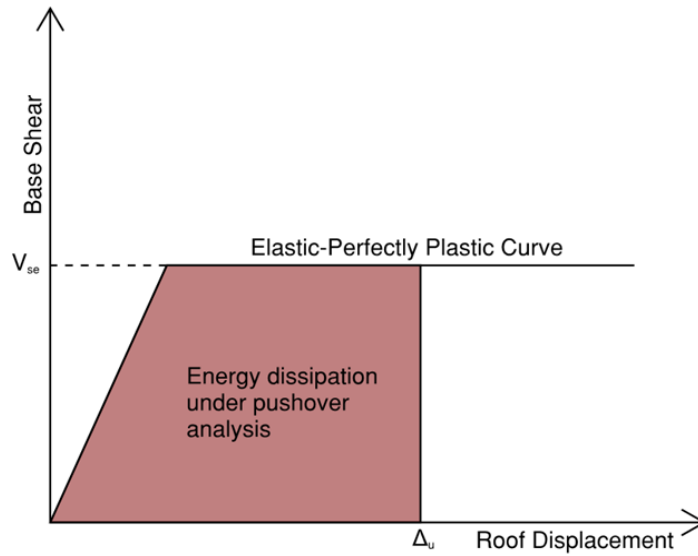


Figure 2.6 : Push over curve of elastic displacement

There are few other regions where the frequency of earthquakes is as small as 40 years (FEMA 749, 2000).

Moreover, places like Mexico and Taiwan are experiencing significant shaking every 20 years or so. If we look at the second image talking about a linear elastic model, we can see that because it does not go in a plastic state, to capture the energy the same as the inelastic model, the base shear must increase, that means the external force increases as the displacement increases. (Remember in linear analysis, force is always proportional to displacement). At a special force and displacement value, the area under the curve in this elastic model reaches the same as the area under the EPP curve. The base shear corresponding to this value is called V_e and the base shear at which the structure yields is called V_{se} .

R factor is calculated using equations (2.11), (2.12) and (2.13) given below.

$$R = \frac{V_e}{V_{se}} \quad (2.11)$$

$$R_a = \frac{R}{I} \quad T > T_B \quad (2.12)$$

$$R_a(T) = D + \left[\frac{R}{I} - D \right] \frac{T}{T_B} \quad T \leq T_B \quad (2.13)$$

Table 2.5 : Importance factor table is given in TBDY 2018

Building Usage Classification	Building Usage Purpose	Building Importance Factor
BKS = 1	A) Places which will be used immediately after earthquake. Like Hospitals, fire fighting building, PTT, Power Station, rescue stations, police stations, communication, operation centers, structures containing high toxic materials. B) Schools, dormitories, jails, military buildings. C) Museums D) toxic, blasting material containing buildings	1.5
BKS = 2	Shopping Centers, Sport Complexes, Cinema, Theatre	1.2
BKS = 3	Building which are not included in first and second categories like residence buildings, offices, hotels, industrial buildings	1.0

2.6.2 Building usage classification and importance factors

Building usage classification is a factor, which is needed for finding Earthquake Design classification (DTS). It shows the purpose of usage of the building, and it can be found from table 3.1 TBDY 2018 and given in Table 2.5 above. It is denoted (BKS).

Building an Importance factor is a factor that shows how much building is essential to be safe after an earthquake. For this reason, a building which is having high importance factor will face less damage. It is denoted (I) and can be found from table 3.1 TBDY 2018.

2.6.3 Seismic design category

It is a factor that is needed to find the building height range. If we want to build a building, every region has its height limit, which can be found from Table 2.6 given below.

To find Allowable Building Height for a region, we need to find first Seismic Design Classification (DTS). It can be found from Table 2.7 below with the help of S_{DS} and BKS.

Table 2.6 : Building usage classification (BKS) given in TBDY 2018

Acceleration Design Spectrum Factor (S_{DS}) for DD-2 Earthquake level and short period	Building Usage Classification	
	BKS = 1	BKS = 2, 3
$S_{DS} < 0.33$	DTS 4a	DTS = 4
$0.33 \leq S_{DS} < 0.5$	DTS = 3a	DTS = 3
$0.5 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

Table 2.7 : Building height classification (BYS) given in TBDY 2018

Building Height Category	Building Allowable Height According to Building Height Category and Seismic Design Category (m)		
	DTS= 1, 1a, 2, 2a	3, 3a	4, 4a
BYS=1	$H_N > 70$	$H_N \leq 91$	$H_N \leq 105$
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	$H_N \leq 7$	$H_N \leq 10.5$	

2.6.4 Irregularity in buildings

There are two types of irregularities in building

2.6.4.1 Horizontal irregularity

a) A1 Torsion irregularity

Under earthquake motion in any direction, in any story, if the ratio of maximum relative story displacement over average relative displacement is greater than 1.2 as given in equation (2.14), then the building has A1 torsion irregularity. For finding relative

displacement, a $\pm 5\%$ eccentricity should be considered. Calculations given in table A. 5 and A. 6 of Appendix A.

$$\eta_{bi} = (\Delta_i^{(X)})_{max} / (\Delta_i^{(X)})_{avg} > 1.2 \quad (2.14)$$

b) A2 Slab Discontinuity

If the sum of all discontinuity areas on a floor is greater than 1/3 of the gross floor area, then the building has an A2 irregularity.

If there is a sudden change in rigidity and strength of the slab, this case also has an A2 irregularity.

c) A3 Projection in Plan

In any direction, in any story, if the length of the projection part is greater than 20% of the total length of the slab in the same direction.

2.6.4.2 Vertical irregularity

a) B1 Weak Story irregularity

Under the earthquake load in any direction, if the ratio of the total effective shear area of any story over the total effective shear area of its adjacent story is less than 0.8, then the building has B1 irregularity. Formulas given equations (2.15) and (2.16).

$$[\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} < 0.8] \quad (2.15)$$

$$(\sum A_e)_i = (\sum A_w)_i + (\sum A_g)_i + (0.15 \sum A_k)_i \quad (2.16)$$

b) B2 Rigidity Irregularity (Soft Story)

In a building, if the ratio of relative displacement of any story over the relative displacement of its adjacent story is greater than 2.0, then the building has a soft story irregularity. Calculations given in table A. 7 and A. 8 of Appendix A.

c) B3 Discontinuity in Vertical Elements of Structure

If there is a discontinuity in columns or shear walls of a structure, or maybe upper shear walls set on lower columns, then there will be B3 irregularity.

2.6.5 Relative story displacement calculation and controlling

The difference between the displacement of a story and its lower story is called Relative Story displacement. For example, if a story displaces 2 cm and its lower story moves 1cm, then the Relative story displacement of the top story is $2-1 = 1$ cm.

In TBDY 2018, relative story displacement controlling is defined in a different method than DBYBHY 2007 code. In DBYBDY 2007, relative story displacement over the height of the story must be less than or equal to 0.02. However, TBDY 2018 is defined as the equation below. In this equation, λ is a parameter which is defined as; the ratio of DD-3 acceleration spectrum over DD-2 acceleration Spectrum. The limit is defined in two ways:

- a) If there is flexible material between wall and columns or shear walls, then the limit is 0.008κ
- b) If there is not any flexible material between wall and columns or shear wall, then the limit is 0.016κ .

All calculations given in table A.9 and table A.10 of Appendix A and calculations done using equations (2.17) till (2.20) given below.

$$\Delta i = u_i - u_{i-1} \quad (2.17)$$

$$\delta i = \frac{R}{I} \Delta i \quad (2.18)$$

$$\lambda * \frac{\delta i (max)}{h_i} \leq 0.008 \kappa \quad \text{for condition (a)} \quad (2.19)$$

$$\lambda * \frac{\delta i (max)}{h_i} \leq 0.016 \kappa \quad \text{for condition (b)} \quad (2.20)$$

This κ is a constant, which is 1 for Concrete Structure and 0.5 for Steel Structure.

2.6.6 P-Δ effect controlling

A structure is composed of structural and non-structural elements. To limit the damage that can affect non-structural elements, we should check the P-Δ effect and compare it with limits. The limit is given in TBDY 2018 code, as we see in equation (2.21).

$$\theta_{II,i} = \frac{\Delta i (avg) * \sum_{k=i}^N W_k}{V_i * h_i} \quad (2.21)$$

After calculating the P- Δ effect, if the maximum $\theta_{II,i}$ magnitude be less than the equation below, then we will not consider the P- Δ effect in our calculation. In equation (2.22), C_h is a constant, which is 0.5 for concrete and 1 for steel and composite structures. Calculations done for P- Δ effect is given in Table A.11 and A. 12.

$$\theta_{II,max} \leq 0.12 \frac{D}{C_h * R} \quad (2.22)$$

In all stories if $\theta_{II,max}$ value be greater than the equation above, then we have to consider the P- Δ effect in our calculation just by multiplying all internal loads with the factor β_{II} as we see in equation (2.23). Moreover, this factor is calculated with the equation written below.

$$\beta_{II} = 0.88 + \frac{C_h R}{D} \theta_{II,max} \geq 1 \quad (2.23)$$

2.6.7 Building performance level

For the performance of building under seismic and gravity load, we have four performance level, which is as follows:

1. Immediate Occupancy performance level (KK, IO)

In this performance level, the structural elements of the building will not go under any damage, or the damage will be in an ignorable level.

2. Limited Damage performance level (SH)

In this performance level, the structural elements will go under some damage but at a limited level that will not cross the nonlinear behavior of the structure.

3. Controlled Damage performance level (KH, LS)

In this performance level, some damage will occur in the structural elements of the system, but that will be under controlled and life safety is maintained at this performance level.

4. Collapse Prevention Performance level (CP, GO)

In this performance level, considerable damage will happen to the structural elements of the building, and the building may collapse in aftershocks but will not collapse and will let the people escape and save their life.

In the Turkish New Building Code 2018, there are tables for target performance. These tables are related to the type of building and Seismic Design Category. The tables below are for new cast in site-building, the existing building, and Prestressed building. There are normal performance levels for DTS of 1, 2, 3, 3a, 4, 4a, and for other performance levels for DTS of 1a, 2a. its related to the owner of building that which performance level he wants from building under seismic load. Performance level selection is given in Table 2.8 below.

Table 2.8 : Performance level table in TBDY 2018 for tall and existing buildings.

b) New designed or available High Rise buildings (BYS=1)				
Seismic ground motion level	Seismic Design Category 1, 1a, 2, 2a, 3, 3a 4, 4a		Seismic Design Category 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)

2.6.8 Combination of seismic load with other loads

After we find the horizontal acceleration spectrum for a defined coordinate dividing it by seismic load reduction factor, we can find the elastic reduced acceleration spectrum for our building.

Then to have the seismic effect of one direction to another side, we have to add 30% of seismic effect to the other side. There are two main seismic directions, which is perpendicular to each other, and they are X and Y seismic directions.

$$S_{aR}(T) = \frac{S_{ae}}{R_a(T)} \quad (2.24)$$

$$E_d^H = \pm E_d^X \pm 0.3 E_d^Y \quad (2.25)$$

$$E_d^H = \pm 0.3 E_d^X \pm E_d^Y \quad (2.26)$$

In equation (2.25) and (2.26), E_d^H is the required horizontal reduced design acceleration spectrum, which can then be combined with other loads.

With the help of equations (2.27) and (2.28), seismic loads with other gravity and live loads can be added.

$$G + Q + 0.2S + E_d^H \quad (2.27)$$

$$0.9G + H + E_d^H \quad (2.28)$$

In here

$Q = \text{Live Load}$

$G = \text{Dead Load}$

$S = \text{Snow load}$

$E_d^H = \text{Horizontal Seismic Load}$

We have to add vertical seismic load also with these loads load if there is

- a) Beam with a span of larger or equal to 20 m
- b) A cantilever with a span of larger or equal to 5 m
- c) If columns are located on a beam directly
- d) If there is an inclined column in the building

Otherwise, we ignore the effect of vertical seismic load in the combinations of loads.

2.6.9 Concrete structural elements effective rigidity factor

For the strength-based design of buildings, there is some factor given for elements to be applied. These factors will decrease the rigidity of element sections. For a different type of system, there is a factor given, and for design building, we select according to the type of system. Rigidity factor table is given in Table 2.9 below.

For considering the live load in design and analysis of the building, we need to use this below table to take the factor of the live load participation factor. For Depos, the factor is 0.8, and for residential building and office and hospital live load factor is 0.3 and denoted by n. n factors are given in Table 2.10 below.

Table 2.9 : Live load participation factor in TBDY 2018

Building Usage Purpose	n
Store, Depo, Warehouse	0.8
School, Dormitories, Sport Complex, Cinema, Theatre, Concert Places, Religious Places, shops and etc	0.6
Residence, Office, Hotel, Hospital, Parking etc	0.3

Table 2.10 : Rigidity factors for Strength based design (DGT)

Concrete Structure System Elements	Effective Section rigidity Factor	
Shear Wall - Slab (In Plane)	Axial	Shear
Shear Wall	0.5	0.5
Basement Shear Wall	0.8	0.5
Slab	0.25	0.25
Shear Wall - Slab (Out of Plane)	Flexural	Shear
Shear Wall	0.25	1.00
Basement Shear Wall	0.5	1.00
Slab	0.25	1.00
Frame Element	Flexural	Shear
Spanderal Beam	0.15	1.00
Frame Beam	0.35	1.00
Frame Column	0.7	1.00
Shear wall	0.5	0.5

2.6.10 Strength-based design

This method gives the procedure to be used for design analysis of the structure by considering the ultimate strength. Designing the structure based on strength and serviceability constraints play a pivotal role in safe construction and ensures the occupants with maximum comfort. The design analysis of the structure fulfills the following requirements. The strength design method considers the additional strength in the design analysis. This additional strength develops in the material due to strain hardening. This additional strength was not considered in other design analysis methods for designing the reinforced concrete bar. The designers can give more steel

with the same area of concrete section and larger concrete area with reduced steel reinforcing when compared to design in allowable stress method.

2.6.11 High ductile columns

In this code for rectangular column section minimum, allowable dimension is 300mm, and for circular section, columns are 350 mm.

After the multiplication of live load with reduction live load factor and combining them with other loads, the ratio of axial compression load derived from G+Q+E over section gross area and concrete strength should be less than or equal to 0.4 as we see in equation (2.29).

$$A_c \geq \frac{N_{dm}}{0.4 * f_{ck}} \quad (2.29)$$

The longitudinal reinforcement ratio in the high ductile column should be 1% minimum and 4% in maximum value. For rectangular sections, we are not allowed to use rebar less than Ø14, and for a circular section, less than six rebar are not allowed. In spliced sections, the ratio of rebar should not be larger than 6%.

Splicing length should be greater than or equal to l_b . Moreover, along the splicing length, the stirrup or tie spacing should be a minimum of 1/3 small column dimension of 150mm. anchorage limits in joints are given in Figure 2-7.

For transverse reinforcement ties with diameter less than Ø8 is not allowed to use and for support section of columns spacing between ties should be like this:

$$\begin{aligned} s &\geq 50mm \\ s &\leq 150mm \\ s &\leq b_{min}/3 \end{aligned} \quad \begin{aligned} A_{sh} &\geq 0.30 s b_k [(A_c / A_{ck}) - 1] (f_{ck} / f_{ywk}) \\ A_{sh} &\geq 0.075 s b_k (f_{ck} / f_{ywk}) \end{aligned} \quad (2.30)$$

For rectangular column section if $N_d > 0.20 A_c f_{ck}$ then the transverse reinforcement area should be calculated using equation (2.30).

2.6.12 Columns stronger than beams condition

For a structural system with beams, columns, and shear walls, in connection point of beam and column, the total moment capacity of columns should be at least 20% greater than the total moment capacity of the beam as we see in equation (2.31). Then

according to this condition, first plastic hinges occur in beams rather than in columns so that it will prevent the collapse of the structure.

$$(M_{ra} + M_{ri}) \geq 1.2 (M_{ri} + M_{ij}) \quad (2.31)$$

2.6.13 Column shear safety

In column shear force V_e which is used for shear design of column can be calculated using equation (2.32) below.

$$V_e = \frac{M_a + M_u}{l_n} \quad (2.32)$$

If V_r , which is calculated by a combination of vertical loads and multiplying it by D , is less than V_e , then we use V_r as a shear force in design instead of V_e .

If the formula above satisfy then joint moment capacity is calculated using equation (2.33).

$$\Sigma M_p = M_{pi} + M_{pj} \quad (2.33)$$

If specific calculation not done for the system then $M_{pi} = 1.4M_{ri}$ and $M_{pj} = 1.4M_{rj}$ can be taken into consideration. These values are given in Figure 2.8.

If the formula above did not satisfy then in joint of beams and columns, the moments in column support is taken as moment capacity of column support in calculations as given in equation (2.34).

$$. M_{pa} = 1.4M_{ra} \text{ and } M_{pu} = 1.4M_{ru} \quad (2.34)$$

Furthermore, for columns, always the equations (2.35) and (2.36) should satisfy.

$$V_e \leq V_r \quad (2.35)$$

$$V_e \leq 0.85A_w\sqrt{f_{ck}} \quad (2.36)$$

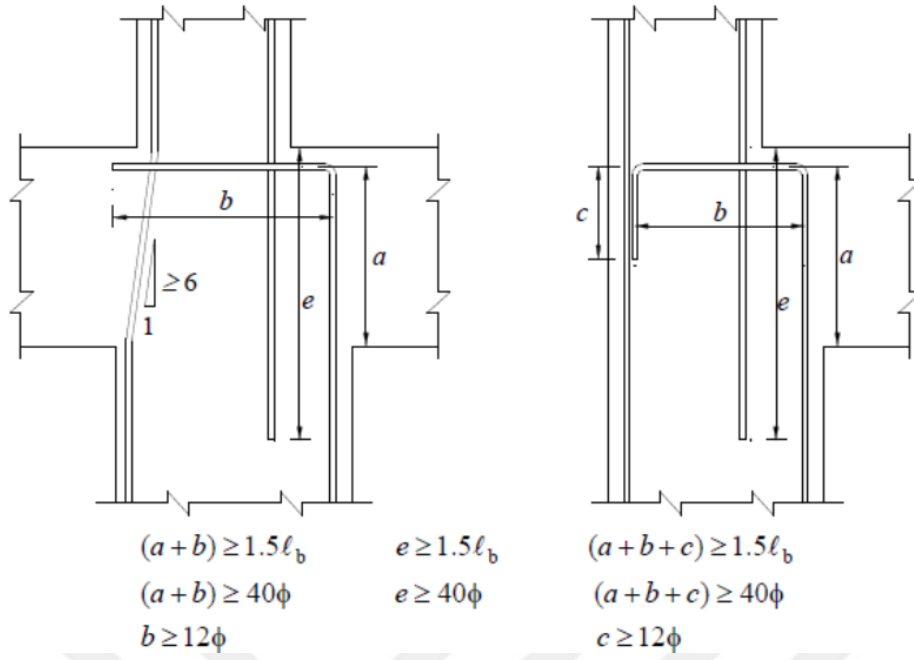


Figure 2.7 : Columns rebar anchorage at joints

Kat No.	M_j 'nin hesaplanması		M_e 'nin hesaplanması	
	Kolon üst ucunda Denk. 7.3'ün sağlaması durumu	Kolon üst ucunda Denk. 7.3'ün sağlamaması durumu	Kolon alt ucunda Denk. 7.3'ün sağlamaması durumu	Kolon alt ucunda Denk. 7.3'ün sağlaması durumu
i+1				
i				
i-1				
	$\Sigma M_p = M_{pi} + M_{pj}$ $M_u = \frac{M_{hu(i)}}{M_{hu(i)} + M_{ha(i+1)}} \Sigma M_p$			
	$\Sigma M_p = M_{pi} + M_{pj}$ $M_u = \frac{M_{ha(i)}}{M_{ha(i)} + M_{hu(i+1)}} \Sigma M_p$			
	$M_{hu(i)}$ i'inci kat kolonu üst ucunda Bölüm 3 ve Bölüm 4'e göre bulunan moment $M_{ha(i)}$ i'inci kat kolonu alt ucunda Bölüm 3 ve Bölüm 4'e göre bulunan moment			

Figure 2.8 : Column edges moment capacity defining formulas

2.6.14 High ductile beams

Beams with columns that make a structural frame should be like this. The width of the beam must be 250 mm or greater than it. Beams height in the section must be greater than three times the slab thickness of 300 mm. And also, the height of the beam must not be greater than 3.5 times its width.

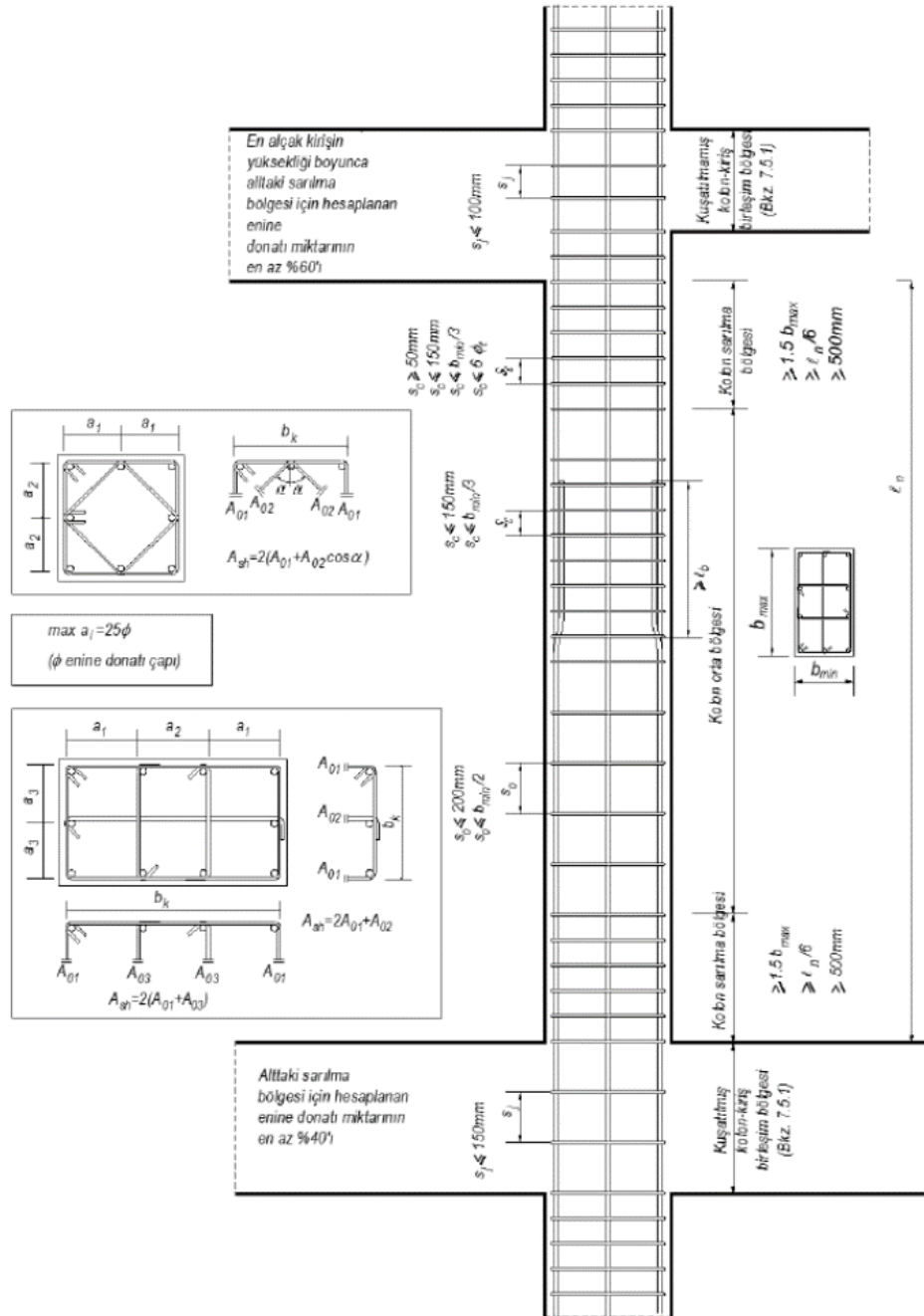


Figure 2.9 : Column longitudinal and transvers reinforcement detailing

Transverse reinforcement diameter must not be less than 12 mm, and spacing must be less than 300mm. Furthermore, extra ties should be used for the entire beam with spacing less than 600mm in height and 400mm in the axis. Column detailing is given in Figure 2.9 above.

In beams, the minimum ratio of longitudinal reinforcement should pass the equation (2.37). Moreover, the diameter of longitudinal rebar must be greater than 12mm, and at least two rebar should be extended all along the entire beam.

$$\rho \geq 0.8 \frac{f_{cta}}{f_{yd}} \quad (2.37)$$

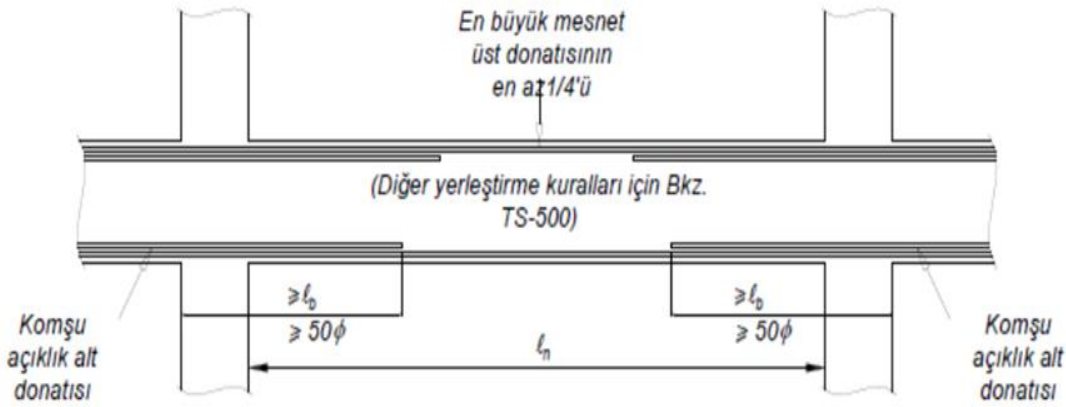


Figure 2.10 : Beam longitudinal reinforcement detailing

For seismic design category DTS = 1, 1a, 2, 2a structural building type, we should provide 50% of the negative moment section rebar at the bottom. This is because when an earthquake happens, it periodically changes its direction, and also it changes moment direction in the beams supporting section. Moreover, the ratio of rebar in the section must be less than 2% of the gross section area. Detailing of beam longitudinal reinforcement is given in Figure 2.10.

For transverse reinforcement or stirrups, there is a special area that is defined as two times the height of the beam. This section of the beam must be surrounded so carefully, and the first stirrup must be located 50mm from the face of support. Moreover, stirrup with a diameter of less than Ø8 should not be used. The spacing between the stirrups must be smaller of 1/4 of section height, Ø8 or 150mm. The shear force, which is

calculated for shear design of beam, is V_e . If V_r is less than V_e , we use V_r as a shear load for design. And V_e should satisfy equation (2.38).

$$V_e \leq V_r \qquad V_e \leq 0.85b_w d \sqrt{f_{ck}} \qquad (2.38)$$

2.6.15 High ductile shear walls

For shear walls, we say high ductile if and if the axial load of vertical loads and earthquake load, which is $G+Q+E$ over the net section area of shear wall and concrete strength less than or equal to 0.35. for shear walls, the ratio of a length over width in the section must be at least six or the length of shear wall in the section must be six times greater than its width.

In shear walls, we have a critical height, which is denoted H_{cr} and can be calculated using equation (2.39).

$$2l_w \geq H_{cr} \geq \max \left[l_w; \frac{H_w}{6} \right] \qquad (2.39)$$

And if $H_w/l_w > 2$ then, in the section of shear walls, there are end zones at the end part of shear walls. Within the critical height of the shear wall, the length of end zones is more generous of 2 times width or 20% of section length. Moreover, this length differs outside the critical height, which is more generous of shear wall width or 10% of section length.

In shear walls, the transverse and longitudinal reinforcement ratio in the section must not be less than 0.0025, and the spacing of transverse reinforcement must be greater than 250mm. In the center part of the shear wall, we must use special earthquake stirrups, and anchoring of 135 degrees must be applied. Splicing length must be greater than $1.5l_b$. The horizontal spacing of longitudinal reinforcement must be at least 200mm.

In the critical height of the shear wall, the ratio of rebar in end zones must be greater than 0.002, and this ratio outside the H_{cr} must not be less than 0.001.

2.6.16 Seismic design of high rise building in TBDY 2018

According to TBDY 2018 buildings with Building Height Classification (BYS) = 1 and;

- a) For DTS = 1, 1a, 2, 2a and $H_N > 70\text{ m}$
- b) For DTS = 3, 3a and $H_N > 91\text{ m}$
- c) For DTS = 4, 4a and $H_N > 105\text{ m}$

For buildings that satisfy this condition will go under classification of High Rise or Tall building. For a building that goes under the tall building category, the only high ductile structural system is allowed. Furthermore, for tall building shear wall width must not be less than 300mm.

The design of the tall building is done by applying just three steps, which are as follows in TBDY 2018 code.

Step I: Design of building under DD-2 earthquake level.

Step II: Performance evaluation of building under DD-3 or DD-4 for KK or SH.

Step III: Performance evaluation of building under DD-1 for GO or KH.

2.6.17 Step I: design stage

In this step, the building will be designed under the DD-2 earthquake level load by using a method of strength-based design (DGT). In this step first, the DD-2 Elastic acceleration spectrum is calculated from the AFAD site. Then we are finding R and D. After that, the Reduced elastic spectrum curve is found by dividing the elastic acceleration spectrum with R., and then the building should analyze. The result will be taken as internal loads for the design of sections. The steps of the design of structural elements are given in pervious chapter sections.

In this section, for calculating loads, we have to use the modal analysis method. In this method, the total modal participation factor in X and Y direction must be greater or equal to 95% of all modes. Moreover, modes that have a participation factor of 3% are considered in the calculation.

2.6.18 Step II: performance evaluation for KK or SH

In this step, the building will be analyzed under the DD-4 earthquake level for Immediate Occupancy KK performance level and under the DD-3 earthquake level for SH performance level. In this step, we do the analysis linear or nonlinear it depends on normal performance or further performance evaluation.

For the KK performance level, we use a linear analysis method, but for SH, we use a nonlinear analysis method. For the Strength-based design approach for analyzing a building, we use effective rigidity factors for elements which is given in Figure 2.11 below, and the eccentricity effect is ignored. The damping ratio in this method is decreased by 2.5%. It means we take a damping ratio of 2.5%. So to have it, we have to multiply the acceleration spectrum by 1.25.

The loads which are used in the first step for a design we use it also in the second step for performance evaluation of the system. For linear modal analysis of the system for KK in this step, we use R and D equal to 1, and the minimum base shear rule is not applicable for this step.

Table 2.11 : Rigidity reduction factor for KK or SH performance level

Concrete Structure System Elements	Effective Section rigidity Factor	
	Axial	Shear
Shear Wall - Slab (In Plane)		
Shear Wall	0.75	1.00
Basement Shear Wall	1.00	1.00
Slab	0.5	0.8
Shear Wall - Slab (Out of Plane)	Flexural	Shear
Shear Wall	1.00	1.00
Basement Shear Wall	1.00	1.00
Slab	0.5	1.00
Frame Element	Flexural	Shear
Spandrel Beam	0.3	1.00
Frame Beam	0.7	1.00
Frame Column	0.9	1.00
Shear wall	0.8	1.00

For normal performance evaluation, after analyzing the building under the DD-4 earthquake load, we can have the internal load in each element as the load to be compared for performance checking.

For further performance evaluation, after analyzing the building under DD-3 earthquake load by the method of time history nonlinear analysis method. In this method, we take a 2×11 seismic record from the PEER database and then analyze the building for each individually. Then to have the result, we have to take the average of 2×11 seismic records.

Now to evaluate the performance of the building elements, we need to find the section capacity of the element. In finding section capacity of elements, we use expected material strength, which is given in the Figure 2.12 below.

Table 2.12 : Expected material strength is given in TBDY 2018

Concrete	$f_{ce} = 1.3f_{ck}$
Rebar	$f_{ye} = 1.2f_{yk}$
Rebar (S235)	$f_{ye} = 1.5f_{yk}$
Rebar (S275)	$f_{ye} = 1.3f_{yk}$
Rebar (S355)	$f_{ye} = 1.1f_{yk}$
Rebar (S460)	$f_{ye} = 1.1f_{yk}$

Now to check the performance of an element in building for normal performance evaluation is like below:

- a) For high ductile elements of the building, the ratio of the internal load overcapacity should be equal to or less than 1.5. In equation (2.40), E is vertical loads effect and K is the correspondent capacity of section.

$$E/K \leq 1.5 \quad (2.40)$$

- b) For elements which are not ductile the ratio of the internal load overcapacity should be equal to or less than 0.7. equation (2.41) is used for calculating the ratio.

$$E/K \leq 0.7 \quad (2.41)$$

For further performance evaluation, we first find the rotation and strain of the elements and then comparing them with the limits which are given in 5.8.1 section of TBDY 2018.





3. CASE STUDY: DESIGN OF A HIGH RISE RC-BUILDING

In this case study, a high rise building is linear analyzed and designed according to Turkey New published code TBDY 2018. After the analysis and design, according to IYBDY, a time history analysis is done, and its performance is checked and compared with different local and international codes.

3.1 Building General Information

The building, which is considered for this study, is going to be built in Kocaeli with longitude and latitude of (29.977887, 40.803903). This building plan dimension is 36 m x 30 m. This building is for multiple purposes, such as residential and office with parking lots. The building is composed of a total of 24 stories, which are two basement floors, one ground floor, and 21 ordinary stories. The basements are 4 m, and other ordinary stories are 3.5m in height. The total height of the building is 88m basement floor included.

Because I do not have any information about the soil and ground which building is going to be constructed on it, I randomly selected the ZC type of ground. Kocaeli is exposed to first degree seismic hazard, according to Turkey hazard map 2018. The concrete and reinforcement classes used in structural elements of the tall building are C40 and S420a, respectively. An analysis of structure accomplished under seismic and vertical loads. In design stage of building a 20 cm slab assigned for the structure and then rigid diaphragm assigned to it. However, in seismic performance evaluation of tall building rigid diaphragm is changed to elastic diaphragm and real slab structure is assigned to the ETAB software. Waffle slab structure is assigned for this structure because of long span it has in two directions. Through this study, waffle slab is also designed and detailed.

3.2 Structural System

The building has a typical structural system that is composed of the frame (beam and columns) and shear walls. All loads will be carried together with frame and shear walls. Because the span is so long about 12m×10m, a waffle slab system is considered for the slab of structure. The plan and elevation view of building is given in Figure 3.1 and Figure 3.2.

Table 3.1 : Frame elements name and dimension in building

Exterior Beams		
Name	b (cm)	h (cm)
B1	50	75
B2	50	75
B3	50	75
B4	50	75
Interior Beams		
Name	b (cm)	h (cm)
B5	60	70
B6	45	60
B7	45	60
B8	45	60
B9	50	75
Columns		
Name	b (cm)	h (cm)
C1	90	90
C2	80	80
C3	45	60
Shear Walls		
Name	a (cm)	b (cm)
P1	1040	50
P2	600	50

Waffle slab is a slab that has intersected ribs inside its flat slab. The structure has an exterior of boundary shear walls and core shear walls at the center of the building. The shear wall thickness is 50cm in the first 12 stories, and it will be continued as 45 cm through the top of the building. The sizes of structure elements are given below in the Table 3.1.

3.3 Loads

The loads considered for this structure are Dead load, live load, Earthquake load, superimposed load. Wind load is ignored for this structure because the wind is not critical on the which is going to be built. Dead load is calculated directly by ETAB.

Live load = 2 kNm/m^2

Superimposed Dead load = 4 kNm/m^2

Concrete specific weight = 25 kNm/m^3

$d' = 4 \text{ cm}$ for all frame elements

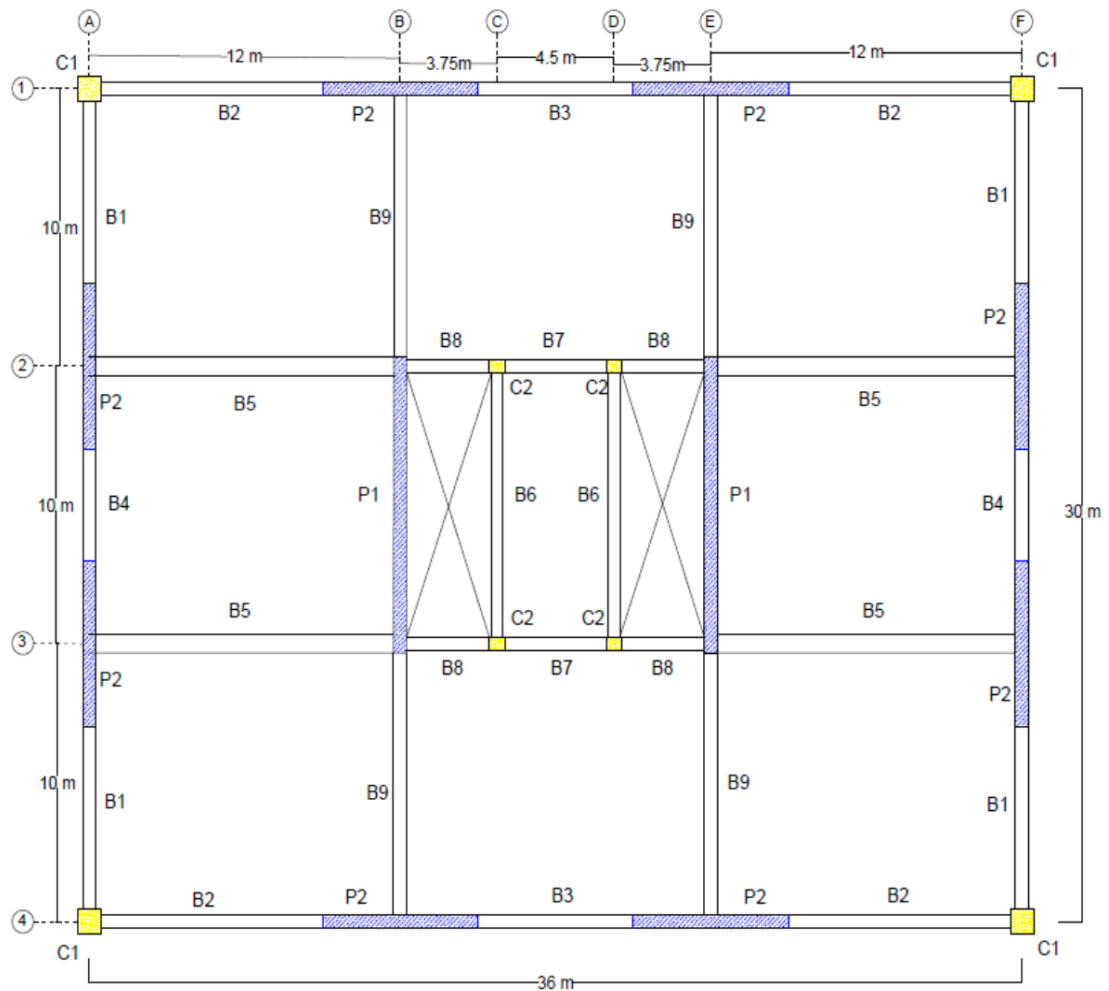


Figure 3.1 : Plan view of the structure

3.4 Structural System Modeling

For analyzing and design of the structure, the ETAB program is used. First, the structure of all information is entered into the program. The beam and column are modelled as frame elements in the system, shear walls modelled as shell and slabs modelled as shell membrane. Then all loads applied to the system as noted above. For earthquake load first, the spectrum is calculated and assigned into the program; then, the modal analysis is used for calculating fundamental periods. Slabs are then idealized as the rigid diaphragm. The Figure 3.2 shows the ETAB model of the structure.

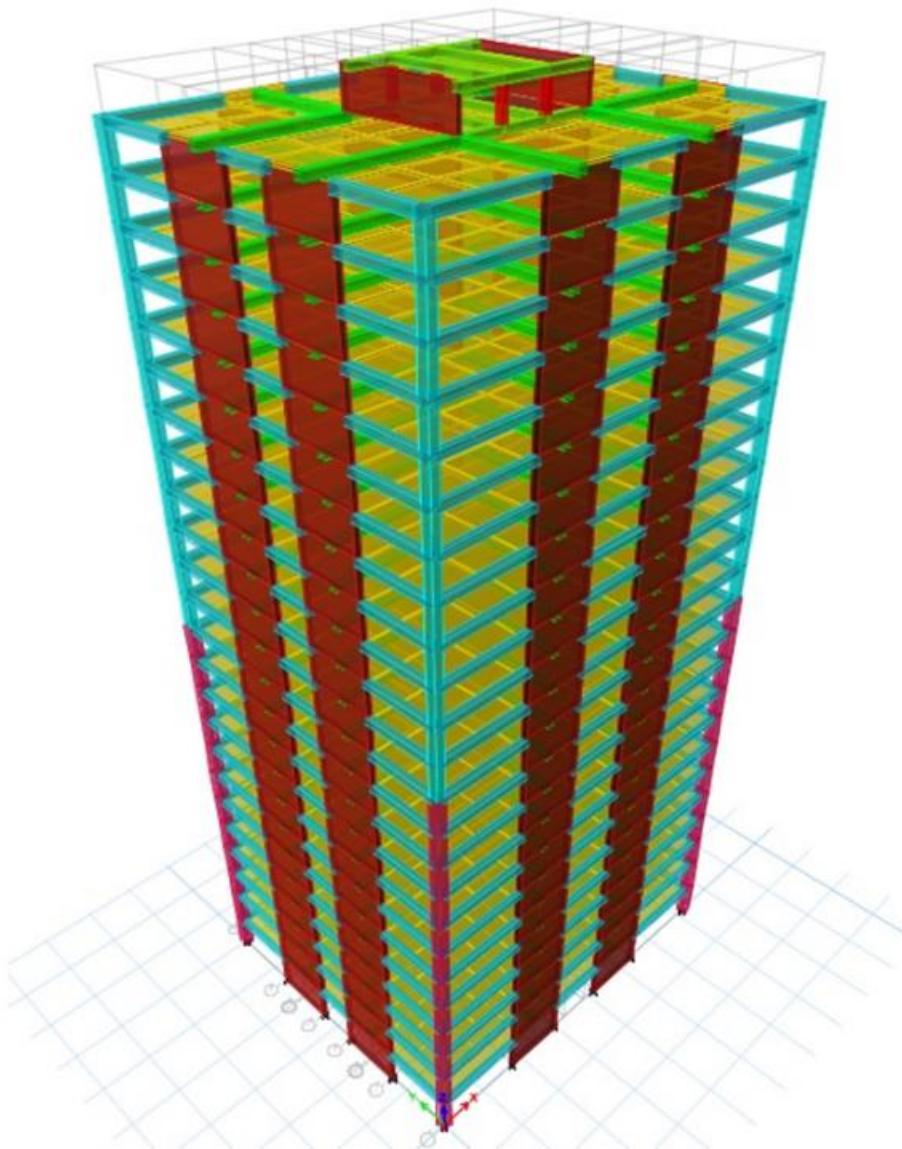


Figure 3.2 : 3D view of the entire structure

3.5 Design of Building According to TBSC 2018

3.5.1 Calculation method in ETAB

There are three methods of analysis for earthquake loads, which are: Equivalent Earthquake load method, Modal Analysis method, and Time History analysis method. For this structure, modal analysis selected for the analysis of seismic load with the help of the ETAB program. All floor is considered as a rigid diaphragm. A total of 25 modes are used to find the earthquake load, and their resultant is found by the method of SRSS. In linear modal analysis of system generally SRSS method will give us the required result and no need for CQC method. With a total of 25 modes, the sum of the mass participation factor is 0.9814 and 0.9735 in X and Y direction, respectively which is given in Table 3.2 below. In 4.8.1.2 section of TBDY 2018 says that the sum of mass participation factor must be greater than 0.95. Moreover, %5 damping is considered in all cases.

3.6 Elastic Spectrum and Modal Analysis of Structure

Calculating the elastic spectrum, according to TBDY 2018, is much more different than DBYBHY 2007 code, as we mentioned before. Here to calculate the Elastic spectrum, we first need to S_1 and S_s . These parameters can be found from Earthquake Hazard Map. First, we are entering the coordinate of building into AFAD site tdth.afad.gov.tr then AFAD will give us the S_1 and S_s . then, with the help of these terms and some other term, which can be taken from TBDY 2018 we can construct Elastic spectrum.

In this case study, this building with this purpose and criteria of office and residential goes under $BKS = 3$ category, and the Importance factor for this building is one according to table 3.1 of TBDY 2018.

DTS for this building is taken from table 3.2 of TBDY 2018 with the help of S_{Ds} and BKS factors. All building parameters are given in Table 3.3 below.

Earthquake reduction factor R and D is taken from table 4.1 TBDY 2018 with cross-sectioning of structural system type of building.

After that, we can find the allowable height of the building, which we are permitted to construct on that location. This can be found after knowing the BYS and DTS of buildings. According to table 3.4 of TBDY 2018, we will be able to find the performance level of the building, which can be found after knowing the DTS and Earthquake level.

Table 3.2 : Modal mass participating ratios and modal periods.

Modal Participating Mass Ratios						
Case	Mode	Period (sec)	UX	UY	Sum UX	Sum UY
Modal	1	2.181	0.72420	0.00000	0.7242	0
Modal	2	1.618	0.00000	0.68870	0.7242	0.6887
Modal	3	1.333	0.00000	0.00000	0.7242	0.6887
Modal	4	0.624	0.12360	0.00000	0.8478	0.6887
Modal	5	0.413	0.00000	0.15400	0.8478	0.8427
Modal	6	0.378	0.00000	0.00000	0.8478	0.8427
Modal	7	0.297	0.05260	0.00000	0.9003	0.8427
Modal	8	0.182	0.00000	0.05900	0.9003	0.9017
Modal	9	0.179	0.00000	0.00001	0.9003	0.9017
Modal	10	0.174	0.02970	0.00000	0.9301	0.9017
Modal	11	0.114	0.01850	0.00000	0.9486	0.9017
Modal	12	0.107	0.00000	0.02830	0.9486	0.93
Modal	13	0.105	0.00000	0.00000	0.9486	0.93
Modal	14	0.093	0.00000	0.00460	0.9486	0.9346
Modal	15	0.082	0.01210	0.00000	0.9607	0.9346
Modal	16	0.069	0.00000	0.00002	0.9607	0.9346
Modal	17	0.069	0.00000	0.01850	0.9607	0.9531
Modal	18	0.064	0.00480	0.00000	0.9655	0.9531
Modal	19	0.064	0.00150	0.00000	0.967	0.9531
Modal	20	0.058	0.00380	0.00000	0.9708	0.9531
Modal	21	0.051	0.00000	0.01210	0.9708	0.9652
Modal	22	0.05	0.00000	0.00000	0.9708	0.9652
Modal	23	0.048	0.00590	0.00000	0.9767	0.9652
Modal	24	0.04	0.00470	0.00000	0.9814	0.9652
Modal	25	0.039	0.00000	0.00820	0.9814	0.9735

Table 3.3 : All factor found from TBDY 2018 code for this structure

Name of Factor	Magnitude
BKS	3
Importance Factor	1
DTS	1
R	7
D	2.5
BYS	1
n	0.3
H	88 m basements included
Ground Type	ZC
Ss	1.486
S1	0.406

According to table 2.1 and 2.2 of TBSC 2018, we can get the values F_s of and F_1

$$F_s = 1.3$$

$$F_1 = 1.4$$

$$S_{DS} = S_s \times F_s = 1.486 \times 1.3 = 1.9318 \text{ s}$$

$$S_{D1} = S_1 \times F_1 = 0.406 \times 1.4 = 0.5684 \text{ s}$$

Now calculating Elastic Spectrum which is given in Figure 3.3

$$T_A = 0.2 \times \frac{0.5684}{1.9318} = 0.0588 \text{ s}$$

$$T_B = \frac{0.5684}{1.9318} = 0.294 \text{ s}$$

$$T_L = 6 \text{ s}$$

$$S_{ae}(T) = \left[0.4 + 0.6 \times \frac{T}{0.0588} \right] 1.9318 \quad 0 \leq T \leq T_A$$

$$S_{ae}(T) = 1.9318 \quad T_A \leq T \leq T_B$$

$$S_{ae}(T) = \frac{0.5684}{T} \quad T_B \leq T \leq T_L$$

$$S_{ae}(T) = \frac{0.5684 \times 6}{T^2} \quad T_L \leq T$$

Now we are calculating R and constructing its graph which is given in Figure 3.4 below. Reduced elastic horizontal design spectrum is given in Figure 3.5 below.

$$R_a = \frac{R}{I} = \frac{7}{1} = 7 \quad T > 0.294 \text{ s}$$

$$R_a(T) = 2.5 + \left[\frac{7}{1} - 2.5 \right] \frac{T}{0.294} \quad T \leq 0.294 \text{ s}$$

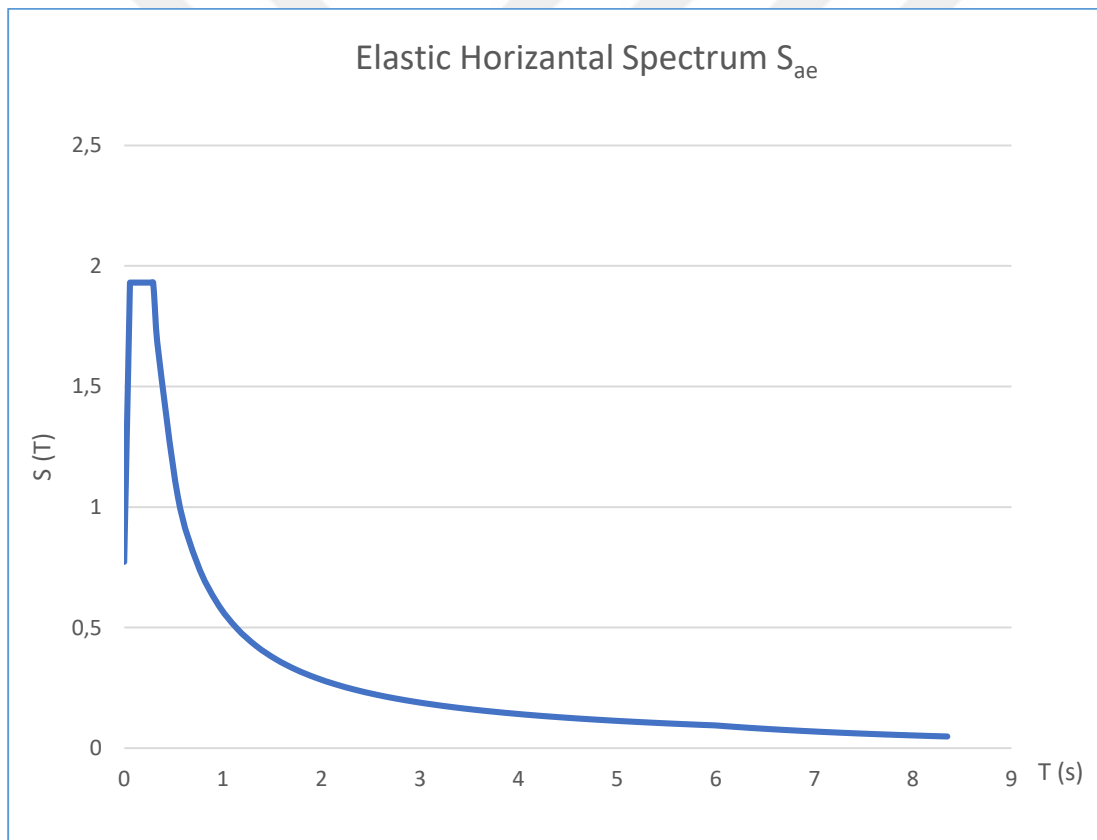


Figure 3.3 : Elastic horizontal design spectrum for ZC type ground

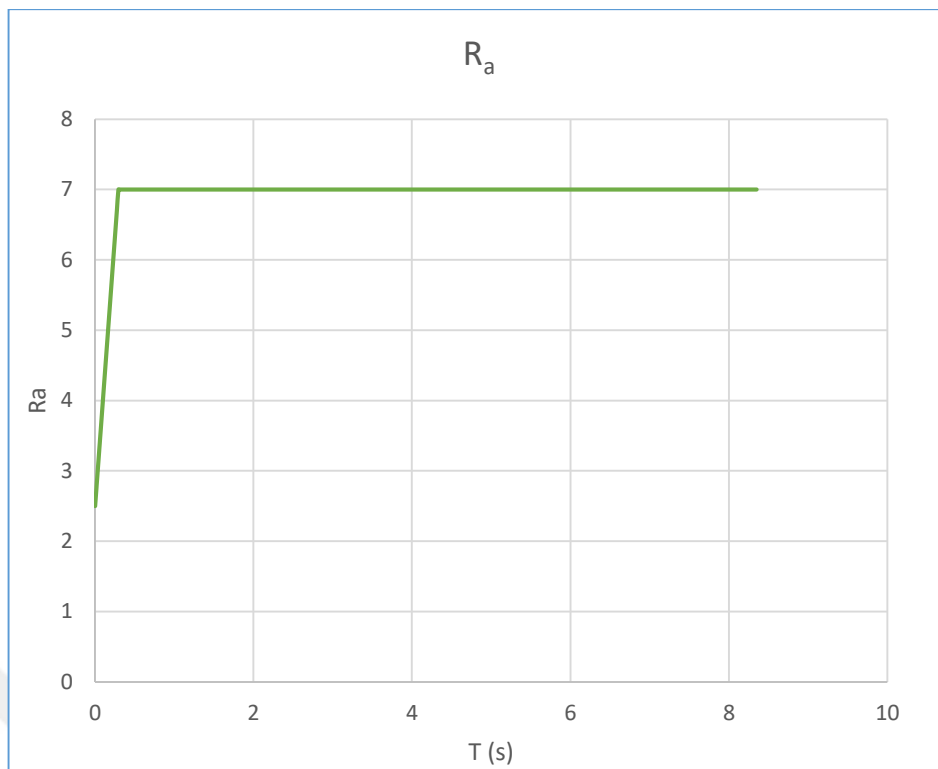


Figure 3.4 : Seismic load Reduction factor R for the structural system

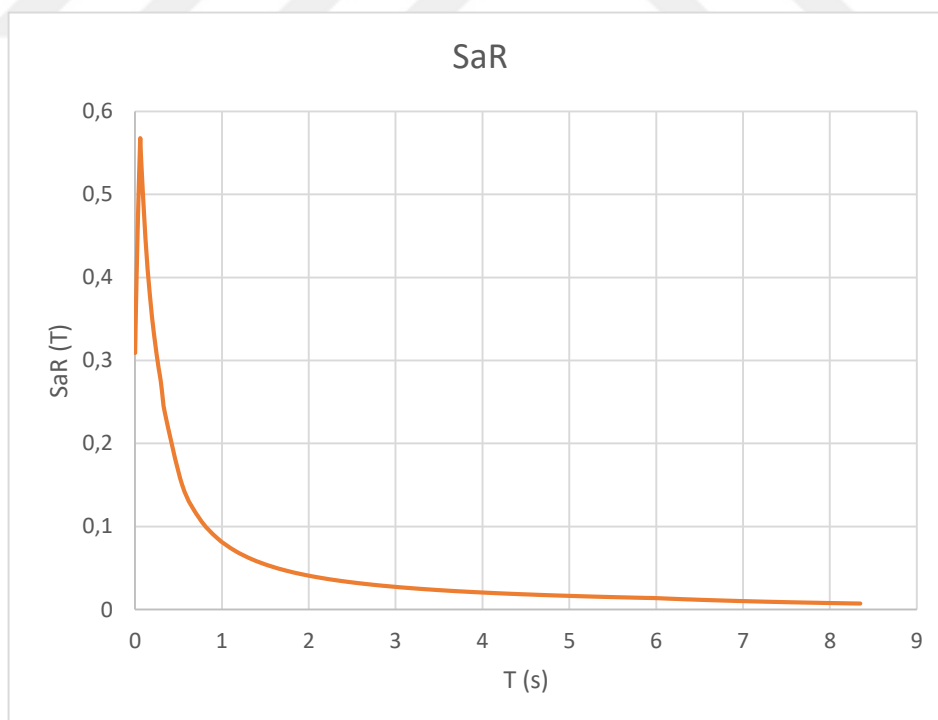


Figure 3.5 : Reduced elastic design spectrum

3.7 Structural Irregularity Checking

3.7.1 Horizontal irregularity

a) A1 Torsional Irregularity

As I mentioned in the previous chapter, if the ratio of maximum relative displacement over average relative displacement in any seismic direction and any story be greater than 1.2, then the structure has A1 type irregularity.

As we see in the table below, which is constructed for X and Y direction. Then maximum displacement and average displacement of all stories are taken from ETABs after analysis of the structure. After that, we calculated the maximum and average relative displacement of all stories and took their ratios. Then this parameter is compared with limits, and as we see in the table below, none of the story ratios is greater than the limit. So we can say there is not A1 irregularity in structure, and its already apparent from symmetrical geometry of plan that, there should not be a significant torsional irregularity. Calculations given in Table A.5 and Table A.6.

b) A2 Floor Discontinuity

If the total area of all discontinuities on a floor is 1/3 of the total area of a floor, then there is A2 irregularity in structure. Stairs and lifts openings are also included in this calculation. Alternatively, if there is a sudden change in the rigidity of the slab. According to the result below, there is not A2 type irregularity.

Total Area of Floor = 1080 m^2

Total opening Area in plan = 80 m^2

$$A_b/A = \frac{80}{1080} = 0.074 < 1/3 \quad \text{OK}$$

c) A3 Projection in Plan irregularity

If there is a projection in the floor of any story and its length or wide be greater than 20% of the floor dimension on that direction, then the structure has A3 irregularity,

Because as we see in the plan of structure, there is not any projection in the plan, so we do not have A3 irregularity.

3.7.2 Vertical irregularity

a) B1 weak story irregularity

In two significant earthquake directions, if the ratio of the effective shear area of a story over the effective shear area of its adjacent story is less than 0.8, then there is a sudden change in the shear area and B1 irregularity.

As we see in elevation and sections of frame elements, all shear walls and columns are continuous from the bottom up to the top of the building, and there is not a significant change in cross-section of shear walls and columns. So, in conclusion, there is not B1 irregularity because the effective shear area ratio is not less than 0.8.

b) B2 soft story irregularity

Under earthquake load in two main directions X and Y, the ratio of average relative displacement of a story over its adjacent story be greater than two then the structure has a B2 irregularity.

As we see in the table below, calculations are done for two directions individually. After analyzing the system in ETABs, I took story displacements and then calculated relative displacements and then took their ratio. As it is evident in the table below that none of the story ratio is greater than 2. So there is not any soft-story that exist in the structure. Calculations given in Table A.7 and Table A.8 in Appendix A.

3.8 Relative Story Displacement Controlling

Relative story displacement is a difference between two adjacent stories of a building. According to TBDY 2018, it is needed to be checked for a building before design. Story displacements are taken from ETABs after analysis of the structure. Then according to formulas mentioned in the previous chapter, effective relative story displacement is calculated, and then the result is compared with limits given in code.

R and I factor are found from tables 4.1 and 3.1 of TBDY 2018, respectively. This factor is 1 for this structure because it is a concrete structure.

As we mentioned above, there are two limits for checking in TBDY 2018. However, we use the equation (3.1) for comparing and checking because, in this structure, we assume there is not elastic material between structural and non-structural elements.

$$\lambda * \frac{\delta i (max)}{h_i} \leq 0.008 \kappa \quad \text{for condition (a)} \quad (3.1)$$

As we see in tables below, some calculations done for two directions under earthquake load and then results compared with limits. In conclusion, there is not any story that crossed the limit.

3.9 P-Δ Effect controlling

It is a term that must be checked and controlled as per the TBDY 2018 code. If the P-Δ effect exists in the structure, then a factor β must be calculated according to TBDY 2018 code, and all internal loads must be multiplied with this factor. This factor must be greater than one, which means if there is P-Δ, then all internal loads must be magnified with a factor.

Limits given in TBDY 2018 is different from DBYBHY 2007. It is a limit that a constant multiplied with D and divided by R and C_h factors. R and D factors are taken from table 4.1 of TBDY 2018. In equation (3.2), C_h is constant, which is 0.5 for this structure because it is a concrete structure.

$$\theta_{II,max} \leq 0.12 \frac{D}{C_h * R} \quad (3.2)$$

$$\theta_{II,max} \leq 0.12 \frac{2.5}{0.5 * 7} = 0.086$$

As we see in the tables below that θ_{II} factor in any story is not greater than 0.086. so for this structure, P-Δ is not significant, and we are not considering it in calculations. Calculations for two main directions is given in Table A.11 and Table A.12 of Appendix.

3.10 Load Combination

Table 3.4 : Load combinations for the analysis of structure in ETABS

Load Combination	
Load Name	Symbol
Dead Load	G
Live Load	L
Super Imposed Dead Load	S
Earthquake Load X Direction	Ex
Earthquake Load Y Direction	Ey
Number	Load Combination
1	$1.4G+1.4S+1.6Q$
2	$G+Q+S+Ex+0.3Ey$
3	$G+Q+S+Ex-0.3Ey$
4	$G+Q+S-Ex+0.3Ey$
5	$G+Q+S-Ex-0.3Ey$
6	$G+Q+S+Ey+0.3Ex$
7	$G+Q+S+Ey-0.3Ex$
8	$G+Q+S-Ey+0.3Ex$
9	$G+Q+S-Ey-0.3Ex$
10	$0.9G+0.9S+Ex+0.3Ey$
11	$0.9G+0.9S+Ex-0.3Ey$
12	$0.9G+0.9S-Ex+0.3Ey$
13	$0.9G+0.9S-Ex-0.3Ey$
14	$0.9G+0.9S+Ey+0.3Ex$
15	$0.9G+0.9S+Ey-0.3Ex$
16	$0.9G+0.9S-Ey+0.3Ex$
17	$0.9G+0.9S-Ey-0.3Ex$

3.11 Section Design of Concrete Structure Element

The structure analysis and design are done by a computer program, which is known as ETABs. This design is done according to the rules and standards of the new Turkish code TBDY 2018. As per table 3.4 of TBDY 2018 for high rise buildings and Seismic motion level of DD-2, the structure should be designed according to its strength or Strength-Based Design (DGT). Design is done, and then detail is all available in Appendix A. Moreover, as an example, I have done some section designs by hand to show the procedure of the design.

3.11.1 Beam section design calculation

The beam I am going to design as an example is beam B5 and located in the 8th story of structure and on-axis 3-3 of the plan which we can see in Figure 3.6 below.

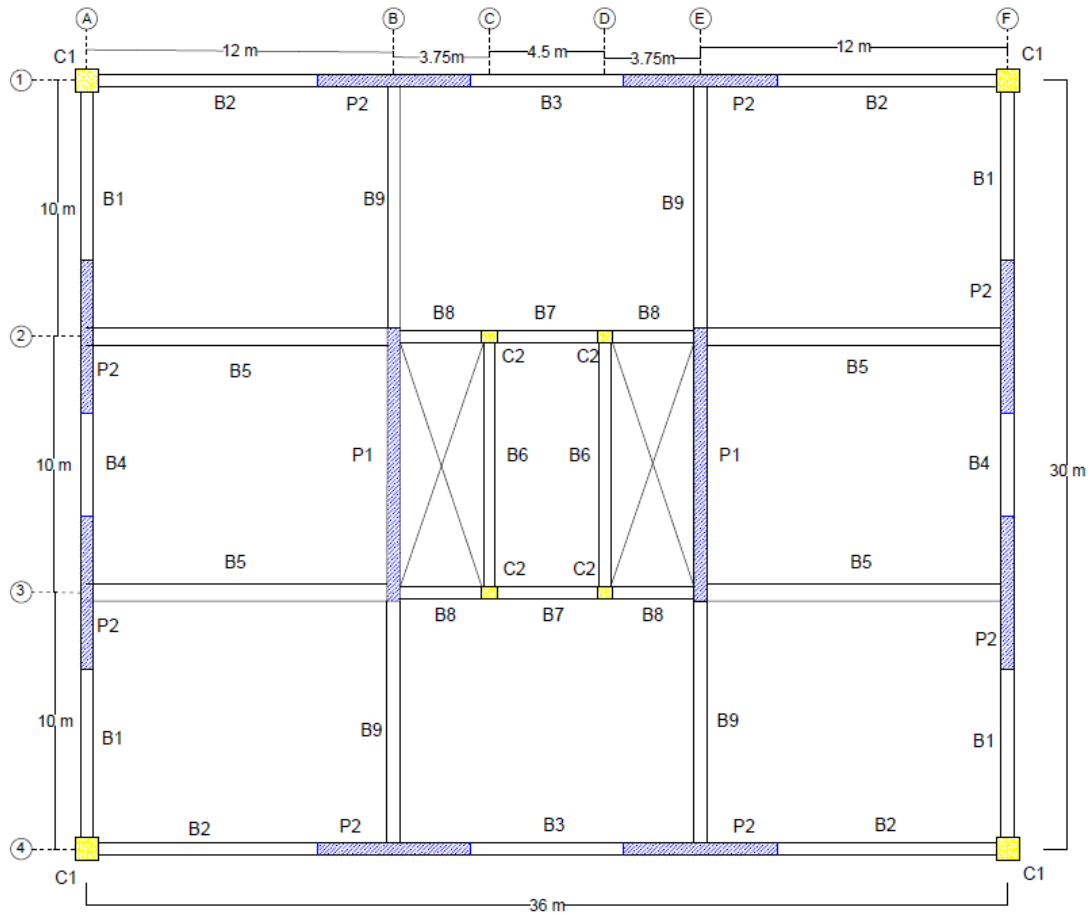


Figure 3.6 : Plan view of structure

First, we have to find b_e or effective web length according to ACI 318-14 code.

$$b_e \leq \frac{\text{Beam span Length}}{4} = \frac{1200}{4} = 300 \text{ cm}$$

$$b_e \leq b_w + 2(8h_f) = 60 + 2 * (8 * 12) = 252 \text{ cm}$$

$$b_e \leq b_w + \frac{2 * \text{travers length}}{2} = 60 + \frac{2 * 1000/2}{2} = 1060 \text{ cm}$$

The minimum of these values is taken as web effective length, so b_e is 252 cm.

$$L = 12m$$

$$b_w = 60 \text{ cm}$$

$$h = 70 \text{ cm}$$

$$b_e = 252 \text{ cm}$$

$$\phi = 0.9 \text{ for tension control} \quad K_1 = 0.76 \quad d = 64 \text{ cm}$$

$$M_{\text{midpoint}} = 505 \text{ kN.m} \quad M_{\text{left}} = -841 \quad M_{\text{right}} = -790 \text{ kN.m}$$

Mid-span flexural Design

After two times of iteration using equation (3.3), we can find the real demand of rebar area

$$A_s \geq \frac{M_u}{\phi * f_y * j * d} \quad (3.3)$$

$$A_s \geq \frac{M_u}{\phi * f_y * j * d} = \frac{505 * 10^6 \text{ N.mm}}{0.9 * 420 \text{ N/mm}^2 * 0.9 * 640 \text{ mm}} = 23.19 \text{ cm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f_c * b_e} = \frac{23.19 * 420 \text{ MPa}}{0.85 * 40 \text{ MPa} * 252 \text{ cm}} = 1.136 \text{ cm}$$

$$\begin{aligned} A_s &\geq \frac{M_u}{\phi * f_y * \left(d - \frac{a}{2}\right)} = \frac{505 * 10^6 \text{ N.mm}}{0.9 * 420 \text{ N/mm}^2 * \left(640 \text{ mm} - \frac{11.36 \text{ mm}}{2}\right)} \\ &= 21.06 \text{ cm}^2 \end{aligned}$$

According to TS 500 code, the minimum and maximum rebar ratio are:

$$\rho_{min} = 0.8 \frac{f_{ctd}}{f_{yd}} = 0.8 \frac{1.46}{365} = 0.004$$

$$A_{s,min} = 0.004 * (70 * 60) = 16.8 \text{ cm}^2$$

$$A_{max} = 0.02 * (70 * 60) = 84 \text{ cm}^2$$

So, according to TS 500 and TBDY 2018 area of longitudinal rebar is OK.

$$16.8 \text{ cm}^2 \leq A_s \leq 84 \text{ cm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f_c * b_e} = \frac{21.06 * 420 \text{ MPa}}{0.85 * 40 \text{ MPa} * 252 \text{ cm}} = 1.032 \text{ cm}$$

$$\text{for } 28 \leq f'_c \leq 56$$

$$\beta_1 = 0.85 - 0.05 \frac{f'_c - 28}{7} = 0.85 - 0.05 \frac{40 - 28}{7} = 0.762$$

$$c = \frac{a}{\beta_1} = \frac{1.032}{0.762} = 1.3502 \text{ cm}$$

$$\varepsilon_s = \frac{d - c}{c} * \varepsilon_{cu} = \frac{640 \text{ mm} - 13.502 \text{ mm}}{13.502 \text{ mm}} * 0.003 = 0.1392$$

$$\varepsilon_s = 0.1392 > 0.005 \text{ so the section is Tension Controlled}$$

$$\varepsilon_s = 0.1392 > 0.0021 \text{ so the tension rebar yeild}$$

Now selection Rebar $\phi=22 \text{ mm}$

$$\text{Area rebar } 22 \text{ mm} = \frac{22^2}{4} \pi = 3.801 \text{ cm}^2$$

$$\text{Number of Rebar} = \frac{21.06}{3.801} = 5.5 = 6 \quad \text{so } 6\phi 22$$

Supports Flexural Design

$$M_u = 841 \text{ kN.m}$$

$$A_s \geq \frac{M_u}{\phi * f_y * j * d} = \frac{841 * 10^6 \text{ N.mm}}{0.9 * 420 \text{ N/mm}^2 * 0.9 * 640 \text{ mm}} = 38.62 \text{ cm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f_c * b_e} = \frac{38.62 * 420 \text{ MPa}}{0.85 * 40 \text{ MPa} * 60 \text{ cm}} = 7.95 \text{ cm}$$

$$A_s \geq \frac{M_u}{\phi * f_y * \left(d - \frac{a}{2}\right)} = \frac{841 * 10^6 \text{ N.mm}}{0.9 * 420 \text{ N/mm}^2 * \left(640 \text{ mm} - \frac{7.95 \text{ mm}}{2}\right)} = 36.92 \text{ cm}^2$$

According to TS 500 code, the minimum and maximum rebar ratio are:

$$\rho_{min} = 0.8 \frac{f_{ctd}}{f_{yd}} = 0.8 \frac{1.46}{365} = 0.004$$

$$A_{s,min} = 0.004 * (70 * 60) = 16.8 \text{ cm}^2$$

$$A_{max} = 0.02 * (70 * 60) = 84 \text{ cm}^2$$

So, according to TS 500 and TBDY 2018 area of longitudinal rebar is OK.

$$16.8 \text{ cm}^2 \leq A_s \leq 84 \text{ cm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f_c * b_e} = \frac{36.92 * 420 \text{ MPa}}{0.85 * 40 \text{ MPa} * 60 \text{ cm}} = 7.6 \text{ cm}$$

$$c = \frac{a}{\beta_1} = \frac{7.6}{0.762} = 9.973 \text{ cm}$$

$$\varepsilon_s = \frac{d - c}{c} * \varepsilon_{cu} = \frac{640 \text{ mm} - 99.73 \text{ mm}}{99.73 \text{ mm}} * 0.003 = 0.016$$

$$\varepsilon_s = 0.016 > 0.005 \text{ so the section is Tension Controlled}$$

$$\varepsilon_s = 0.016 > 0.0021 \text{ so the tension rebar yeild}$$

Now selection Rebar $\phi=22$ mm

$$\text{Area rebar } 22 \text{ mm} = \frac{22^2}{4} \pi = 3.801 \text{ cm}^2$$

$$\text{Number of Rebar} = \frac{36.92}{3.801} = 9.71 = 10 \quad \text{so } 10\phi 22$$

Shear Design of Beam

In equation (3.4) states that the shear capacity should be greater than demand.

$$V_u = 370 \text{ kN} \quad \phi = 0.75$$

$$\frac{V_u}{\phi} = \frac{370}{0.75} = 493.33 \text{ kN}$$

$$\phi V_n \geq V_u \quad (3.4)$$

In equation (3.5) states that the shear capacity is composed of concrete and rebar shear capacity.

$$V_n = V_c + V_s \quad (3.5)$$

$$V_c = \lambda * \frac{\sqrt{f'_c} * b_w * d}{6} = 1 * \frac{\sqrt{40} \text{ N/mm}^2 * 600 \text{ mm} * 640 \text{ mm}}{6} = 404 \text{ kN}$$

$$\frac{V_c}{2} = 202 \text{ kN} \quad \text{because } V_u > \frac{V_c}{2} \text{ so Stirrups Required} \quad (3.6)$$

$$V_n = \frac{V_u}{\phi} = V_c + V_s \quad V_s = 90 \text{ kN}$$

$$\frac{A_v}{s} = \frac{V_s}{d * f_{yt}} = \frac{90 * 1000 \text{ N}}{640 \text{ mm} * 300 \text{ N/mm}^2} = 468 \text{ mm}^2/\text{m}$$

If we use $\phi 10\text{mm}$ as stirrups then

$$A_v = 2 * \text{area one leg of stirrup} = 2 * 78.5 = 157 \text{ mm}^2$$

$$s = \frac{157 \text{ mm}^2}{468 \text{ mm}^2/\text{m}} = 0.33 \text{ m} = 33 \text{ cm}$$

However, according to TBDY 2018 and TS 500 codes, the distance between stirrups must not exceed the minimum of below values.

$$s_k \leq h_k/4 = 64 \text{ cm}/4 = 16 \text{ cm}$$

$$s_k \leq 8\phi = 8 * 22 \text{ mm} = 17.6 \text{ cm}$$

$$s_k \leq 15 \text{ cm}$$

So the minimum of these values is 15 cm then we will put stirrups at spacing given below.

From support face-first stirrup $\phi 10$ at 5 cm

Then up to $2h_k = 2 * 64 = 128 \text{ cm}$ $\phi 10$ at 15 cm

And for rest of the beam, we put $\phi 10$ at 20 cm

Now we are checking the beam shear capacity according to TS 500 code. Design is done by ACI code. It must satisfy the TS 500 code too.

$$f_{ctd} = f_{ctk} / 1.5 = 2.2 / 1.5 = 1.46 \text{ N/mm}^2$$

$$V_{cr} = 0.65 * f_{ctd} * b_w * d = 0.65 * 1.46 \text{ N/mm}^2 * 600 \text{ mm} * 640 \text{ mm} = 364.4 \text{ kN}$$

$$V_w = \frac{A_{sw}}{s} * f_{yd} * d = \frac{157 \text{ mm}^2}{150 \text{ mm}} * 300 \text{ N/mm}^2 * 640 \text{ mm} = 201 \text{ kN at support}$$

$$V_w = \frac{A_{sw}}{s} * f_{yd} * d = \frac{157 \text{ mm}^2}{200 \text{ mm}} * 300 \text{ N/mm}^2 * 640 \text{ mm} = 151 \text{ kN at Midspan}$$

$$V_r = V_c + V_w = 364.4 + 201 = 565 \text{ kN at supports}$$

$$V_r = V_c + V_w = 364.4 + 151 = 515 \text{ kN at Midspan}$$

Now we have to find V_e , which is found with the calculation of beam edges section moment capacity. It is done with the help of an Xtract software. The equations (3.7) and (3.8) calculate the section shear capacity and plastic moment capacity respectively.

$$V_e = V_{dy} \pm (M_{pi} + M_{pj}) / l_n \quad (3.7)$$

$$M_{pi} = 1.4M_{ri} \text{ and } M_{pj} = 1.4M_{rj} \quad (3.8)$$

Shear load occurred by gravity loads

$$\text{Dead load shear} = 145 \text{ kN}$$

$$\text{Live load shear} = 51 \text{ kN}$$

$$\text{Super Imposed load shear} = 51 \text{ kN}$$

Total shear occurred by gravity loads is 247 kN then

$$V_{dy} = 247 \text{ kN}$$

$$M_{ri} = 822 \text{ kN.m} \quad \text{and} \quad M_{rj} = 421 \text{ kN.m}$$

$$M_{pi} = 1.4 * 822 \text{ kN.m} = 1150.8 \text{ kN.m}$$

$$M_{pj} = 1.4 * 421 \text{ kN.m} = 590 \text{ kN.m}$$

$$V_e = 247 + (1150.8 + 590) / 12 \text{ m} = 392 \text{ kN}$$

$$V_e = 247 - (1150.8 + 590) / 12 \text{ m} = 102 \text{ kN}$$

$$\text{So } V_e = 392 \text{ kN} < V_r = 565 \text{ kN}$$

$$V_e \leq 0.85 * b_w * d * \sqrt{f_{ck}} = 0.85 * 600 \text{ mm} * 640 \text{ mm} * \sqrt{40} \text{ N/mm}^2 = 2064 \text{ kN}$$

According to TBDY 2018 code beam section has enough shear capacity because

$$V_e < V_r < 2064 \text{ kN}$$

And as per TS 500 which is given in equation (3.9)

$$\frac{A_{sw}}{s} \geq 0.3 * \frac{f_{ctd}}{f_{wyd}} * b_w \quad (3.9)$$

$$\frac{157}{150} \geq 0.3 * \frac{1.45}{300} * 600$$

$$1.0466 \geq 0.87 \quad \text{so it satisfied the rule at supports}$$

3.11.2 Column design

The column which I decided to design as an example is located in the second story of building and axis 4-F in the name of C1.

$$N_d = 7398 \text{ kN}$$

$$A_c = 90 \text{ cm} * 90 \text{ cm} = 8100 \text{ cm}^2$$

$$\frac{N_d}{A_c * f_{ck}} \leq 0.4 \quad \text{then} \quad (3.10)$$

$$\frac{7398000 \text{ N}}{810000 \text{ mm}^2 * 40 \text{ N/mm}^2} = 0.228 < 0.4$$

In equation (3.10) it satisfies the rule of TBDY 2018, which is given for high ductile columns.

The minimum and maximum rebar ratio are given as below in TBDY 2018.

$$\rho_{min} = 0.01 \quad A_{smin} = 0.01 * 8100 = 81 \text{ cm}^2$$

$$\rho_{max} = 0.04 \quad A_{smin} = 0.04 * 8100 = 324 \text{ cm}^2$$

According to ACI 318-14:

$$\gamma = \frac{h - 2 * d'}{h} = \frac{90 - 2 * 5}{90} = 0.889$$

$$P_u = 3397 \text{ kN}$$

$$M_{u3} = 143 \text{ kN.m}$$

$$M_{u2} = 11 \text{ kN.m}$$

$$e_x = \frac{M_{uy}}{P_u} = \frac{143}{3397} = 0.04$$

$$e_y = \frac{M_{ux}}{P_u} = \frac{11}{3397} = 0.0032$$

$$\frac{e_x}{l_x} = \frac{0.04}{0.9} = 0.044$$

$$\frac{e_y}{l_y} = \frac{0.0032}{0.9} = 0.0035$$

$$\frac{e_x}{l_x} > \frac{e_y}{l_y} \quad (3.11)$$

$$\text{if } \frac{P_u}{f'_c * A_g} < 0.4$$

$$\text{then } \frac{3397000 \text{ N}}{40 \text{ N/mm}^2 * 810000 \text{ mm}^2} = 0.104 < 0.4 \quad (3.12)$$

$$\alpha = \left(0.5 + \frac{P_u}{f'_c * A_g} \right) * \left(\frac{f_y + 40000}{100000} \right) \quad f_u \text{ must be in ksi} \quad (3.13)$$

$$\alpha = (0.5 + 0.104) * \left(\frac{60000 + 40000}{100000} \right) = 0.604$$

$$e_{ox} = e_x + \frac{\alpha * e_y * l_x}{l_y} = 0.04 + \frac{0.604 * 0.0032 * 0.9}{0.9} = 0.04193$$

$$M_{oy} = P_u * e_{ox} = 3397 * 0.04193 = 142.4 \text{ kN.m}$$

Now we use e_{ox} and M_{ox} to find the rebar ratio required for this section.

With the help of Integration diagrams given in ACI 318-14 appendix, we can find the rebar ratio.

$$\frac{P_u}{A_g} = \frac{3397 \text{ kN}}{0.81 \text{ mm}^2} = 4194 \text{ kN/m}^2 = 0.628 \text{ ksi}$$

$$\frac{M_{oy}}{A_g * h} = \frac{142.4 \text{ kN.m}}{0.81 \text{ mm}^2 * 0.9 \text{ m}} = 195.4 \text{ kN/m}^2 = 0.029 \text{ ksi}$$

$$f'_c = 6 \text{ ksi} \quad f_y = 60 \text{ ksi}$$

Because $\gamma = 0.889$ is between 0.75 and 0.9, so we have to interpolate.

According to the first integration graph which is given below

$$\gamma = 0.75$$

ρ_s is less than 0.01 which is minimum ratio

$$\gamma = 0.9$$

ρ_s is less than 0.01 which is minimum ratio

So according to TBDY 2018, we have to put a minimum ratio of rebar in column section which is 0.01

$$\rho_s = 0.01 \quad A_s = 0.01 * 8100 \text{ cm}^2 = 81 \text{ cm}^2$$

$$\text{Area } \emptyset 22 = 3.801 \text{ cm}^2$$

$$\frac{81 \text{ cm}^2}{3.801 \text{ cm}^2} = 21.3 = 22 \quad \text{so } 22 \emptyset 22$$

$$\frac{N_d}{A_c * f_{ck}} = 0.228 > 0.2 \quad \text{so according to 7.3.4.1 (a) TBDY 2018}$$

If we use $s = 7 \text{ cm}$ in equation (3.14) and (3.15)

$$A_{sh} \geq 0.3 * s * b_k * \left[\left(\frac{A_c}{A_{ck}} \right) - 1 \right] * \left(\frac{f_{ck}}{f_{ywk}} \right) \quad (3.14)$$

$$A_{sh} \geq 0.3 * 70 * 800 * \left[\left(\frac{810000}{640000} \right) - 1 \right] * \left(\frac{40}{300} \right)$$

$$A_{sh} \geq 595 \text{ mm}^2$$

$$A_{sh} \geq 0.075 * s * b_k * \left(\frac{f_{ck}}{f_{ywk}} \right) \quad (3.15)$$

$$A_{sh} \geq 0.075 * 70 * 800 * \left(\frac{40}{300} \right)$$

$$A_{sh} \geq 560 \text{ mm}^2$$

So we need 595 mm^2 as shear ties

$$\text{Area } \emptyset 12 \text{ mm} = 113 \text{ mm}^2$$

We put three ties on each side with a total 6 legs

$$6 * \emptyset 12mm = 6 * 113 = 678 mm^2 \quad \text{so its OK}$$

3.11.3 The rule of ‘columns must be stronger than beams’

This phenomenon is so important in building for the prevention of sudden collapse of buildings. It states that the column capacity must be greater than beam capacity as we see in equation (3.16). It will let people who live in building to scape and save their life.

As an example to show the procedure, I selected a joint, which is located in the second story. In this joint, there are two columns and one beam. First, I calculated the column and beam capacity with the help of the Excel sheet, which I made and put it in the Appendix.

Because I will put the same rebar in the column on the first floor and the second floor, so the capacity is the same.

$$M_{ra} = 740 \text{ kN.m}$$

$$M_{ru} = 740 \text{ kN.m}$$

$$M_{ri} = 662 \text{ kN.m}$$

$$(M_{ra} + M_{ru}) \geq 1.2 (M_{ri} + M_{rj}) \quad (3.16)$$

$$(740 + 740) / 662 = 2.23 \geq 1.2 \quad \text{OK}$$

So, according to this calculation, columns are much stronger than beams, and this will increase the entire ductility of the building.

3.11.4 Checking column shear safety

According to TBDY 2018 code, we must check the shear capacity of the column, so as an example, I selected column C1 in the second story, which is located in axis 4-F. So first, we have to find the moment capacity of the beam and column located in this region.

In TBDY 2018 says that, if a column in joint satisfy the equation of 7.3 then can find column joint moment using equations (3.17) and (3.18):

$$V_e = (M_a + M_u) / l_n \quad (3.17)$$

$$\sum M_p = M_{pi} + M_{pj} \quad (3.18)$$

$$M_{rj} = 662 \text{ kN.m}$$

$$\sum M_p = M_{pj} = 1.4 * M_{rj} = 926 \text{ kN.m}$$

$$M_{nu(i)} = 740 \text{ kN.m}$$

$$M_{na(i+1)} = 740 \text{ kN.m}$$

$$M_u = \frac{M_{nu(i)}}{M_{nu(i)} + M_{na(i+1)}} * \sum M_p = 481 \text{ kN.m}$$

This value is the same for the top and bottom edge of the column then

$$V_e = (481 + 481) / 4 = 240 \text{ kN}$$

$$V_{Dead\ load} + V_{live\ load} + V_{Super\ Imposed} = 65 \text{ kN}$$

$$V_{e,D=2.5} = 2.5 * 65 = 163 \text{ kN}$$

$$V_{cr} = 0.65 * f_{ctd} * b_w * d = 0.65 * 1.45 * 900 * 800 = 678.6 \text{ kN}$$

$$V_c = 0.8 * V_{cr} = 0.8 * 678.6 = 542 \text{ kN}$$

$$\rho_{sh} = \frac{678.6}{70 * 800} = 0.0121$$

$$V_w = \rho_s * A_c * f_{yd} = 0.0121 * 900 * 900 * 365 = 3577 \text{ kN}$$

$$V_r = V_c + V_w = 542 + 3577 = 4120 \text{ kN}$$

$$V_e < V_r$$

$$V_e \leq 0.85 * A_w * \sqrt{f_{ck}} \quad (3.19)$$

$$V_e \leq 0.85 * b_w * d * \sqrt{f_{ck}} = 0.85 * 900 * 800 * \sqrt{40} = 3870 \text{ kN}$$

$$V_e < V_r < 3870 \text{ kN}$$

So, according to TBDY 2018, this column shear capacity is enough to cover the shear load coming from all load combinations.

3.11.5 Shear wall design

Shear wall, a vertical structural element that is used to carry most of the seismic load. It is designed to resist in-plane resisting forces typically wind and seismic loads. Most of the international codes carry shear wall design, and TBDY 2018 does.

The design of high ductile shear wall is done in the ETABs program. After all the model is done, and all load applied to the system, then the system is designed under seismic and gravity loads together. After analysis and design, all results are checked, and I put it in the appendix.

As an example, to know the procedure of design, one of the shear walls is designed manually, which is a shear wall of P2 in the axis of 4-4 of the plan. Then some design check is done for the shear wall to ensure that it governs the TBDY 2018 code.

According to part 7.6.1.1 of TBDY 2018, the sum of axial loads on a shear wall from G, Q, and E over the net area of concrete and concrete strength must be less than 0.35 if the shear wall is highly ductile as we see in equation (3.20). A shear wall is a wall that is made of wholly concrete, and its length must be greater than six times its width. For other shear walls the section limit checking is given in Table 3.5 below.

$$\frac{N_{dm}}{f_{ck} * A_c} \leq 0.35 \quad (3.20)$$

$$N_{dm} = 7968 + 7414 = 15382 \text{ kN}$$

$$f_{ck} = 40 \text{ MPa} \quad A_c = 6000 * 500 = 3000000 \text{ mm}^2$$

$$\frac{15382000 \text{ N}}{40 \text{ MPa} * 3000000 \text{ mm}^2} = 0.128 < 0.35 \quad OK$$

$$\frac{Length}{Width} \geq 6 \quad \text{then} \quad \frac{6000 \text{ mm}}{500 \text{ mm}} = 12 > 6 \quad OK$$

For shear walls, which their total height over its section length is greater than two, we have end zones on two sides of it. Moreover, every shear wall has a critical height, which is essential in rebar adjustment and ties adjustment. It is found by the following formula, which is given in TBDY 2018.

$$H_w = 88 \text{ m} \quad \ell_w = 6 \text{ m}$$

$$\frac{88 \text{ m}}{6 \text{ m}} = 14.6 > 2 \quad \text{then we have end zones}$$

$$2 * \ell_w \geq H_{cr} \geq \max \left[\ell_w : \frac{H_w}{6} \right] \quad (3.21)$$

$$2 * 6 \geq H_{cr} \geq \max \left[6 : \frac{88}{6} \right]$$

$$12 \text{ m} \geq H_{cr}$$

$$H_{cr} \geq 14.6 \text{ m} \quad \text{then } H_{cr} = 15 \text{ m}$$

So we take the H_{cr} height 15 m according to results, which is starting from the basement up to third story height.

Longitudinal reinforcement

The design is done by the ETABS program, and results are taken and checked according to TBDY 2018. Now we are doing a longitudinal reinforcement check for this shear wall, but first, we have to find the length of end zones and its central part. This is different within critical and height and out of critical height.

Within critical height, the length of end zones must be greater than two times width or 20% length of the shear wall.

Out of critical height, this length must be greater than width or 10% length of the shear wall as in equation (3.22).

$$\ell_{u1} \geq 2 * b_w \quad \text{or} \quad \ell_{u1} \geq 0.2 * \ell_w \quad (3.22)$$

$$\ell_{u1} \geq 2 * 50 \text{ cm} = 100 \text{ cm} \quad \text{or} \quad \ell_{u1} \geq 0.2 * 600 \text{ cm} = 120 \text{ cm}$$

So within critical height, the length of end zones is 120 cm.

$$\ell_{u2} \geq b_w \quad \text{or} \quad \ell_{u2} \geq 0.1 * \ell_w \quad (3.23)$$

$$\ell_{u2} \geq 50 \text{ cm} \quad \text{or} \quad \ell_{u2} \geq 0.1 * 600 \text{ cm} = 60 \text{ cm}$$

Then out of critical height, the length is 60 cm using equation (3.23).

After design for P2 shear wall according to ETABS the requirement longitudinal reinforcement is as follow; for end zones Ø 22/15 and for the center of shear wall Ø 18/20. Now we are checking the minimum requirement according to TBDY 2018 code.

In the center of the shear wall, the total area of longitudinal reinforcement over the net area between to end zones must be greater than 0.0025.

$$\rho_{govde} = \frac{A_{s,govde}}{b_w * d} = \frac{36 * 2.54}{360 * 50} = 0.00508 > 0.0025 \quad OK \quad \text{within } H_{cr}$$

$$\rho_{govde} = \frac{A_{s,govde}}{b_w * d} = \frac{36 * 2.54}{480 * 50} = 0.0038 > 0.0025 \quad OK \quad \text{out of } H_{cr}$$

In the end zones of the shear wall, the total area of rebar in any end zone over the net area of the shear wall section must be greater than 0.002.

$$\rho_{end\ zones} = \frac{A_{s,uc}}{b_w * d} = \frac{17 * 3.8}{600 * 50} = 0.00215 > 0.002 \quad OK$$

So, according to TBDY 2018, the shear wall satisfies the minimum requirements.

Traverse reinforcement

For the traverse reinforcement ratio, first, we have to find design shear force V_e using equation (3.24) given below. This is found by formulas given in TBDY 2018. The value of V_e is the smallest of the formula given below or the 1.2D of all vertical loads.

$$V_e = \beta_v * \frac{(M_p)_t}{(M_d)_t} * V_d \quad (3.24)$$

$$\beta_v = 1.5 \quad (\text{because load is carried with shear wall and column together})$$

$$V_d = 1340 \text{ kN}$$

$$(M_d)_t = 6340 \text{ kN.m}$$

$$(M_p)_t = 1.25 (M_r)_t = 26430 \text{ kN.m}$$

$$V_e = 1.5 * \frac{26430 \text{ kN.m}}{6340 \text{ kN.m}} * 1340 \text{ kN} = 8370 \text{ kN}$$

$$V_{vertical\ load} = 1803 \text{ kN}$$

$$V_{e,1.2D} = 1.2 * 2.5 * 1803 = 5409 \text{ kN}$$

So our design shear load is $V_e = 1803 \text{ kN}$

$$9777 \text{ kN} = 6000 \text{ mm} * 500 \text{ mm} * (0.65 * 1.46 \text{ MPa} + \rho_{sh} * 300 \text{ MPa})$$

$$\rho_{sh} = \frac{5409000 \text{ N}}{2847000 \text{ N} * 300} = 0.006332$$

Now selecting $\emptyset 14/10$

$$\rho_{sh} = \frac{2 * 1.54}{10 * 40} = 0.0077 \quad OK$$

$$V_r = 0.5 * 6 * (0.65 * 1460 + 0.0077 * 300000) = 9777 \text{ kN}$$

$$V_e \leq V_r \quad 1803 \text{ kN} < 9777 \text{ kN} \quad OK$$

$$V_e \leq 0.85 * A_{ch} * \sqrt{f_{ck}}$$

$$0.85 * A_{ch} * \sqrt{f_{ck}} = 0.85 * 6000 * 500 * \sqrt{40} = 16127 \text{ kN} < 5409 \text{ kN}$$

So, according to TBDY 2018 code, this structure satisfies all the traverse reinforcement requirements.

Table 3.5 : Section limit checking for high ductile shear walls

Name	Ndm (kN)	Fck (MPa)	Ac (cm ²)	ratio	Limit
P1	15382	40	30000	0.1282	0.35
P2	18640	40	30000	0.1553	0.35
P3	46320	40	52000	0.2227	0.35

3.12 Slab Design

The slab is one of the central parts of a structure. Slabs are horizontal surfaces that carry mostly the gravity loads. It is designed such that transfer vertical loads safely to beams and then to column and provide a safe and stable structure. The slabs may be supported by walls or by beams which are constructed monolithically with them. Slabs are only designed for gravity loads like dead load, live load, and superimposed dead load. There are different types of slabs available according to their shape. Some of them are like this below:

Flat slab or also we called beamless slab, which does not have a beam and is supported only by columns or caps. This slab is flat, so it gives an attractive appearance. Flat slab is light in weight, so it is also economical. Moreover, it needs an easy constructible framework.

The conventional slab is so typical in the construction industry. There are two types of conventional slab. If the ratio of longer span to shorter span is greater than 2, then the slab is on way slab. In this type of slab, bending occurs only in a shorter direction, so

we only provide minimum reinforcement in a longer direction due to temperature, creep, and shrinkage. If the ratio of a longer span to a shorter span is less than two, then the slab is called a two-way slab. Hence the bending occurs in two directions, so the main reinforcement is provided in both directions.

Waffle slab is a ribbed slab with inner beams in both directions with a constant spacing. Because Its Shape Is like a square grid, so it is also called grid slabs. PVC trays or (pods) are used for the framework of the hollow hole shapes in the waffle slab. We use a waffle slab when a large span is required. Waffle slab is resisted for sagging and cracking. This type of slab carries a large amount of load compared with a conventional slab, and also it is economical in the material usage side of view. Waffle slab is also light in weight and performs a good appearance.

In this thesis, a waffle slab is used because spans are such generous, and conventional slabs give a large thickness, which is uneconomical.

3.12.1 Waffle slab design calculations

The design of the slab is done with the SAFE program, a CSI program that is used for the design of slab and foundations.

Loads are given for this slab

Dead load = calculated by program directly

$$\text{Live load} = 2 \text{ Kg/m}^2$$

$$\text{super imposed dead load} = 4 \text{ Kg/m}^2$$

An effective rigidity factor is given to the program according to the Table 3.6 below, which is 0.25 in bending for the slab. Moreover, for shear, the rigidity factor is one; it means we are not allowed to decrease the rigidity of the slab in shear.

The design of the waffle slab is done according to ACI 318-14 code. Preliminary sizing of the slab is calculated according to ACI 318-14. For a slab without drop panels, with edge beams and rebar with $f_y = 420 \text{ MPa}$ has a minimum thickness of:

$$h (\text{thickness}) = l_n/33 \text{ for interior and exterior panels} \quad (3.25)$$

$$h = l_n/33 = \frac{1200 \text{ cm}}{33} = 37 \text{ cm}$$

so it means if we want to select a conventional two-way slab, we need a minimum thickness of 37 cm slab. However, not only it is uneconomical, but it also fails in two-way shear. Typical section of slab is given in Figure 3.7 below.

In this case, we have four options

- 1) Increase slab thickness
- 2) Use headed shear reinforcement in the slab
- 3) Use drop panels at column
- 4) Use waffle slab

Calculations done to find the slab dimensions

- a) The width of the ribs should be at least 10.5 cm, so we use 15 cm for our slab.
- b) The overall depth of ribs should be 3.5 times the minimum width.

Table 3.6 : Rigidity reduction factor for slab

Betonarme Taşıyıcı Sistem Elemanı	Etkin Kesit Rijitliği Çarpanı	
<i>Perde – Döşeme (Düzlem İçi)</i>	<i>Eksenel</i>	<i>Kayma</i>
Perde	0.50	0.50
Bodrum perdesi	0.80	0.50
Döşeme	0.25	0.25
<i>Perde – Döşeme (Düzlem Dışı)</i>	<i>Eğilme</i>	<i>Kesme</i>
Perde	0.25	1.00
Bodrum perdesi	0.50	1.00
Döşeme	0.25	1.00
<i>Çubuk eleman</i>	<i>Eğilme</i>	<i>Kesme</i>
Bağ kirişi	0.15	1.00
Çerçeve kirişi	0.35	1.00
Çerçeve kolonu	0.70	1.00
Perde (eşdeğer çubuk)	0.50	0.50

$$3.5 \times 15 \text{ cm} = 52 \text{ cm}$$

We use 47 cm for the overall depth of the slab

c) Clear spacing should not exceed 76 cm, so we use 75 cm

d) Slab thickness shall be at least

$$1) \frac{1}{12} \times \text{clear spacing} = 6.25 \text{ cm}$$

$$2) 5 \text{ cm}$$

We use 10 cm thickness for this slab, and a clear depth of rib will be 37 cm.

In the middle part of the slab in the exit area, which is 4.5m x 10m, we use here one-way slab because the span is not much longer to apply waffle slab, and it would be uneconomical. The ratio of long dimensions over the short one is greater than two so it is like a one-way slab. The load goes in one way, so we put the main reinforcement in one way, and on the other side, we put reinforcement just for shrinkage, creep, and crack.

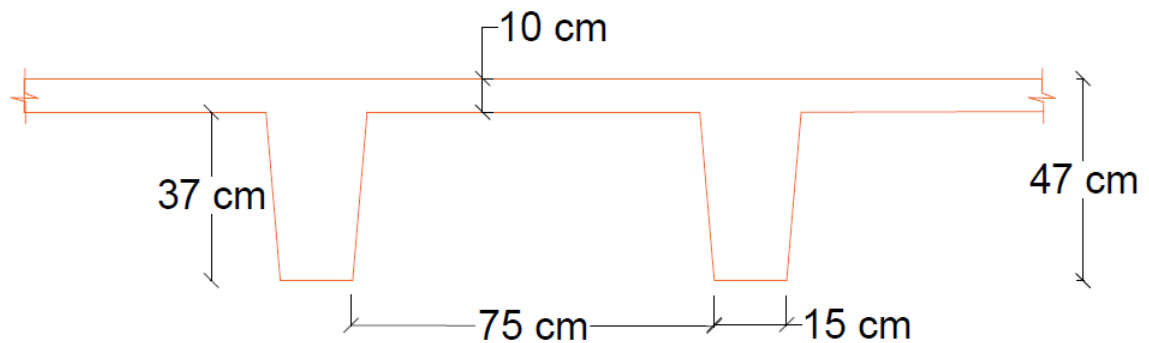


Figure 3.7 : Typical waffle slab section

3.12.2 Drop panel sizing

A drop panel is a concrete pad or cushion that is found between a column and a concrete floor slab to prevent the column from piercing the slab. Punching shear is so effective around column support. So to prevent this, we have to use full concrete around column support.

The thickness of the drop panel should be more generous of:

$$a) h_{min} (\text{drop panel}) = 0.25 \text{ of slab thickness without ribs}$$

$$h_{dpmin} = 0.25 \times 36 \text{ cm} = 9 \text{ cm}$$

b) drop panel thickness should be at the same level with thickness of rib so

$$h_{dp}(min) = 47 \text{ cm}$$

So, according to this calculation, the minimum drop panel thickness should be 47 cm.

Drop panel dimension is calculated as below:

The dimension of drop panels is found by extending 1/6 of the span length on each side from the centerline.

$$L_{1 dp (min)} = \frac{1}{6} \times 1200 + \frac{1}{6} \times 1200 = 400 \text{ cm}$$

$$L_{1 dp (min)} = \frac{1}{6} \times 1000 + \frac{1}{6} \times 1000 = 333 \text{ cm}$$

We use a drop panel of 400 cm by 400 cm in dimension for interior columns, and for edge and corner column, we are going to use half of this dimension.

3.12.3 Middle strip and column strip

After sketching the waffle slab in SAFE and entered all the information related to waffle slab and drop panel and then entered the gravity loads and significant rigidity factor, then middle strip and column method are used to find the internal loads in the slab. To perform this, we need to find first the dimension of strips and then to draw it in SAFE and later analyzing it. Column strip width is equal to one-fourth of the shorter span. Moreover, the remaining width between column strips is the middle strip. I drew column strip and a middle strip along the X and Y direction individually. Middle strips for X and Y direction is given Figure 3.8 and Figure 3.9 below.

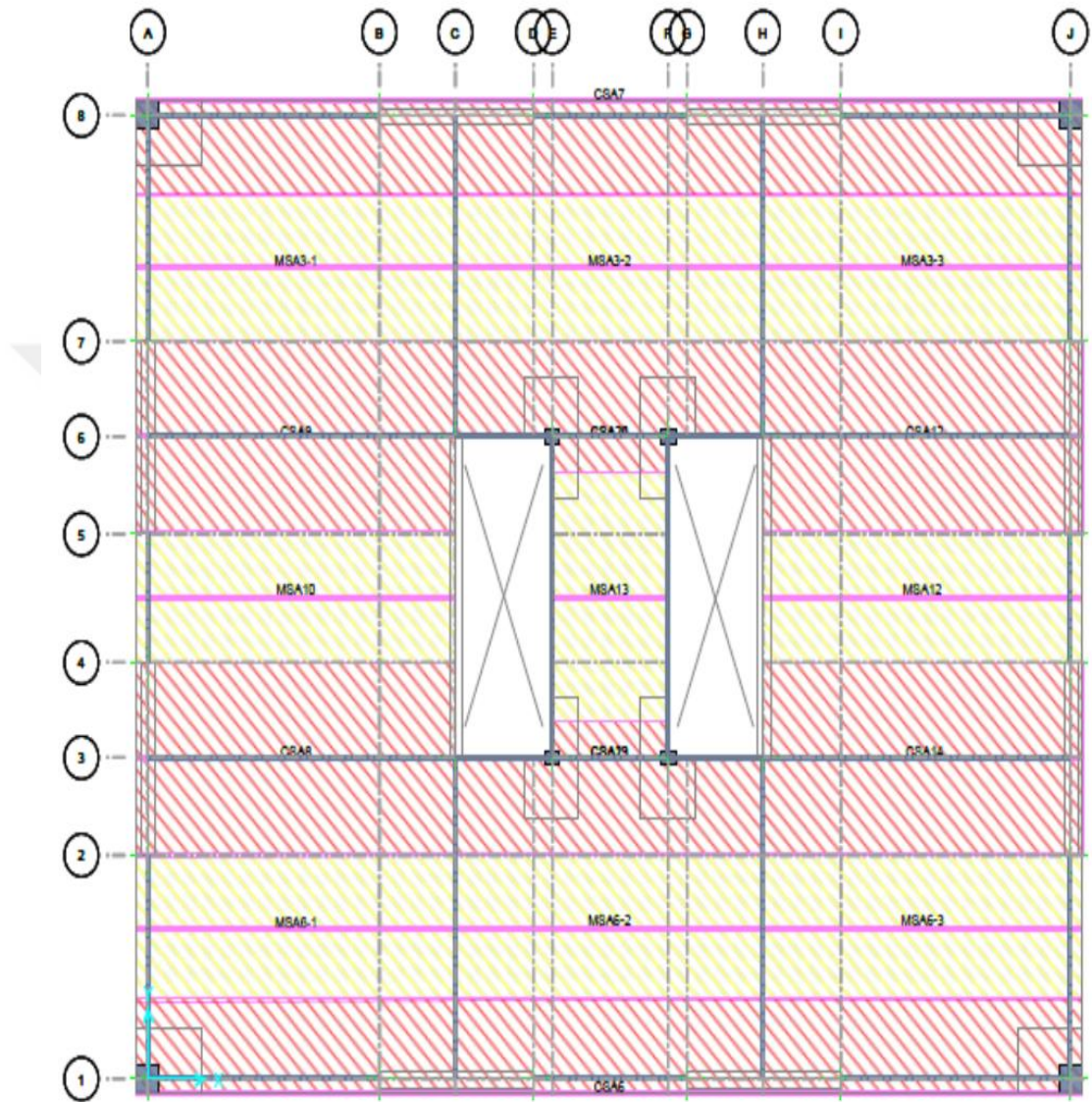


Figure 3.8 : Middle and Column strips for layer A in the X direction

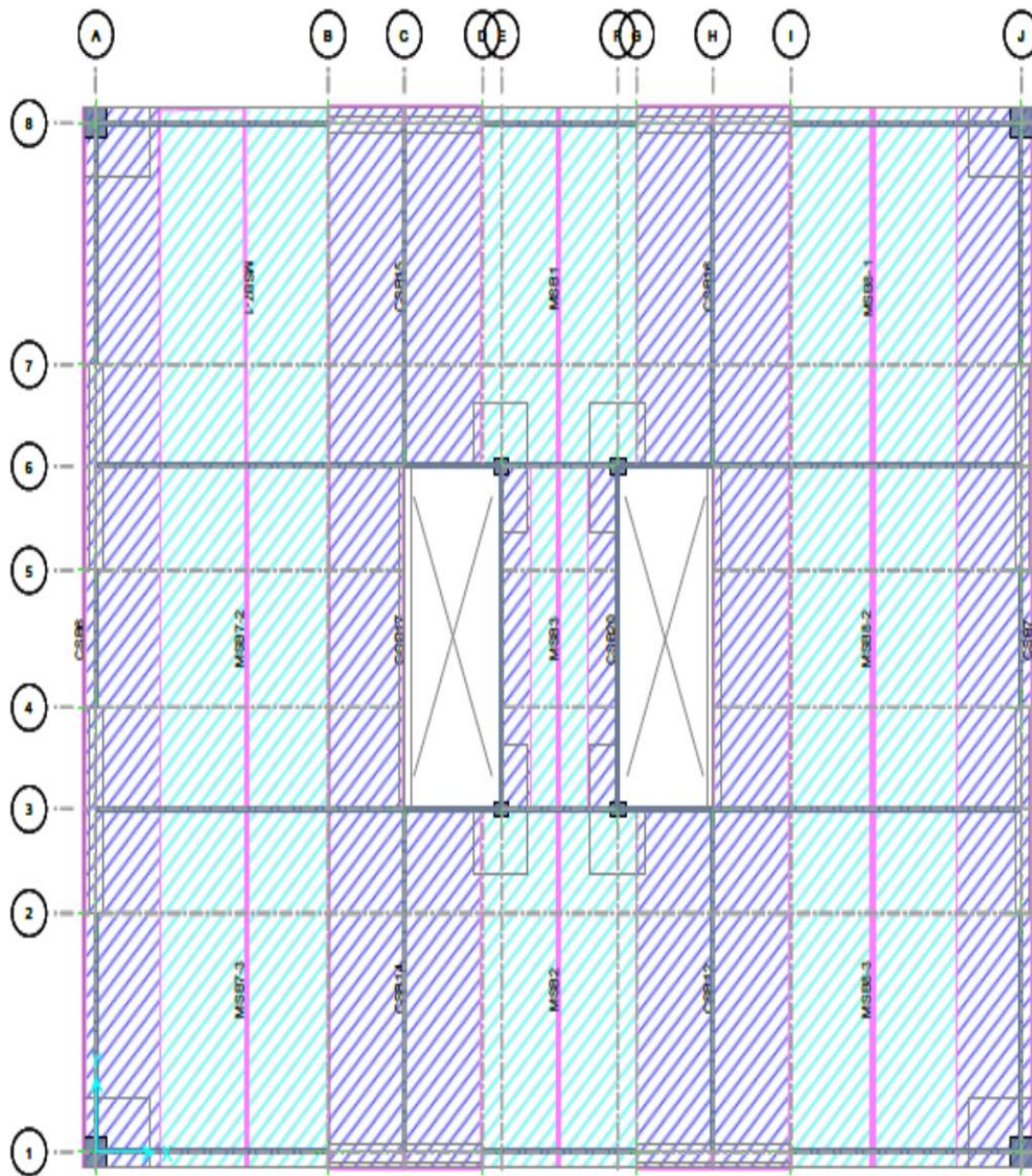


Figure 3.9 : Middle and Column strips for layer B in Y direction

3.12.4 Load combination

The default load combination of the SAFE program is used, which is only for gravity loads. Gravity loads are dead load, live load, and superimposed dead load.

Dead load (DL)

Live load (LL)

Superimposed dead load (SDL)

$$\text{Load combination 1} = 1.4 (D + SDL) \quad (3.26)$$

$$\text{Load combination 1} = 1.2 (D + SDL) + 1.6 LL \quad (3.27)$$

As we see in the figure, the Strip moment in X and Y direction. After the analysis is done by SAFE, design is also done directly with SAFE, and rebar detailing is done with the AUTOCAD program. Which is given in Appendix.

However, as a check of capacity, the capacity check done for the middle strip and column strip below.

3.12.5 Middle strip capacity check

First, we will check the strip MSA6 for layer A

$$\text{Middle strip length} = 12 \text{ m}$$

$$\text{Middle strip width} = 10 - \frac{1}{4} \times 2 \times 10 = 5 \text{ m}$$

$$\text{Strip positive moment} = 240 \text{ kN.m}$$

$$\text{No. of ribs} = \frac{500 \text{ cm}}{90 \text{ cm}} = 5 \text{ sections}$$

$$\text{moment supported by one section} = \frac{240 \text{ kN.m}}{5} = 48 \text{ kN.m}$$

$$a = \frac{A_s \times f_y}{0.85 \times f_c \times be} = \frac{2 \times 154 \times 420}{0.85 \times 40 \times 900} = 4.22 \text{ mm}$$

$$M_n = A_s \times \phi \times f_y \times \left(d - \frac{a}{2}\right) \quad (3.28)$$

$$M_n = 2 \times 154 \times 0.9 \times 420 \times \left(422 - \frac{4.22}{2}\right) = 50 \text{ kN.m}$$

Now checking the capacity of MSB7, MSB8 for layer B

$$\text{Middle strip length} = 10 \text{ m}$$

$$\text{Middle strip width} = 12 - \frac{1}{4} \times 2 \times 10 = 7 \text{ m}$$

$$\text{Strip positive moment} = 315 \text{ kN.m}$$

$$\text{No. of ribs} = \frac{700 \text{ cm}}{90 \text{ cm}} = 8 \text{ sections}$$

$$\text{moment supported by one section} = \frac{315 \text{ kN.m}}{8} = 40 \text{ kN.m}$$

$$a = \frac{A_s \times f_y}{0.85 \times f_c \times be} = \frac{2 \times 154 \times 420}{0.85 \times 40 \times 900} = 4.22 \text{ mm}$$

$$M_n = A_s \times \phi \times f_y \times \left(d - \frac{a}{2}\right) \quad (3.29)$$

$$M_n = 2 \times 154 \times 0.9 \times 420 \times \left(407 - \frac{4.22}{2}\right) = 47 \text{ kN.m}$$

3.12.6 Column strip capacity check

Checking negative moment capacity of strip CSA8 for layer A

$$\text{Column strip length} = 12 \text{ m}$$

$$\text{column strip width} = 10 \times \frac{1}{4} = 2.5 \text{ m}$$

$$\text{Strip negative moment} = 385 \text{ kN.m}$$

$$\text{No. of ribs} = \frac{250 \text{ cm}}{90 \text{ cm}} = 3 \text{ sections}$$

$$\text{moment supported by one section} = \frac{385 \text{ kN.m}}{3} = 128 \text{ kN.m}$$

$$a = \frac{A_s \times f_y}{0.85 \times f_c \times be} = \frac{7 \times 154 \times 420}{0.85 \times 40 \times 150} = 88 \text{ mm}$$

$$M_n = A_s \times \phi \times f_y \times \left(d - \frac{a}{2}\right) \quad (3.30)$$

$$M_n = 7 \times 154 \times 0.9 \times 420 \times \left(423 - \frac{88}{2}\right) = 170 \text{ kN.m}$$

Checking negative moment capacity of strip CSB14 for layer B

$$\text{Column strip length} = 10 \text{ m}$$

$$\text{column strip width} = 10 \times \frac{1}{4} = 2.5 \text{ m}$$

$$\text{Strip negative moment} = 310 \text{ kN.m}$$

$$\text{No. of ribs} = \frac{250 \text{ cm}}{90 \text{ cm}} = 3 \text{ sections}$$

$$\text{moment supported by one section} = \frac{310 \text{ kN.m}}{3} = 104 \text{ kN.m}$$

$$a = \frac{A_s \times f_y}{0.85 \times f_c \times b \times e} = \frac{6 \times 154 \times 420}{0.85 \times 40 \times 150} = 76 \text{ mm}$$

$$M_n = A_s \times \phi \times f_y \times \left(d - \frac{a}{2}\right) \quad (3.31)$$

$$M_n = 6 \times 154 \times 0.9 \times 420 \times \left(420 - \frac{76}{2}\right) = 133 \text{ kN.m}$$

3.13 Seismic performance Evaluation

In the design of the building, according to the Turkish New Building Code 2018, we have three steps to follow, which is the first step is the design of a high-rise building then evaluation of performance against earthquake. Our building with Seismic Design Category (DTS) of 1 and has a height of more than 70 m goes under the class of high-rise buildings. In the first step, we designed a building according to the Strength-based design method (DGT) then we have to check its performance against earthquakes. Performance checking can be done in 2 steps.

In the second step, we have to check the performance of building under the DD-4 earthquake level which is given in Figure 3.10 for Immediate Occupancy (KK) or Limited Damage (SH) performance levels. As our building is tall and has a DTS of 1 so according to table 3.4 b of Turkish New Building Code 2018, we have to check its performance for Immediate Occupancy (KK) under the DD-4 earthquake level.

In the third step, we have to check the performance of building under the DD-1 earthquake level for Controlled Damage (KH) or Collapse prevention (GO) performance level. As our building is tall and has DTS of 1, so according to table 3.4 b of the Turkish Building Seismic Code 2018, we have to check its performance for Collapse prevention (GO) under the DD-1 earthquake level.

3.13.1 Immediate occupancy (KK) performance level

In this step, as we mentioned before, we are going to check the performance of the building under the DD-4 performance level. In this step, we use Effective Rigidity factors, which are given in table 13.1 of Turkey New Building Code 2018. we have to ignore the eccentricity effect and should change the damping ratio from 5% to 2.5%. To find the internal loads in each member of the system, we give R and D values 1. In this performance level to find the internal loads, we use the method of Linear Modal Analysis. After we analyzed the building by the method of Linear Modal analysis, we take the result and check the performance of the building. In the modal analysis, we use the SRSS method for finding resultant internal load with a damping ratio of 2.5%, which is half of the design damping ratio.

After we found internal loads, then for high ductility elements, the ratio of internal loads over the capacity of that element should be less than 1.5 it means $E/K = 1.5$ or less than it.

If one element is out of this limit, then we have to change the section dimension and repeat the first step, which is design again, and then we check its performance.

First, we have to find the DD-4 spectrum, so we need some data to plot it. Data is taken from Turkey Seismic Hazard Map (TDTH) after entering the coordinates of our building. The Table 3.7 is taken from the TDTH map. By using Excel, I made a spectrum of DD-4 which is given in Figure 3.10 and Table A.3 then defined it to the ETABS software and analyzed the building.

Table 3.8 : Table of values taken from AFAD page for DD-4

Ss	0.295
S1	0.074
Fs	1.3
F1	1.5
SDs	0.3835
SD1	0.111
TA	0.0579
TB	0.2894
R	1
D	1

To check the performance of building for Immediate Occupancy under DD-4 level of the earthquake we need to find the section capacity of beam, column, and shear wall. For this, I used the Xtract program, which is a section analyzer program to find the capacity of sections. In this program, section dimension is entered, material property, and top, and bottom rebar. Materials property that I entered to the Xtract program is unconfined concrete, confined concrete, and steel property. For the strength of these materials, expected materials strength is defined, which is recommended by the Turkish Building Seismic Code 2018.

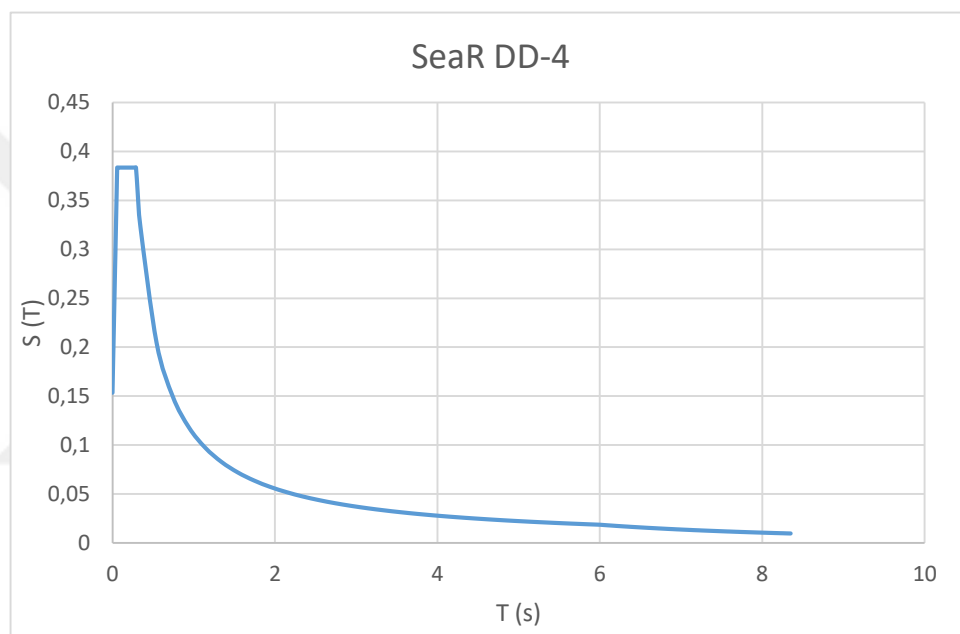


Figure 3.10 : Reduced elastic DD-4 acceleration Spectrum

3.13.2 Beam performance check for immediate occupancy (KK)

To check the performance of beams first, I find the moment capacity of all similar beams by using the Xtract program. After entering all data related to section dimension, material and rebar, I took moment capacity as a result which is given in Figure 3.11, 3.12 and 3.13. Now, as an example, I checked the performance of a beam step by step, and the result of all beam performance checking data is given in a table. Because earthquake load causes a moment at the support section and as the earthquake direction change moment at the support section of the beam also changes from negative

to positive. As a result, we need to check the moment capacity of both the bottom and top sections of the beam.

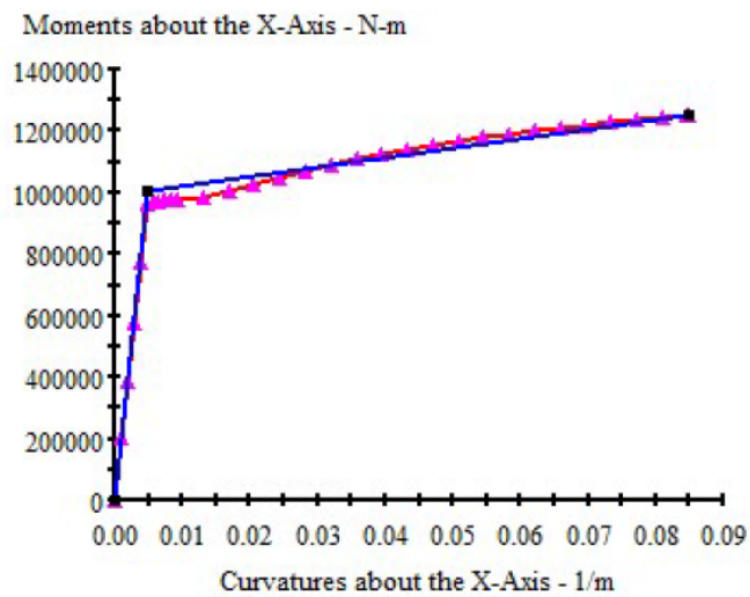


Figure 3.11 : Positive moment-curvature curve of Beam B5 taken from Xtract

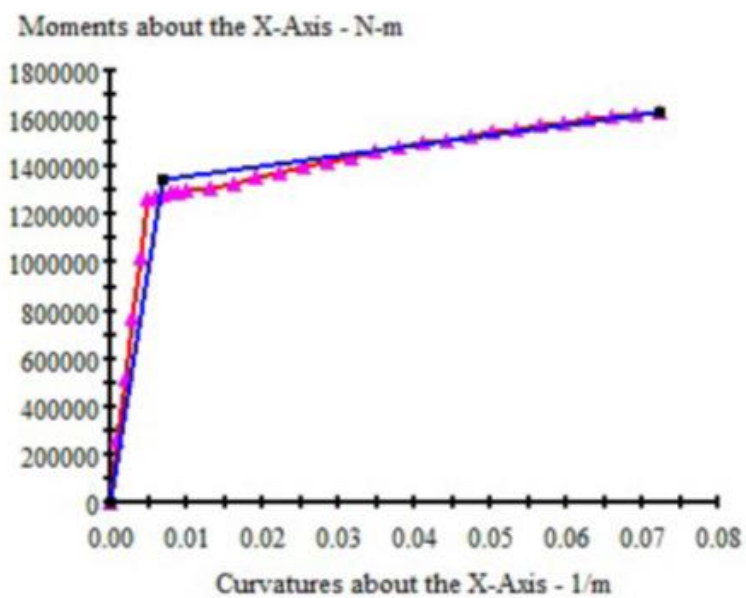


Figure 3.12 : Negative moment-curvature curve of Beam B5 taken from Xtract

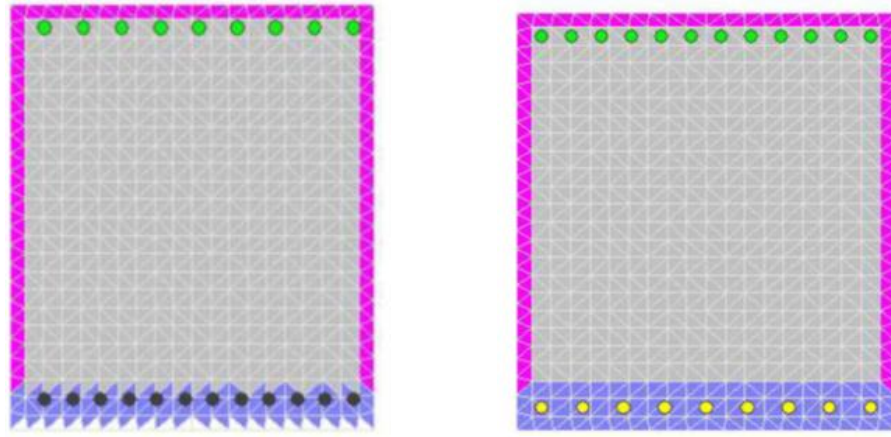


Figure 3.13 : Beam B5 section for positive and negative moment

Beam name = B5

$$h = 70 \text{ cm} \quad L = 12 \text{ m}$$

$$b = 60 \text{ cm}$$

$$\text{top bar number} = 12 \text{ } \phi 22$$

$$\text{bottom bar number} = 9 \text{ } \phi 22$$

$$\text{Effective negative moment capacity} = 1351 \text{ kN.m}$$

$$\text{Effective positive moment capacity} = 1000 \text{ kN.m}$$

$$\text{moment EQ from DD - 4} = 276 \text{ kN.m}$$

$$\text{moment vertical loads } G + Q = 705 \text{ kN.m}$$

For negative moment equation (3.31) is used.

$$E/I = \frac{\text{Moment EQ (DD - 4)}}{\text{Capacity} - \text{Moment Vertical loads}} \quad (3.31)$$

$$E/I = \frac{276}{1351 - 706} = 0.824 < 1.5$$

For a positive moment equation (3.32) is used.

$$E/I = \frac{\text{Moment EQ (DD - 4)}}{\text{Capacity} - \text{Moment Vertical loads}} \quad (3.32)$$

$$E/I = \frac{276}{1000 - 706} = 1.15 < 1.5$$

For all other beams, the performance checking is given in Table 3.8. As we see in the calculation above and also the Table 3.8, all the results are less than 1.5, so it means all beams pass the performance check for Immediate Occupancy (KK). All beams under a load of DD-4 level do not cross the limit of (KK), and it means the beams will be safe under DD-4 level earthquake load.

3.13.3 Column performance check for immediate occupancy (KK)

Now in this part, checking the performance of columns for (KK) under DD-4 earthquake level load. Because columns are more sensitive to shear load than the flexural moment, so we are going to check performance under shear load. First, we need to find the shear capacity of the column section and then taking the result of the load on the column after analyzing the building in ETABS under DD-4 earthquake load. Then the ratio of the internal shear load over the capacity of the section for shear will give us a number which should be less than 1.5 or in other words $E/K = 1.5$ or less than it.

Shear capacity of the concrete columns can be found from a combination of the concrete itself and traverse reinforcement shear capacity as we see in equation (3.33), (3.34) and (3.35) given below.

$$V_r = V_c + V_w \quad (3.33)$$

$$V_c = 0.8 \times V_{cr} \quad (3.34)$$

$$V_{cr} = 0.65 \times f_{ctd} \times b \times d \times \left(1 + 0.07 \times \frac{N_d}{A_c}\right) \quad (3.35)$$

$$V_{cr} = 0.65 \times 1.46 \times 900 \times 850 \times \left(1 + 0.07 \times \frac{(6668 + 2354) \times 10^3}{(900 \times 900)}\right) \times 10^{-3}$$

$$V_{cr} = 1325 \text{ kN}$$

$$V_c = 0.8 \times V_{cr} = 1060 \text{ kN}$$

We have six legs of 12 mm transverse reinforcement, so the area of one leg is 113 mm^2 and then multiplying it by 6.

$$\rho_{sh} = \frac{A_{sh}}{s \times b_k} = \frac{6 \times 113 \text{ mm}^2}{100 \text{ mm} \times (900 \text{ mm} - 2 \times 30 \text{ mm})} = 0.008071$$

$$V_w = \rho_{sh} \times A_c \times f_{ye} \quad (3.36)$$

$$V_w = 0.008071 \times 900 \times 900 \times 462 \times 10^{-3} = 2378 \text{ kN}$$

$$V_r = 2378 + 1060 = 3439 \text{ kN}$$

$$V_{EQ}(DD - 4) = 153 \text{ kN}$$

Table 3.9 : Table of Beam Performance check for DD-4 earthquake level load

Beams Performance Check for DD-4 Earthquake Level Load															
Stories	Name Beam	Story	Beam Direction	Station	EQ Direction	Top reinforcement		Bottom reinforcement		EQ Moment	Moment vertical Loads	Negative Moment Capacity	Positive Moment Capacity	Ratio for Negative moment	Ratio for Positivemoment
						Number	#bar	Number	#bar	kN.m	kN.m	kN.m	kN.m	E/I	E/I
1-15	B5	9	Along X	I-End	Ex	12	22	9	22	276	705	1040	945	0.824	1.15
1-15				J-End		12	22	9	22	245	618	1040	945	0.581	0.75
1-15	B2	9	Along X	I-End	Ex	12	22	6	22	495	185	1335	687	0.430	0.99
1-15				J-End		12	22	6	22	444	231	1335	687	0.402	0.97
1-15	B3	9	Along X	I-End	Ex	12	22	7	22	887	115	1335	798	0.727	1.30
1-15				J-End		12	22	7	22	893	115	1335	798	0.732	1.31
1-15	B8	9	Along X	I-End	Ex	8	22	4	22	245	187	698	357	0.479	1.44
1-15				J-End		8	22	4	22	200	85	698	357	0.326	0.74
1-15	B7	9	Along X	I-End	Ex	8	22	4	22	230	43	698	357	0.351	0.73
1-15				J-End		8	22	4	22	230	43	698	357	0.351	0.73
1-15	B1	11	Along y	I-End	Ey	12	22	6	22	435	110	1335	687	0.355	0.75
1-15				J-End		12	22	6	22	389	115	1335	687	0.319	0.68
1-15	B4	9	Along y	I-End	Ey	10	22	6	22	780	48	1118	686	0.729	1.22
1-15				J-End		10	22	6	22	776	46	1118	686	0.724	1.21
1-15	B9	13	Along y	I-End	Ey	10	22	7	22	365	488	1118	797	0.579	1.18
1-15				J-End		10	22	7	22	350	378	1118	797	0.473	0.84
1-15	B6	6	Along y	I-End	Ey	8	22	4	22	42	87	698	357	0.069	0.16
1-15				J-End		8	22	4	22	40	87	698	357	0.065	0.15
16-25	B5	16	Along X	I-End	Ex	10	24	7	24	244	705	1232	878	0.463	1.41
16-25				J-End		10	24	7	24	218	618	1232	878	0.355	0.84
16-25	B2	16	Along X	I-End	Ex	10	24	5	24	377	185	1232	632	0.360	0.84
16-25				J-End		10	24	5	24	422	231	1232	632	0.422	1.05
16-25	B3	16	Along X	I-End	Ex	10	24	5	24	723	115	1232	632	0.647	1.40
16-25				J-End		10	24	5	24	725	115	1232	632	0.649	1.40
16-25	B8	16	Along X	I-End	Ex	6	24	4	24	217	187	626	423	0.494	0.92
16-25				J-End		6	24	4	24	170	85	626	423	0.314	0.50
16-25	B7	16	Along X	I-End	Ex	6	24	3	24	197	43	626	320	0.338	0.71
16-25				J-End		6	24	3	24	197	43	626	320	0.338	0.71
16-25	B1	16	Along y	I-End	Ey	8	24	4	24	375	110	1062	546	0.394	0.86
16-25				J-End		8	24	4	24	420	115	1062	546	0.444	0.97
16-25	B4	16	Along y	I-End	Ey	10	24	5	24	683	48	1232	632	0.577	1.17
16-25				J-End		10	24	5	24	672	46	1232	632	0.567	1.15
16-25	B9	16	Along y	I-End	Ey	10	24	6	24	348	488	1232	812	0.468	1.07
16-25				J-End		10	24	6	24	361	378	1232	812	0.423	0.83
16-25	B6	16	Along y	I-End	Ey	5	24	3	24	22	87	524	320	0.050	0.09
16-25				J-End		5	24	3	24	22	87	524	320	0.050	0.09

$$V_{gravity\ loads} = 50\ kN$$

$$E/I = \frac{153\ kN}{(3439\ kN - 50\ kN)} = 0.045 < 1.5\ (OK)$$

As we see in the calculation above and Table 3.9 below, all the ratios are significantly under the allowable limit. Moreover, the maximum ratio is 0.12, as we see in the table below. So exactly, it means that columns of this building under DD-4 level earthquake load are safe and satisfy them (KK) performance level.

3.13.4 Shear wall performance check for immediate occupancy (KK)

Shear wall, which is the main part of a building. Most of the shear in a building is bear by the shear wall. As we know, vertical loads like dead load and live load do not cause too much shear force in a building. On the other hand, horizontal loads like mostly earthquake load cause shear force. This force must be bear by some mechanism in a building, and that can be done by shear and column—most of this shear bear by the shear wall. So we have to check the performance of the shear wall against the shear load. First, we need to find the capacity of the shear wall, and then we need to take out the shear load, which is coming from the DD-4 earthquake load. Then if the ratio of internal loads from DD-4 over the capacity of the shear wall against shear load will give us a number which must be less than 1.5 or $E/I = 1.5$ or less than it. To find the capacity of the shear wall, we have to insert expected materials strength into the formulas.

In here, as an example, we are going to check the performance of a shear wall for (KK) performance level.

$$Shear\ wall\ name = P1$$

$$Location = Story\ 1$$

$$Length = 600\ cm$$

$$Width = 50\ cm$$

Table 3.12 : Table of Columns performance check for DD-4 earthquake level load

Columns Performance Check for DD-4 Earthquake Level Load															
Story	Column Name	Location	h	d	Nd	Vcr	Vc	rho shear	Vw	Vr	VE DD4	V vertical loads	Ratio	limit	Check
			cm	cm	kN	kN	kN		kN	kN	kN	kN	E/I	E/I	
1	C1	1-J	90	85	9022	1325.316	1060.253	0.008071	2378.826	3439.08	153	50	0.045145	1.5	OK
	C3	4-F	45	55	4875	558.7238	446.979	0.01046	1049.923	1496.902	60	46	0.041354	1.5	OK
4	C1	1-J	90	85	7981	1256.163	1004.93	0.008071	2378.826	3383.757	124	92	0.03767	1.5	OK
	C3	4-F	45	55	4168	511.7577	409.4062	0.01046	1049.923	1459.329	127	32	0.088977	1.5	OK
8	C1	1-J	90	85	6404	1151.403	921.1222	0.008071	2378.826	3299.949	154	94	0.048036	1.5	OK
	C3	4-F	45	55	3235	449.7786	359.8228	0.01046	1049.923	1409.745	158	42	0.115519	1.5	OK
12	C1	1-J	90	85	4769	1042.79	834.2317	0.008071	2378.826	3213.058	164	103	0.052732	1.5	OK
	C3	4-F	45	55	2411	395.0402	316.0322	0.01046	1049.923	1365.955	154	48	0.116848	1.5	OK
16	C2	1-J	80	75	3217	783.1053	626.4842	0.00916	2131.418	2757.902	134	92	0.050264	1.5	OK
	C3	4-F	45	55	1674	346.0813	276.8651	0.01046	1049.923	1326.788	135	52	0.1059	1.5	OK
20	C2	1-J	80	75	1745	685.3204	548.2563	0.00916	2131.418	2679.674	115	91	0.044424	1.5	OK
	C3	4-F	45	55	975	299.6468	239.7174	0.01046	1049.923	1289.64	114	54	0.09226	1.5	OK
24	C2	1-J	80	75	331	591.3883	473.1107	0.00916	2131.418	2604.528	137	142	0.055634	1.5	OK
	C3	4-F	45	55	300	254.8065	203.8452	0.01046	1049.923	1253.768	100	54	0.083349	1.5	OK

$$f_{ctd} = 1.46 \text{ MPa}$$

$$f_{ye} = 1.1 f_{yk} = 1.1 \times 420 = 462 \text{ MPa}$$

$$\text{travers reinforcement} = \emptyset 14/10\text{cm}$$

$$\rho_{sh} = \frac{2 \times 1.54 \text{ cm}^2}{10 \text{ cm} \times 44 \text{ cm}} = 0.0077$$

$$V_r = A_{ch} \times (0.65 \times f_{ctd} + \rho_{sh} \times f_{ye}) \quad (3.37)$$

$$V_r = 50 \text{ cm} \times 600 \text{ cm} \times (0.65 \times 1.46 \text{ MPa} + 0.0077 \times 462 \text{ MPa})$$

$$V_r = 13520 \text{ kN}$$

$$V_E(DD - 4) = 1620 \text{ kN}$$

$$V_{\text{vertical loads}} = 50 \text{ kN}$$

$$E/I = \frac{1620}{13520 - 50} = 0.12 < 1.5 \text{ (OK)}$$

As we see in the calculation above and also Table 3.10, which satisfies the (KK) performance level and does not cross the limit. The maximum ratio for the shear wall is 0.18, which is significantly less than 1.5, and it means all the shear wall will be safe under the DD-4 level of earthquake.

As we see all beams, columns, and shear walls do not cross the limit. It shows that all structural elements of this building are safe under the DD-4 earthquake level load. If a weak earthquake happens, this building will be safe under that load, and Immediate Occupancy can be provided. So our building passes the second step, which was an analysis of building under the DD-4 earthquake load.

Table 3.15 : Shear wall performance check for DD-4 earthquake level load

Shear Wall Performance Check for DD-4 Earthquake Level Load														
Story	Pier Label	b	h	Ach	fctd	fye	fce	rho shear	Vr	VE DD4	V vertical loads	ratio	Limit	Check
		cm	cm	cm ²	Mpa	Mpa	Mpa		kN	kN	kN	E/I	E/I	
1	P1	50	600	30000	1.46	462	52	0.0077	13519.2	1620	35	0.12014061	1.5	OK
	P5	50	600	30000	1.46	462	52	0.0077	13519.2	2406	39	0.178484	1.5	OK
	P9	50	1000	50000	1.46	462	52	0.0077	22532	3998	222	0.17920215	1.5	OK
4	P1	50	600	30000	1.46	462	52	0.0077	13519.2	1255	75	0.0933488	1.5	OK
	P5	50	600	30000	1.46	462	52	0.0077	13519.2	1891	95	0.140865	1.5	OK
	P9	50	1000	50000	1.46	462	52	0.0077	22532	3910	388	0.17657153	1.5	OK
8	P1	50	600	30000	1.46	462	52	0.0077	13519.2	1145	70	0.08513518	1.5	OK
	P5	50	600	30000	1.46	462	52	0.0077	13519.2	1434	90	0.10678224	1.5	OK
	P9	50	1000	50000	1.46	462	52	0.0077	22532	2670	359	0.12041672	1.5	OK
12	P1	50	600	30000	1.46	462	52	0.0077	13519.2	1065	66	0.07916332	1.5	OK
	P5	50	600	30000	1.46	462	52	0.0077	13519.2	1265	87	0.09417668	1.5	OK
	P9	50	1000	50000	1.46	462	52	0.0077	22532	2060	339	0.09282206	1.5	OK
16	P1	45	600	27000	1.46	462	52	0.0088	13539.42	962	72	0.07143165	1.5	OK
	P5	45	600	27000	1.46	462	52	0.0088	13539.42	1080	90	0.08030086	1.5	OK
	P9	45	1000	45000	1.46	462	52	0.0088	22565.7	1780	325	0.08003345	1.5	OK
20	P1	45	600	27000	1.46	462	52	0.0088	13539.42	816	76	0.06060867	1.5	OK
	P5	45	600	27000	1.46	462	52	0.0088	13539.42	907	92	0.06744788	1.5	OK
	P9	45	1000	45000	1.46	462	52	0.0088	22565.7	1590	316	0.07146164	1.5	OK
24	P1	45	600	27000	1.46	462	52	0.0088	13539.42	473	106	0.03521069	1.5	OK
	P5	45	600	27000	1.46	462	52	0.0088	13539.42	440	128	0.03280786	1.5	OK
	P9	45	1000	45000	1.46	462	52	0.0088	22565.7	905	402	0.04083253	1.5	OK

3.13.5 Collapse prevention (GO) performance level

This is the last step for the design and performance evaluation of a high-rise building. For the design of a building under seismic load, it should be designed in such a way that it does not collapse in a massive earthquake. The collapse prevention performance level is associated with the onset of total or partial collapse. At this level, the corresponding structural damage is essential, but with high enough resistance to gravity load. Generally, the building damage is severe in this level. In this level, building performance mainly relates to the vertical load-carrying system, and the structure needs to stable under vertical loads only. The building suffers large permanent drifts; however, load-bearing column and walls function.

Hinges and extensive cracking develop in ductile elements, but the design should be in such a way that these hinges first occur in beams, not in columns. This phenomenon is because to prevent collapse. Aftershock activities can cause the building to collapse, so it just gives the people who live there to escape from the building and save their life.

Table 3.17 : Values taken from AFAD for DD-1 earthquake level spectrum

Ss	2.582
S1	0.739
Fs	1.2
F1	1.4
SDs	3.0984
SD1	1.0346
TA	0.0668
TB	0.3339
R	1
D	1

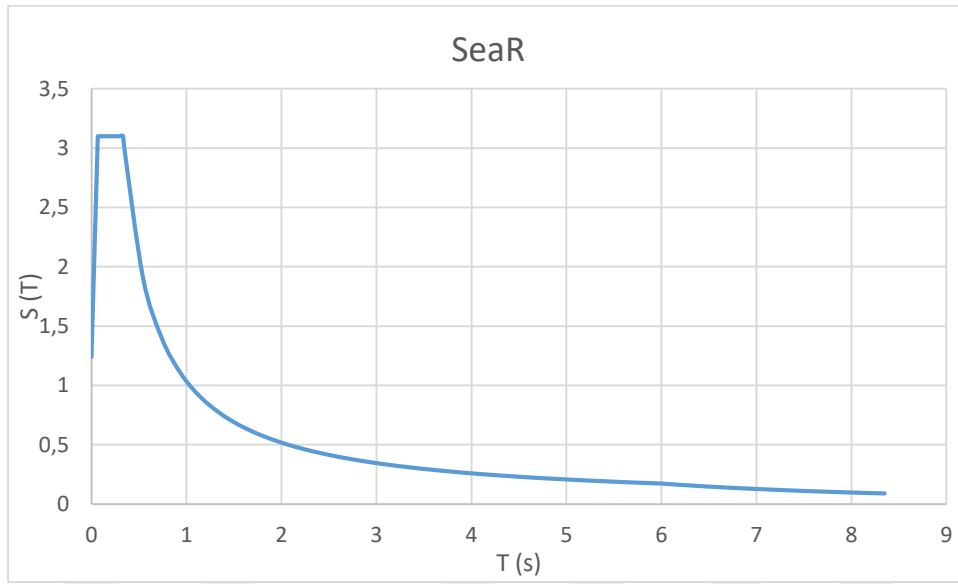


Figure 3.14 : Elastic DD-1 acceleration spectrum.

Our building goes under the category of high rise building, and according to table 3.4 (b) of the Turkish New Building Code 2018, it must be checked for the performance level of Collapse Prevention (GO). The structure must be analyzed for a 2×11 seismic record, and then the result will be an average of 22 seismic record results. For this analysis, we use a damping ratio of 2.5%, and eccentricity should be ignored. The DD-1 earthquake level must scale these 22 seismic records.

For materials strength, we use expected material strength f_{ye} and f_{ce} , and under this seismic load, we apply nonlinear time history analysis.

After this, we are going to find the plastic hinge rotation limit and maximum strain limit for concrete and steel. Then we are comparing these values with limits for each element of the building, and all of them must be under this limit to be safe and do not collapse under large earthquakes.

First of all, we are taking DD-1 data for our building coordinate from the AFAD page, which is given in Table 3.10 and then by using an excel sheet to plot the DD-1 acceleration spectrum as we see in Figure 3.14 above. For this spectrum, we take R and D values 1.

3.13.6 Peer ground motion database

For nonlinear time history analysis of structure, we need a 2×11 seismic record. Every two seismic record is orthogonal to each other, which is H1 and H2 in the peer database. Seismic Records selection is made according to magnitude and fault cracking type. The magnitude of selected seismic records is between 6.5 and 7.5, which is defined as a large earthquake happens in Istanbul and adjacent provinces. Table 3.12 is 2×11 peer seismic records with all details.

Table 3.18 : Peer strong ground motion seismic records

PEER Strong Ground Motion Seismic 2x11 Records							
No	Index	Magnitude	Location	fault type	year	Station	Vs 30
1	143	7.35	Tabas, Iran	Reverse	1978	Tabas	766
2	1602	7.14	Duzce, Turkey	Strike	1999	Bolu	293
3	1787	7.13	Hector Mine	Strike	1999	Hector	726
4	162	6.95	Imperial Valley-02	Strike	1940	El-Centro Array#9	213.4
5	1111	6.9	Kobe, Japan	Strike	1995	Nishi Akashi	609
6	1148	7.51	Kocaili Turkey	Strike	1999	Arcelik	523
7	900	7.28	Landers	Strike	1992	Yermo Fire Station	354
8	864	7.28	Landers	Strike	1992	Joshua Tree	379
9	22	6.8	El-Alamo	Strike	1956	El-Centro Array#9	213
10	12	7.36	Kern Country	Reverse	1952	LA-Hollywood	316
11	280	7.2	Trinidad	Strike	1980	Rio-Dell Overpass	312

These 2x11 seismic records just loaded to ETABS as time history function and each named as their names were in the PEER database. After then, all these records scaled by the target spectrum, which DD-1 level earthquake spectrum. In this case of study, just horizontal ground motions are considered, and vertical ground motions are ignored. Scaling of these seismic records done by the method of Frequency. After all of them scaled, for each record, a nonlinear time history function is made, and the building analyzed under these functions, and then the average of these 22 results is taken as a final result.

3.13.7 Defining materials

To define hinges for beams, columns, and shear walls, we need to find the moment-curvature curve of these elements. To find these curves, I used the Xtract program. After plotting these curves for similar members, I uploaded these data to the ETABS program. In ETABS, there is an option in Defining Nonlinear Hinges. In that option, we need to give moment-curvature curve data and negative and positive capacity of sections. Moreover, to have the moment curvature curve of sections, we first need to define the material of section. Three materials are defined in the Xtract program;

- Unconfined Concrete Material
- Confined Concrete Material
- Steel Material

Unconfined Concrete material is concrete which does not have transverse reinforcement. In the Xtract Program, we usually define unconfined concrete for covering concrete because it is not inside circumference of transverse reinforcement. For this concrete, we give the expected concrete strength as maximum compressive strength and strain of 0.002. unconfined concrete will fail at a strain of 0.005, and the strength of concrete will drop to zero and fails. The unconfined concrete curve is given in Figure 3-15. In Figure 3-15 Mpa is Mega Pascal which is unit of pressure.

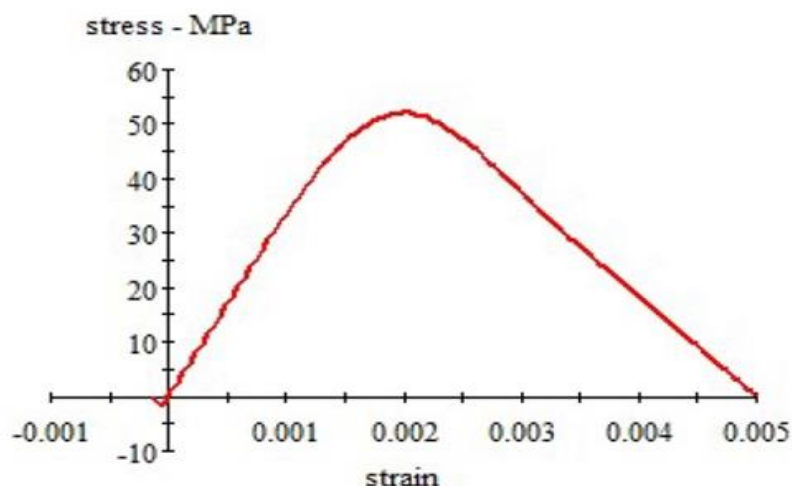


Figure 3.15 : Unconfined concrete material stress-strain curve Modelled in Xtract software

Confined concrete is concrete which surrounded by transverse reinforcement. The compressive strength of confined concrete is higher than unconfined concrete which is given in Figure 3.16. Factors that interfere in making the strength of concrete higher are transverse reinforcement size and ratio, area of longitudinal reinforcement, transverse reinforcement tensile strength, and concrete unconfined compressive strength.

In the Turkish New Building Code 2018, there are formulas to calculate the strength of confined concrete. I used these formulas to calculate confined concrete strength for beam, column and shear wall.

As we see in Table 3.13 below, the calculation is done for confined compressive strength and strain related. Average confined concrete compressive strength is 57 MPa, and strain related to this is about 0.0029 on average. To define the moment-curvature curve for beams, we need this information to insert it in Xtract and get the result.

As we see in Table 3.14 below, there are calculations done for columns. In these calculations, found the confined compressive concrete strength for columns. As we see the average compressive strength of concrete for columns is 58 MPa, and strain related to this is 0.0031.

For shear walls also this calculation is done. As a result, I found the compressive confined strength of concrete. This strength is 78 MPa in value, and strain related to this is 0.00696 on average. After defining these values to Xtract, we can get confined concrete material for our sections.

The behavior of reinforcing steel may control the response of reinforced concrete structural elements subjected to earthquake loading. Thus, it is necessary to develop an analytical model that predicts the fundamental characteristics of steel within a range of loading that is appropriate for these structural systems.

Typical load histories for reinforcing steel follow from a consideration of the observed response of reinforced concrete structural elements. Characteristics of steel response are established through laboratory testing of steel coupons.

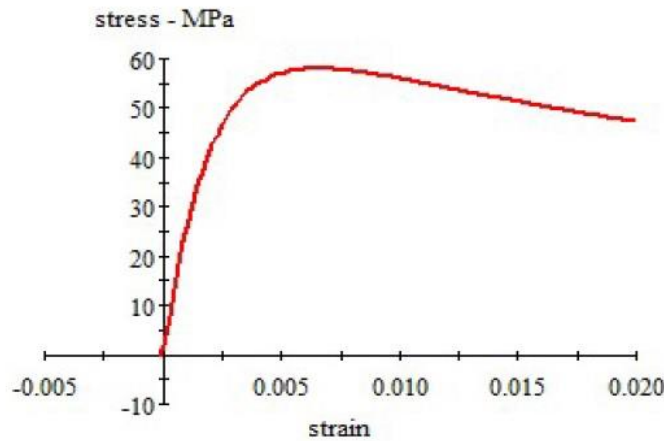


Figure 3.16 : Confined concrete C40 material stress-strain curve taken from Xtract

Previous research provides theories and techniques for the development of an analytical model that predicts steel response. Experimental data provide information for final calibration and refinement of the proposed model. Experimental data define the response of reinforcing steel subjected to uniaxial, pseudo-static monotonic tensile loading, cyclic tensile loading and reversed cyclic loading. This information is extended by monotonic and cyclic testing of reinforcement at variable load rates.

1. The initial response is linear-elastic for stress demand less than the initial yield strength.
2. For strain demand exceeding that corresponding to the initial yield strength, there is a slight drop in strength below the initial yield strength. Strength is maintained at this lower yield strength for a moderate increase in strain demand. This range of response is referred to as the yield plateau or the Lüders plateau, and the material yield strength typically is defined to be the average strength for loading within this Strain range. Stress and strain curve of steel reinforcement is given Figure 3-17.

Table 3.19 : Confined concrete compressive strength for beams, columns and shear wall

Confined Concrete Compressive Strength for Beams

Beam name	Direction	b	h	a1	a2	s	Bar size	number bar	As	ke	shear Ash	rho shear	fyw	fex and fey	fco	lamda c	epsilon co	fcc	epsilon cc
		cm	cm	cm	cm	cm	mm		mm2		cm2		Mpa		Mpa		mm/mm		mm/mm
B5	X	60	70	50	60	15	22	21	7982.8	0.528108	2.26	0.00279012	420	0.61886482	52	1.080291	0.002	56.18	0.00280
B2	X	50	75	40	65	15	22	18	6842.4	0.489715	2.26	0.00342424	420	0.70429869	52	1.091016	0.002	56.73	0.00291
B3	X	50	75	40	65	15	22	19	7222.5	0.490394	2.26	0.00342424	420	0.70527517	52	1.091138	0.002	56.74	0.00291
B8	X	45	60	35	50	15	22	12	4561.6	0.473361	2.26	0.00386325	420	0.76805866	52	1.098967	0.002	57.15	0.00299
B7	X	45	60	35	50	15	22	12	4561.6	0.473361	2.26	0.00386325	420	0.76805866	52	1.098967	0.002	57.15	0.00299
B1	Y	50	75	40	65	15	22	18	6842.4	0.489715	2.26	0.00342424	420	0.70429869	52	1.091016	0.002	56.73	0.00291
B4	Y	50	75	40	65	15	22	16	6082.1	0.488362	2.26	0.00342424	420	0.70235383	52	1.090773	0.002	56.72	0.00291
B9	Y	50	75	40	65	15	22	17	6462.3	0.489038	2.26	0.00342424	420	0.70332492	52	1.090894	0.002	56.73	0.00291
B6	Y	45	60	35	50	15	22	12	4561.6	0.473361	2.26	0.00386325	420	0.76805866	52	1.098967	0.002	57.15	0.00299
Average																		57	0.0029

Confined Concrete Compressive Strength for Columns

column name	b	h	ai	s	bar size	bar number	As	ke	shear Ash	rho shear	fwy	fex	fco	λc	εco	fcc	εcc
	cm	cm	cm2	cm	mm		mm2		cm2		Mpa		Mpa		mm/mm	Mpa	mm/mm
C1	90	90	35280	10	22	22	8362.92	0.112074	9.18	0.010929	420	0.514418	52	1.067065	0.002	55.4873668	0.002671
C2	80	80	28125	10	22	18	6842.389	0.206528	7.65	0.010338	420	0.896724	52	1.114874	0.002	57.9734416	0.003149
C3	45	60	12100	10	22	10	3801.327	0.164131	6.12	0.015692	420	1.081749	52	1.137435	0.002	59.1466393	0.003374
Average																58	0.0031

Confined Concrete Compressive Strength for Shear Walls

shear wall name	b	h	ai	s	As	ke	shear Ash	rho shear	fwy	fex	fco	λc	εco	fcc	εcc
	cm	cm	cm2	cm	cm2		cm2		Mpa		Mpa		mm/mm	Mpa	mm/mm
P1	50	600	29040	10	220.6	0.722239	6.15	0.013977	420	4.239871	52	1.475958	0.002	76.7498	0.00676
P9	50	1040	29040	10	377	0.794826	6.15	0.013977	420	4.665989	52	1.516146	0.002	78.8396	0.007161
Average														78	0.00696

3. Increasing strain demand results in increased strength. This strain-hardening regime is maintained to a peak strength that typically exceeds the yield strength by thirty to sixty per cent
4. At severe tensile strain demand, reinforcement begins to neck, and strength is reduced.
5. At a maximum strain demand, the steel reinforcement fractures and load capacity is lost.

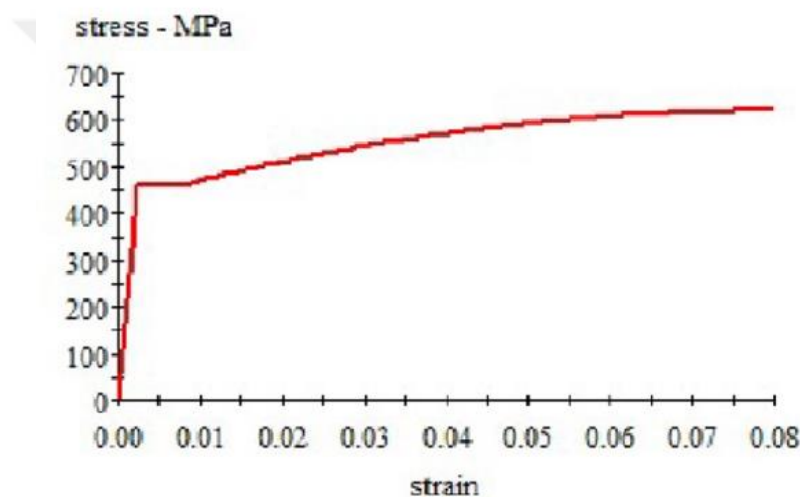


Figure 3.17 : Stress-strain curve of steel S420a modelled in Xtract software

3.13.8 Effective rigidity factor

After the load is applied in an element, then will happen in tension part of the concrete section. As crack happens, then rigidity of section which EI decrease in an amount. As we need to use effective rigidity factor for the design of elements which is given in the table in code, like this, if we are analyzing the building for nonlinear time history analysis, we need to find effective rigidity factor of the beam, column, and shear wall. After we find these values, we are inserting this in the property modifier of the section in ETABS.

Now at first, calculation is done for a beam effective rigidity factor as an example, and the other beams are given in Table 3.14 given below.

$$\text{Beam name} = B5$$

$$h = 70 \text{ cm} \quad b = 60 \text{ cm}$$

$$L = 12 \text{ m} \quad L_s = 6 \text{ m} \quad (\text{which is } \frac{1}{2} \text{ of beam length for beams})$$

$$\phi_y = 0.004972 \quad \text{taken from Xtract analysis of section}$$

$$f_{ye} = 462 \text{ MPa}$$

$$f_{ce} = 52 \text{ MPa}$$

$$d_b = 22 \text{ mm}$$

$$\eta = 1 \quad \text{for beams}$$

$$M_y = 1267 \text{ kN.m} \quad \text{taken from Xtract analysis}$$

$$E_c = 34000 \text{ MPa}$$

$$\theta_y = \frac{\phi_y L_s}{3} + 0.0015 \eta \left(1 + 1.5 \frac{h}{L_s} \right) + \frac{\phi_y d_b f_{ye}}{8 \sqrt{f_{ce}}} \quad (3.38)$$

$$\begin{aligned} \theta_y &= \frac{0.004972 \times 6}{3} + 0.0015 \times 1 \times \left(1 + 1.5 \frac{0.7}{6} \right) + \frac{0.004972 \times 22 \times 462}{8 \sqrt{52}} \\ &= 0.01258 \text{ rad} \end{aligned}$$

$$(EI)_e = \frac{M_y}{\theta_y} \times \frac{L_s}{3} = \frac{1267 \text{ kN.m}}{0.01258 \text{ rad}} \times \frac{6 \text{ m}}{3} = 201390.8 \text{ kN/m}^2$$

$$EI = 583100 \text{ kN/m}^2$$

$$\frac{(EI)_e}{EI} = 0.35 \quad (\text{effective rigidity factor})$$

For all similar beams, this calculation is done, and the effective rigidity factor is found. As we see in Table 3.14 average effective rigidity factor is 0.29. we are then entering these values into ETABS in the property modifier of section.

Now in here, calculations are done to find the effective rigidity factor for columns. The effective rigidity factor for columns is much lower than beams.

$$\text{Column name} = C1$$

$$h = 90 \text{ cm} \quad b = 90 \text{ cm}$$

$$L = 3.5 \text{ m} \quad L_s = 1.75 \text{ m} \quad (\text{which is } \frac{1}{2} \text{ of column length for columns})$$

$$\phi_y = 0.003102 \quad \text{taken from Xtract analysis of section}$$

$$f_{ye} = 462 \text{ MPa}$$

$$f_{ce} = 52 \text{ MPa}$$

$$d_b = 22 \text{ mm}$$

$$\eta = 1 \quad \text{for columns}$$

$$M_y = 1130 \text{ kN.m} \quad \text{taken from Xtract analysis}$$

$$E_c = 34000 \text{ MPa}$$

$$\theta_y = \frac{\phi_y L_s}{3} + 0.0015 \eta \left(1 + 1.5 \frac{h}{L_s} \right) + \frac{\phi_y d_b f_{ye}}{8 \sqrt{f_{ce}}} \quad (3.39)$$

$$\begin{aligned} \theta_y &= \frac{0.003102 \times 1.75}{3} + 0.0015 \times 1 \times \left(1 + 1.5 \frac{0.9}{1.75} \right) + \frac{0.003102 \times 22 \times 462}{8 \sqrt{52}} \\ &= 0.005013 \text{ rad} \end{aligned}$$

$$(EI)_e = \frac{M_y}{\theta_y} \times \frac{L_s}{3} = \frac{1130 \text{ kN.m}}{0.005013 \text{ rad}} \times \frac{1.75 \text{ m}}{3} = 131486 \text{ kN/m}^2$$

$$EI = 1858950 \text{ kN/m}^2$$

$$\frac{(EI)_e}{EI} = 0.0707 \quad (\text{effective rigidity factor})$$

For all similar columns, this calculation is done, and the effective rigidity factor is found. As we see in Table 3.14 average effective rigidity factor is 0.1 in magnitude. We are then entering these values into ETABS in the property modifier of section.

Now in here, calculation carried out to find effective rigidity factor for Shear walls. The effective rigidity factor for the shear wall is much lower than beams and columns.

Shear Wall name = P1

h = 600 cm b = 50 cm

L = 85 m L_s = 42.5 m (which is $\frac{1}{2}$ of column length for columns)

ϕ_y = 0.00635 taken from Xtract analysis of section

f_{ye} = 462 MPa

f_{ce} = 52 MPa

d_b = 20 mm

η = 0.5 for columns

M_y = 25600 kN.m taken from Xtract analysis

E_c = 34000 MPa

$$\theta_y = \frac{\phi_y L_s}{3} + 0.0015 \eta \left(1 + 1.5 \frac{h}{L_s} \right) + \frac{\phi_y d_b f_{ye}}{8 \sqrt{f_{ce}}} \quad (3.40)$$

$$\begin{aligned} \theta_y &= \frac{0.00635 \times 42.5}{3} + 0.0015 \times 1 \times \left(1 + 1.5 \frac{6}{42.5} \right) + \frac{0.00635 \times 20 \times 462}{8 \sqrt{52}} \\ &= 0.09188 \text{ rad} \end{aligned}$$

$$(EI)_e = \frac{M_y}{\theta_y} \times \frac{L_s}{3} = \frac{25600 \text{ kN.m}}{0.09188 \text{ rad}} \times \frac{42.5 \text{ m}}{3} = 3946996 \text{ kN/m}^2$$

$$EI = 306000000 \text{ kN/m}^2$$

$$\frac{(EI)_e}{EI} = 0.01289 \quad (\text{effective rigidity factor for shear wall})$$

Table 3.22 : Effective rigidity factor for beams, columns and shear walls

Beams Effective Rigidity Factor																
Beam Name	Direction	b	h	E	Length	Φ_y	Ls	η	db	fye	fce	Ele	θ_y	My	El total	Effective Rigidity Factor
		cm	cm	Mpa	m		m		mm	Mpa	Mpa	kN/m2	rad	kN.m	kN/m2	
B5	X	60	70	34000	12	0.004972	6	1	22	462	52	201390.8	0.012583	1267	583100	0.35
B2	X	50	75	34000	9	0.00481	4.5	1	22	462	52	202265	0.009937	1340	597656.3	0.34
B3	X	50	75	34000	6	0.00481	3	1	22	462	52	173576.1	0.00772	1340	597656.3	0.29
B8	X	45	60	34000	3.75	0.00606	1.875	1	22	462	52	62719.15	0.007075	710	275400	0.23
B7	X	45	60	34000	4.5	0.00606	2.25	1	22	462	52	69042.05	0.007713	710	275400	0.25
B1	Y	50	75	34000	7	0.00481	3.5	1	22	462	52	185201.3	0.008441	1340	597656.3	0.31
B4	Y	50	75	34000	4	0.00464	2	1	22	462	52	120231.7	0.006255	1128	597656.3	0.20
B9	Y	50	75	34000	10	0.00464	5	1	22	462	52	180972.1	0.010388	1128	597656.3	0.30
B6	Y	45	60	34000	10	0.00606	5	1	22	462	52	91464.03	0.012938	710	275400	0.33
															Average	0.29

Columns Effective Rigidity Factor																
column Name	b	h	E	Length	Φ_y	Ls	η	db	fye	fce	Ele	θ_y	My	El total	Effective Rigidity Factor	
	cm	cm	Mpa	m		m		mm	Mpa	Mpa	kN/m2	rad	kN.m	kN/m2		
C1	90	90	34000	3.5	0.003102	1.75	1	22	462	52	131486.9	0.005013	1130	1858950	0.07073	
C2	80	80	34000	3.5	0.00353	1.75	1	22	462	52	91816.31	0.00521	820	1160533	0.07912	
C3	45	60	34000	3.5	0.005212	1.75	1	22	462	52	42134.52	0.00623	450	275400	0.15299	
															Average	0.10095

Shear Walls Effective Rigidity Factor																
Shear wall Name	b	h	E	Length	Φ_y	Ls	η	db	fye	fce	Ele	θ_y	My	El total	Effective Rigidity Factor	
	cm	cm	Mpa	m		m		mm	Mpa	Mpa	kN/m2	rad	kN.m	kN/m2		
P1	50	600	34000	85	0.00635	42.5	0.5	20	462	52	3946996	0.091884	25600	306000000	0.012899	
P9	50	1000	34000	85	0.00364	42.5	0.5	20	462	52	20225004	0.053164	75900	1416666667	0.014276	
															Average	0.014

3.13.9 Defining nonlinear hinge

For nonlinear time history analysis of a building in the ETABS program, we need to define hinges for beams, columns, and shear walls. Plastic hinge is used to describe the deformation of a section of a beam where plastic bending occurs, and the same theory for columns and shear walls can be defined. Hinge means having no capability to resist the moment. Now for defining nonlinear plastic hinge for beams, we need to have a moment-curvature curve of beams. After entering all data of the beam into the Xtract program, this will give us the curve, and then we are entering it into ETABS.

Now first, we will define plastic hinge for beams. First analyzing all beams in Xtract program after defining material as we mentioned before and then giving all beam section information, and as a result, we have the information below.

Beam name = B5

Hinge length = 350 mm half of section depth

$$M_y(\text{negative}) = 1351 \text{ kN.m}$$

$$M_u(\text{negative}) = 1624 \text{ kN.m}$$

$$\phi_y(\text{negative}) = 0.00675 \text{ } 1/m$$

$$\phi_u(\text{negative}) = 0.0723 \text{ } 1/m$$

$$M_y(\text{positive}) = 1008 \text{ kN.m}$$

$$M_u(\text{positive}) = 1250 \text{ kN.m}$$

$$\phi_y(\text{positive}) = 0.00489 \text{ } 1/m$$

$$\phi_u(\text{positive}) = 0.0847 \text{ } 1/m$$

All of the above information about beam B5 is taken from the Xtract program after analyzing. This information can be taken from the moment-curvature curve of the beam, which is given in Figure 3.18 and Figure 3.19 below. Now to input all this information into ETABS, we need to select a scale factor and then to divide all other values upon that and to enter as a scaling value. Plastic hinge rotation limits for beams are given in Table 3.17 below.

Moment Negative SF = 1351 kN.m

Moment Positive SF = 1008 kN.m

Curvature SF for negative and positive = 1

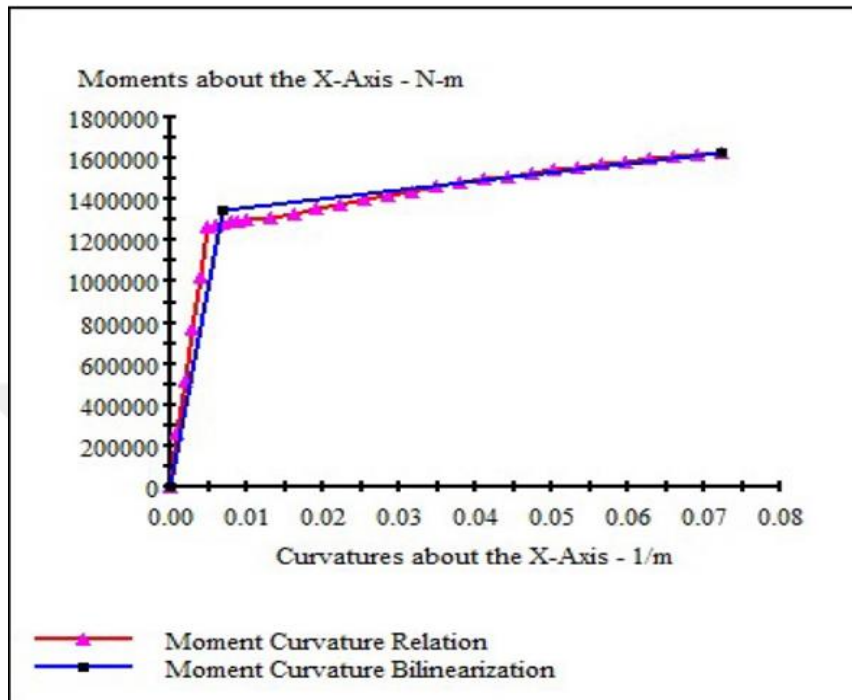


Figure 3.18 : Negative moment-Curvature curve of Beam B5 taken from Xtract

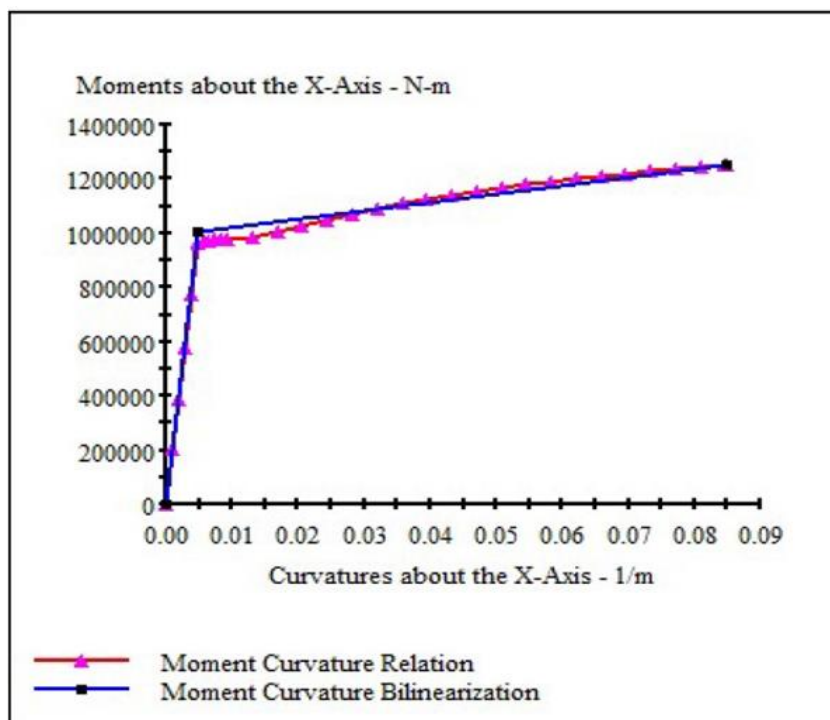


Figure 3.19 : Positive moment-curvature of Beam B5 taken from Xtract

For nonlinear plastic hinge for a beam, M3 moment hinge is defined which is given in Figure 3.20 because beam failure is only due to moment in section. Hinge is defined as we see below and then selected the options to drop to zero after the section moment reach point E.

Hinge Property Data for B5 beam hinge - Moment M3

Displacement Control Parameters

Point	Moment/SF	Curvature/SF
E-	-0.2	-0.5
D-	-0.2	-0.0723
C-	-1.202	-0.0723
B-	-1	0
A	0	0
B	1	0
C	1.24	0.0847
D	0.2	0.0847
E	0.2	0.5

Graph: Moment - Rotation

Type

☐ Moment - Rotation

☒ Moment - Curvature

Hinge Length: 350 mm

☐ Relative Length

Hysteresis Type and Parameters

Hysteresis: Isotropic

No Parameters Are Required For This Hysteresis Type

Load Carrying Capacity Beyond Point E

☒ Drops To Zero

☐ Is Extrapolated

Scaling for Moment and Curvature

☐ Use Yield Moment

Moment SF: Positive: 1008, Negative: 1351 kN-m

☐ Use Yield Curvature

Curvature SF: Positive: 1, Negative: 1 1/m

(Steel Objects Only)

Acceptance Criteria (Plastic Curvature/SF)

☒ Immediate Occupancy

0.003, -0.003

☐ Life Safety

0.012, -0.012

☐ Collapse Prevention

0.06, -0.06

☐ Show Acceptance Criteria on Plot

OK Cancel

Figure 3.20 : M3 Plastic hinge defining for Beam in ETABS program

Now I defined hinge for the column. For column, we need to define Interacting P-M2-M3 hinge type as we see in figure 3-23 and 3-24, which is suitable for columns. In this type of hinge, we need to define an interaction curve for columns and also moment-curvature curves. Moment curvature curve and interaction diagram for the column can be calculated by using the Xtract program.

As mentioned above, first all information about material and section of the column is defined, and then after analyzing, the required curve can be obtained.

Table 3.24 : Scaled interaction diagram values for columns

Column 90 x 90				Column 80 x 80				Column 60 x 45			
Values		Scaled factor		Values		Scaled factor		Values		Scaled factor	
My, Mx	P	My, Mx	P	My, Mx	P	My, Mx	P	My, Mx	P	My, Mx	P
KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN
6.91E-13	43500	1.45E-16	1	5.188E-14	34300	1.55E-17	1	4.01E-14	15100	3.33E-17	1
285.2	41300	0.059815	0.949425	173.7	32700	0.052053	0.953353	28.02	14700	0.023253	0.97351
1729	35200	0.362626	0.809195	1194	27900	0.357806	0.813411	397.9	12500	0.330207	0.827815
3894	25700	0.816695	0.590805	2719	20300	0.814804	0.591837	927.1	9091	0.769378	0.602053
4673	18900	0.980076	0.434483	3264	14900	0.978124	0.434402	1141	6569	0.946888	0.435033
4768	14500	1	0.333333	3337	11500	1	0.335277	1205	4853	1	0.321391
4532	11800	0.950503	0.271264	3174	9288	0.951154	0.270787	1163	3802	0.965145	0.251788
4253	9870	0.891988	0.226897	2979	7764	0.892718	0.226356	1100	3119	0.912863	0.206556
3989	8376	0.836619	0.192552	2794	6590	0.837279	0.192128	1040	2576	0.863071	0.170596
3750	7288	0.786493	0.16754	2624	5715	0.786335	0.166618	984.2	2110	0.816763	0.139735
3537	6384	0.74182	0.146759	2470	4983	0.740186	0.145277	936.2	1794	0.776929	0.118808
2931	4212	0.614723	0.096828	2084	3387	0.624513	0.098746	846.9	1294	0.702822	0.085695
1505	237.8	0.315646	0.005467	1107	270.9	0.331735	0.007898	512.4	-209.8	0.425228	-0.01389
397	-2520	0.083263	-0.05793	282.3	-2053	0.084597	-0.05985	168	-1569	0.139419	-0.10391
151.4	-3140	0.031753	-0.07218	94.61	-2607	0.028352	-0.07601	54.07	-2022	0.044871	-0.13391
-2.6E-13	-3512	-5.5E-17	-0.08074	9.997E-15	-2874	3E-18	-0.08379	-1.7E-14	-2235	-1.4E-17	-0.14801
-2.6E-13	-3512	-5.5E-17	-0.08074	9.997E-15	-2874	3E-18	-0.08379	-1.7E-14	-2235	-1.4E-17	-0.14801
-2.6E-13	-3512	-5.5E-17	-0.08074	9.997E-15	-2874	3E-18	-0.08379	-1.7E-14	-2235	-1.4E-17	-0.14801
-2.6E-13	-3512	-5.5E-17	-0.08074	9.997E-15	-2874	3E-18	-0.08379	-1.7E-14	-2235	-1.4E-17	-0.14801
-2.6E-13	-3512	-5.5E-17	-0.08074	9.997E-15	-2874	3E-18	-0.08379	-1.7E-14	-2235	-1.4E-17	-0.14801

Table 3.15 above shows the integration diagram values for individual columns. There are columns of scaling factors, which means to enter these values into ETABS, we need to scale it. These columns show the scaled values of the integration diagram. To define the plastic hinge for columns, we need to enter integration P-M values into ETABS. After then, I used the Xtract program to find the moment-curvature curve for the columns, and then after scaling, we are going to enter it in plastic hinge information of column. Plastic hinge rotation for columns are given in Table 3-19.

Column name = C1 (90 x 90)

Hinge length = 450 mm half of section depth

$$M_y = 1429 \text{ kN.m}$$

$$M_u = 1857 \text{ kN.m}$$

$$\phi_y = 0.00392 \text{ } 1/m$$

$$\phi_u = 0.06159 \text{ } 1/m$$

$$\text{Moment } SF = 1429 \text{ kN.m}$$

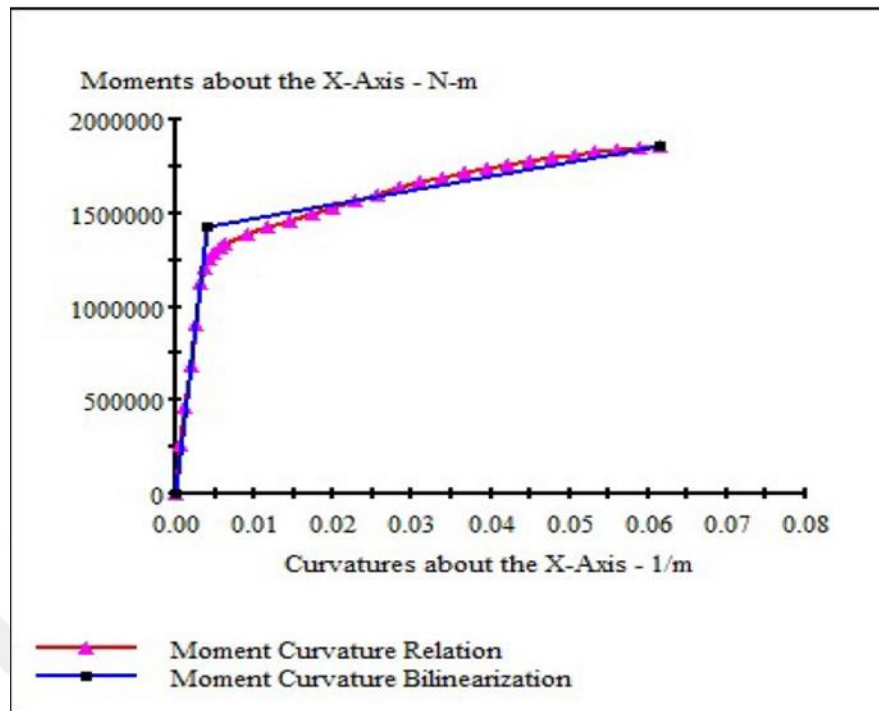


Figure 3.21 : Column C1 moment-curvature curve taken from Xtract

As we see in Figures 3.21 and Figure 3.22 integration diagram and moment-curvature curve, which is taken from Xtract and then scaled, is entered into the ETABS program. All of these data is entered into nonlinear hinge properties of the column. This procedure is done for every column individually.

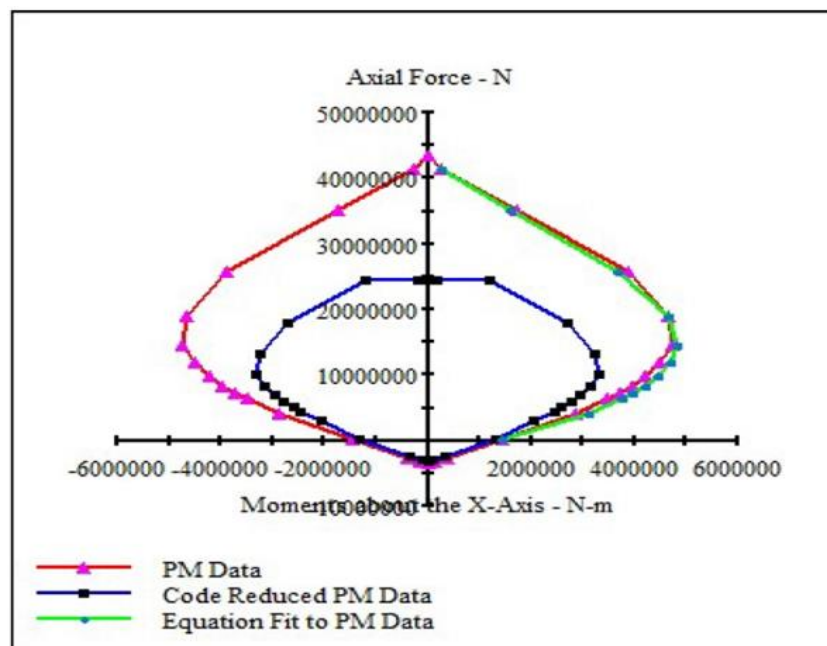


Figure 3.22 : Column C1 interaction diagram taken from Xtract

P-M2-M3 Interaction Surface Definition for column 90x90 hinge

User Interaction Surface Options

☒ Circular Symmetry
☐ Doubly Symmetric about M2 and M3
☐ No Symmetry

Number of Curves: 1
 Number of Points on Each Curve: 13

Scale Factors (Same for All Curves)

P, kN: 43500
 M2, kN-m: 4673
 M3, kN-m: 4673

☐ Include Scale Factors in Plots

First and Last Points (Same for All Curves)

Point	P	M2	M3
1	-0.0807	0	0
13	1	0	0

Interaction Curve Data

Current Curve: 1

Point	P/SF	M2/SF	M3/SF
1	-0.0807	0	0
2	-0.078	0.02	0
3	-0.074828	0.032683	0
4	-0.068681	0.056946	0
5	-0.061567	0.084246	0
6	0	0.3156	0
7	0.1468	0.7418	0
8	0.2269	0.892	0
9	0.4345	0.9801	0
10	0.5908	0.8167	0
11	0.823229	0.3626	0
12	0.973702	0.074685	0

Interaction Surface Requirements - Circular Symmetry

- Only one P-M2-M3 curve is specified.
- P (tension positive) increases monotonically.
- M2 = M3 = 0 at the first and last points.
- The specified curve has all M3 = 0 and all M2 > 0.
- Each curve must be convex and the interaction surface as a whole must be convex (no dimples in surface).

3D Plot

Plan, deg: -90
 Elevation, deg: 0
 Aperture, deg: 0

☐ Show All Lines
☒ Hide P Direction Lines
☐ Hide M2-M3 Lines

☒ Highlight Current Curve

OK Cancel

Figure 3.23 : P-M2-M3 plastic hinge defining for C1 in ETABS

Moment Curvature Data for column 90x90 hinge - Interacting P-M2-M3

Select Curve

Axial Force: 0
 Angle: 0
 Curve #1: 1

Moment Curvature Data for Selected Curve

Point	Moment/Yield Mom	Curvature/SF
A	0	0
B	1	0
C	1.299	0.06159
D	0.2	0.06159
E	0.2	0.5

Note: Yield moment is defined by interaction surface

Acceptance Criteria (Plastic Deformation / SF)

☒ Immediate Occupancy: 0.003
☐ Life Safety: 0.012
☐ Collapse Prevention: 0.05

☐ Show Acceptance Points on Current Curve

3D View

Plan: 315 deg
 Elevation: 35 deg
 Aperture: 0 deg

☐ Hide Backbone Lines
☐ Show Acceptance Criteria
☐ Show Thickened Lines
☒ Highlight Current Curve

Moment Curvature Information

Symmetry Condition: Circular
 Number of Axial Force Values: 1
 Number of Angles: 1
 Total Number of Curves: 1

Angle Is Moment About

0 degrees = About Positive M2 Axis
 90 degrees = About Positive M3 Axis
 180 degrees = About Negative M2 Axis
 270 degrees = About Negative M3 Axis

OK Cancel

Figure 3.24 : Defining moment-curvature of C1 column in ETABS

We are now defining nonlinear plastic hinge for shear walls in the building. To do this, we have to define the fiber hinge for shear walls. For fiber hinge, I divided the shear wall section into small parts, and then for every small piece of a section or cell calculation of concrete area and reinforcement area is done. Also, coordinates of these cells are defined from center of the shear wall. And then this hinge assigned for the critical height of the shear wall, which is about eight-story height. The table below shows the fiber hinge information of shear wall P1, which is 600cm×50 cm in dimension. Nonlinear hinge information is given in Table 3.16.

Table 3.25 : Shear wall nonlinear fiber hinge material, coordinate and area

Shear Wall Nonlinear Fiber Hinge		
Area	Coordinate from Center section	Material Type
cm2	mm	
2000	-2800	C40 /// SSC1
22	-2800	S420a /// SSC1
2000	-2400	C40 /// SSC1
11	-2400	S420a /// SSC1
2000	-2000	C40 /// SSC1
11	-2000	S420a /// SSC1
2000	-1600	C40 /// SSC1
10.2	-1600	S420a /// SSC1
2000	-1200	C40 /// SSC1
10.2	-1200	S420a /// SSC1
10.2	-800	S420a /// SSC1
2000	-800	C40 /// SSC1
2000	-400	C40 /// SSC1
10.2	-400	S420a /// SSC1
1000	-100	C40 /// SSC1
5.1	-100	S420a /// SSC1
5.1	100	S420a /// SSC1
1000	100	C40 /// SSC1
10.2	400	S420a /// SSC1
2000	400	C40 /// SSC1
10.2	800	S420a /// SSC1
2000	800	C40 /// SSC1
10.2	1200	S420a /// SSC1
2000	1200	C40 /// SSC1
10.2	1600	S420a /// SSC1
2000	1600	C40 /// SSC1
2000	2000	C40 /// SSC1
11	2000	S420a /// SSC1
2000	2400	C40 /// SSC1
11	2400	S420a /// SSC1
2000	2800	C40 /// SSC1
22	2800	S420a /// SSC1

3.13.10 Calculating rotation limits for beam and column

The building is analyzed under 2×11 seismic motions, which is taken from the PEER database. For nonlinear time history analysis of the building, we need to compare it with some limits which are given in the Turkish New Building Code 2018. For beams and columns, there is a limit given for rotation of sections. After analysis for 2×11 seismic motion, we take the average of them and then comparing it with limits.

At first, I found section rotational limits for beams. This is calculated by using information taken from the Xtract program. This calculation is done for all beam individually. Now, as an example, calculation done for the rotation limit of a beam using equation (3.41).

$$\begin{aligned}
 & \text{Beam Name} = B5 \\
 & h = 70 \text{ cm} \quad b = 60 \text{ cm} \\
 & \phi_y = 0.004972 \text{ } 1/m \\
 & \phi_u = 0.0679 \text{ } 1/m \\
 & L = 12 \text{ m} \quad L_s = 6 \text{ m} \quad \text{half of length} \\
 & L_p = 35 \text{ cm} \quad (\text{plastic hinge length, half of section depth}) \\
 & d_p = 22 \text{ mm} \\
 & \theta_p(GO, CP) = \frac{2}{3} \left[(\phi_u - \phi_y) L_p \left(1 - 0.5 \frac{L_p}{L_s} \right) + 4.5 \phi_u d_p \right] \quad (3.41) \\
 & \theta_p(GO, CP) = \frac{2}{3} \left[(0.0679 - 0.004972) \times 0.35 \times \left(1 - 0.5 \frac{0.35}{6} \right) + 4.5 \times 0.0679 \right. \\
 & \quad \left. \times 0.022 \right] \\
 & \theta_p(GO, CP) = 0.019 \text{ rad}
 \end{aligned}$$

It means that beam B5 average rotation from 2×11 seismic rotation should not cross the limit of θ_p . If the beam does not cross the limit, it means that this beam will be safe and do not collapse even a large earthquake happens.

Table 3.27 : Plastic hinge rotation limits for beams

Plastic Hinge Rotation Limits For Beams										
Beam Name	Direction	b	h	Φ_y	Φ_u	Ls	Lp	dp	θ_p (GO)	θ_p (KH)
		cm	cm	1/m	1/m	m	cm	mm	rad	rad
B5	X	60	70	0.004972	0.0679	6	35	22	0.019	0.0141
B2	X	50	75	0.00481	0.0514	4.5	37.5	22	0.015	0.0109
B3	X	50	75	0.00481	0.0514	3	37.5	22	0.014	0.0107
B8	X	45	60	0.00606	0.0662	1.875	30	22	0.015	0.0116
B7	X	45	60	0.00606	0.0662	2.25	30	22	0.016	0.0117
B1	Y	50	75	0.00481	0.0514	3.5	37.5	22	0.014	0.0108
B4	Y	50	75	0.00464	0.0631	2	37.5	22	0.017	0.0131
B9	Y	50	75	0.00464	0.0631	5	37.5	22	0.018	0.0137
B6	Y	45	60	0.00606	0.0662	5	30	22	0.016	0.0120

Next, I found section rotational limits for columns. This is calculated by using information taken from the Xtract program. This calculation is done for all columns individually. Now, as an example, calculation done for the rotation limit of a column using equation (3.42).

$$\text{Column Name} = C1$$

$$h = 90 \text{ cm} \quad b = 90 \text{ cm}$$

$$\phi_y = 0.003102 \text{ } 1/m$$

$$\phi_u = 0.0615 \text{ } 1/m$$

$$L = 3.5 \text{ m} \quad L_s = 1.75 \text{ m} \quad \text{half of length}$$

$$L_p = 45 \text{ cm} \quad (\text{plastic hinge length, half of section depth})$$

$$d_p = 22 \text{ mm}$$

$$\theta_p(GO, CP) = \frac{2}{3} \left[(\phi_u - \phi_y) L_p \left(1 - 0.5 \frac{L_p}{L_s} \right) + 4.5 \phi_u d_p \right] \quad (3.42)$$

Table 3.28 : Plastic hinge rotation of Beam B2

Plastic Hinge rotation of Beam B2											
story	tabas iran 0	tabas iran 90	duzce 0	duzce 90	hector mine 0	hector mine 90	imperial elcentro 0	imperial elcentro 90	kobe japan 0	kobe japan 90	kocaili 0
24	0.000584	0.000298	0	0	0	0.000464	0	0	0.000509	0.000296	0
23	0.000154	0.002092	0.000878	0.001699	0.001164	0.002141	0.0016	0.001626	0.002175	0.001995	0.001229
22	0.00248	0.002878	0.002046	0.002408	0.002044	0.003166	0.002129	0.002205	0.002801	0.002824	0.001841
21	0.00279	0.003327	0.0031	0.003875	0.003054	0.004092	0.003007	0.002573	0.004254	0.003429	0.00206
20	0.00214	0.003975	0.003591	0.005203	0.003917	0.004618	0.003848	0.002967	0.004899	0.003844	0.00221
19	0.00271	0.005015	0.003353	0.006041	0.004653	0.004914	0.004428	0.003362	0.005643	0.004041	0.00268
18	0.003042	0.0056	0.003684	0.006338	0.005363	0.004935	0.005539	0.003589	0.006103	0.00427	0.003137
17	0.003689	0.006519	0.0041	0.006238	0.005995	0.00562	0.006266	0.003895	0.006577	0.004598	0.003842
16	0.004702	0.006859	0.004586	0.005831	0.006536	0.005745	0.0065	0.003892	0.006625	0.005002	0.004379
15	0.005495	0.006603	0.004998	0.005161	0.006997	0.006148	0.006347	0.003833	0.007138	0.005328	0.004786
14	0.006035	0.006156	0.00529	0.00463	0.007282	0.006632	0.005927	0.003522	0.006889	0.0051	0.005084
13	0.006253	0.006256	0.005464	0.004827	0.007427	0.006478	0.005887	0.003646	0.007114	0.004948	0.005192
12	0.006287	0.006096	0.005568	0.005053	0.007407	0.006055	0.006051	0.003685	0.006575	0.005298	0.00538
11	0.006055	0.005718	0.006121	0.005138	0.007034	0.005419	0.005917	0.004445	0.006175	0.005471	0.005611
10	0.005559	0.004781	0.006407	0.005232	0.006291	0.006088	0.005408	0.004446	0.005739	0.005411	0.005403
9	0.004962	0.004639	0.006392	0.005007	0.005416	0.006152	0.004719	0.004147	0.005465	0.005243	0.005111
8	0.00453	0.003611	0.004743	0.003342	0.004349	0.005175	0.003087	0.002915	0.004083	0.00425	0.003218
7	0.002701	0.002179	0.002397	0.001632	0.00234	0.003064	0.001495	0.001129	0.001992	0.002348	0.001526
6	0.002691	0.002247	0.002325	0.001584	0.002337	0.002962	0.001479	0.001186	0.001891	0.00237	0.0015
5	0.002587	0.00229	0.002172	0.001455	0.002212	0.002708	0.001361	0.001377	0.001625	0.002285	0.001416
4	0.00233	0.002191	0.001901	0.001217	0.00194	0.00233	0.001141	0.001134	0.001215	0.00205	0.001258
3	0.001831	0.00183	0.001409	0.000802	0.001445	0.001829	0.000772	0.000844	0.000677	0.00158	0.000895
2	0.001123	0.00122	0.000739	0.000236	0.00077	0.001161	0.000245	0.00036	0	0.000913	0.000355
1	0	0	0	0	0	0	0	0	0	0	0

$$\theta_p(GO, CP) = \frac{2}{3} \left[(0.0615 - 0.003102) \times 0.45 \times \left(1 - 0.5 \frac{0.45}{1.75} \right) + 4.5 \times 0.0615 \times 0.022 \right]$$

$$\theta_p(GO, CP) = 0.0195 \text{ rad}$$

It means that Column C1 average rotation from 2×11 seismic rotation should not cross the limit of θ_p . If the Column does not cross the limit, it means that this Column will be safe and do not collapse even a large earthquake happens. For a building to do not collapse, shear walls and columns never should cross the limit. The design should be in such a way that the building under large earthquake must be safe and do not collapse and let the living people escape and save their lives. May large damage happen in the building, but the collapse of the building should be prevented, and that only can happen when columns do not cross the limits.

Table 3.30 : Plastic hinge rotation for columns

Plastic Hinge Rotation Limits For Columns									
column Name	b	h	Φ_y	Φ_u	Ls	Lp	dp	θ_p (GO)	θ_p (KH)
	cm	cm	1/m	1/m	m	cm	mm	rad	rad
C1	90	90	0.003102	0.0615	1.75	45	22	0.0193	0.0145
C2	80	80	0.00353	0.0675	1.75	40	22	0.0196	0.0147
C3	45	60	0.005212	0.05784	1.75	30	22	0.0134	0.0101

Now I calculated limiting values for shear wall sections. Because we are allowed to define fiber hinge for shear wall sections of the building. So I have to calculate strain limits for reinforcement and concrete. There are formulas to calculate strain of reinforcement and concrete, and then after analysis of building under 2×11 seismic records and taking the average, we have to compare it with limits.

$$\text{Shear wall Name} = P1$$

$$b_o = 50 \text{ cm} - 2 \times 4 \text{ cm} = 42 \text{ cm}$$

$$h_o = 600 \text{ cm} - 2 \times 4 \text{ cm} = 592 \text{ cm}$$

$$s = 10 \text{ cm}$$

$$\sum a_i^2 = 18 \times (50 \text{ cm} - 14 \text{ cm})^2 = 0.0648 \text{ m}^2$$

$$\alpha_{se} = \left(1 - \frac{0.0648}{6 \times 0.42 \times 5.92}\right) \left(1 - \frac{0.1}{2 \times 0.42}\right) \left(1 - \frac{0.1}{2 \times 5.92}\right) = 0.86971$$

$$\rho_{sh} = \frac{A_{sh}}{b_k s} = \frac{4 \times 1.54}{44 \times 10} = 0.014$$

$$\omega_{we} = 0.86971 \times 0.014 \times \frac{462}{52} = 0.10817$$

$$\varepsilon_c(GO, CP) = 0.0035 + 0.04\sqrt{0.10817} = 0.01665 < 0.018$$

$$\varepsilon_c(KH) = 0.75 \times 0.01665 = 0.01249$$

For steel, the strain limit is:

$$\varepsilon_s(GO, CP) = 0.4 \times 0.08 = 0.032 \quad \text{for Steel S420}$$

$$\varepsilon_s(KH) = 0.75 \times 0.032 = 0.024 \quad \text{for Steel S420}$$

So now a strain of concrete and steel in shear wall P1 section should not cross the limit of 0.01665 and 0.032, respectively. If we want the building to be safe and do not collapse.

The NGA database constitutes the largest database of processed, recorded ground motions in Stable Continental Regions (SRCs). It was developed as part of a large multi-disciplinary research project coordinated by PEER. The NGA-East database includes the two- and three-component ground-motion recordings from numerous selected events ($M > 2.5$, distances up to 1500 km) recorded in the Central and Eastern North America (CENA) region since 1988. The database contains over 29,000 records from 81 earthquake events and 1379 recording stations. The database includes time series and pseudo-spectral acceleration (PSA) for the 5%-damped elastic oscillators with periods ranging from 0.01 to 10 sec. Additionally, the NGA-East database includes the Fourier amplitude spectral (FAS) of the processed ground motions.

After taking all the seismic records from the peer database, then we need only acceleration data. I entered all these acceleration data to the ETABS program. There is an option of defining time history function defining. In there, we need to select the acceleration file of every data taken from peer database, which is 2×11 records. After

entering, we need to define the damping ratio, which we entered 5% for all situations. As I mentioned before, I defined a target spectrum, which is the DD-1 level earthquake spectrum taken from AFAD site after entering coordinate and information of the building site. We need this targeted spectrum because we have to scale all these data taken from AFAD with the target spectrum. So to perform this, there is an option available in ETABS, which is for scaling. Figure 3-25 shows the insertion of seismic ground motions and Figure 3-26 show the scaling of them.

Under the option of define in ETABS, there is a time history function defining option, so in there, I used the DD-1 spectrum as a target spectrum and then each acceleration file of seismic record as to be scaled.

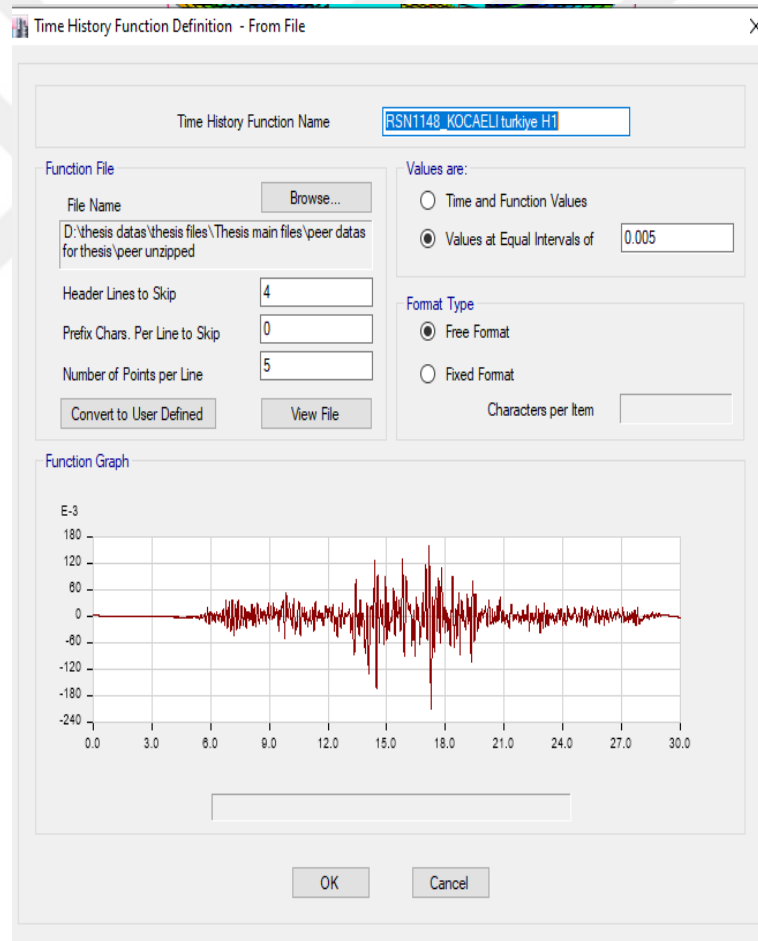


Figure 3.25 : Defining time history functions for 22 records

Then scaling of all records done using the time domain because it takes the best result of scaling (Yasin M. Fahjan, 2010). After scaling of all records done and saved in ETABS, then in the load case option, we define the 22-time history load case by selecting the scaled time history records. After doing all these steps, nonlinear analysis of structure under these 22 seismic records is done, and the result is taken for comparing and evaluation.

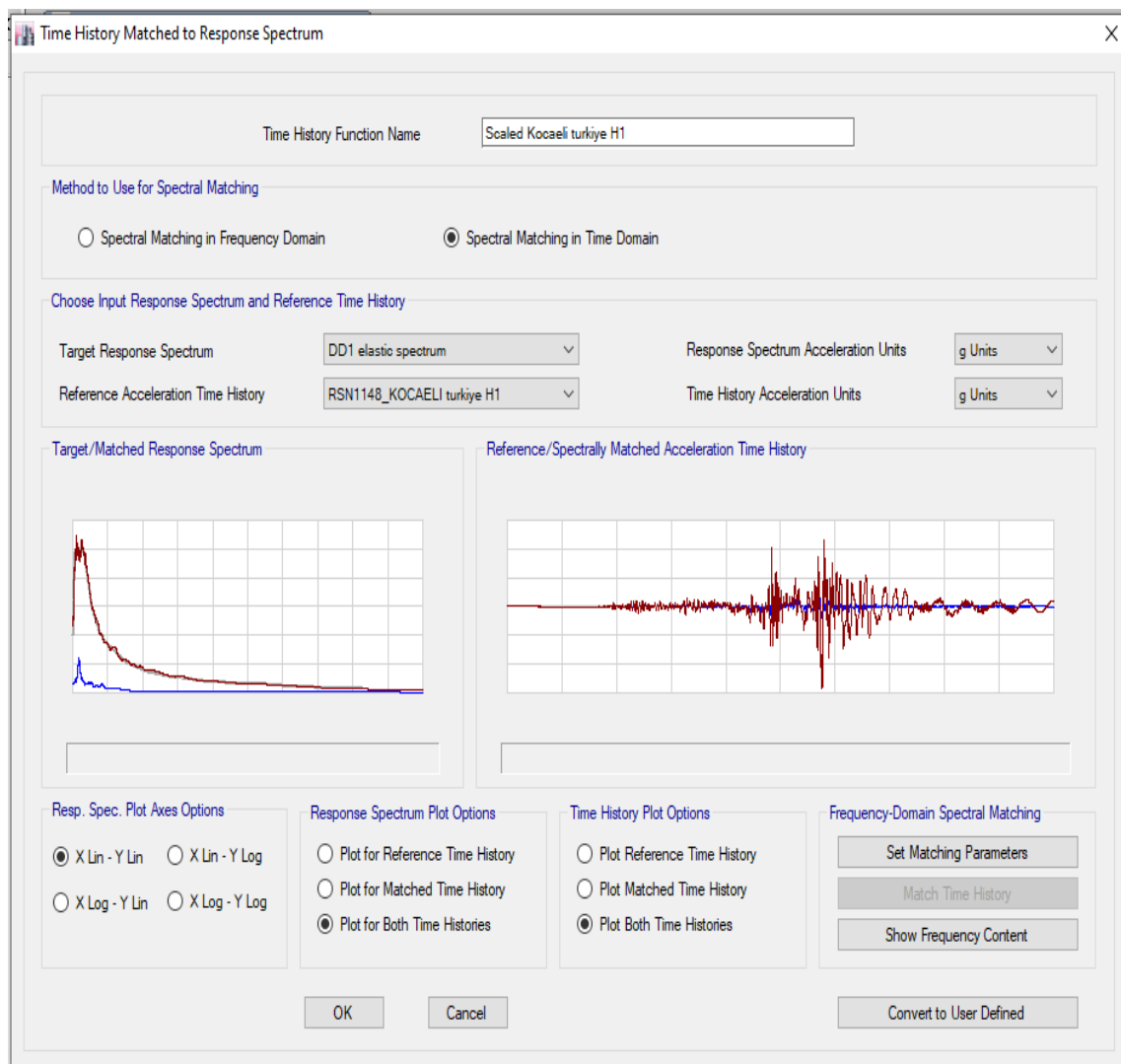


Figure 3.26 : Scaling of seismic records by matching with a target spectrum.

Table 3.31 : Continue of plastic hinge rotation of beam B2

Continue Plastic Hinge rotation of Beam B2												
kocaili 90	landers yermo 0	landers yermo 90	landers joshua 0	landers joshua 90	el alamo elcentro 0	el alamo elcentro 90	kern country 0	kern country 90	trinidad 0	trinidad 90	average	limit
0.000457	0.00025	0	0	0	0	0.000667	0	0	0	0	0.0002	0.015
0.002175	0.002186	0.001656	0.001394	0.001625	0.000832	0.00259	0.001098	0.001043	0.001819	0.001819	0.0016	0.015
0.00259	0.003117	0.002395	0.001632	0.002276	0.001773	0.00347	0.001469	0.001327	0.002839	0.002979	0.0024	0.015
0.003263	0.003357	0.002907	0.002523	0.003743	0.00239	0.004485	0.002087	0.002168	0.003809	0.00401	0.0032	0.015
0.004603	0.003389	0.003401	0.003851	0.004439	0.003002	0.005117	0.002806	0.002902	0.004709	0.004554	0.0038	0.015
0.0061	0.004023	0.00395	0.004466	0.004426	0.003194	0.005397	0.003512	0.003639	0.005276	0.005171	0.0044	0.015
0.006277	0.004034	0.004277	0.004692	0.005448	0.003176	0.005306	0.003747	0.004181	0.005945	0.006035	0.0048	0.015
0.005848	0.003478	0.00461	0.005658	0.006686	0.003471	0.00538	0.004028	0.004402	0.006648	0.00686	0.0052	0.015
0.005891	0.003446	0.005027	0.006179	0.007302	0.003525	0.005713	0.004248	0.004417	0.006813	0.007436	0.0055	0.015
0.006393	0.004185	0.005907	0.00594	0.006037	0.003339	0.005313	0.004482	0.004445	0.006838	0.007249	0.0056	0.015
0.006013	0.004709	0.007087	0.005838	0.006184	0.002987	0.005822	0.004774	0.004544	0.007157	0.006656	0.0057	0.015
0.006264	0.005241	0.007553	0.005919	0.007291	0.002528	0.006155	0.005081	0.005318	0.007508	0.007319	0.0059	0.015
0.006083	0.005264	0.007167	0.005024	0.007668	0.002739	0.006245	0.005448	0.005886	0.007302	0.00753	0.0059	0.015
0.005569	0.004593	0.005777	0.004094	0.007641	0.003289	0.00588	0.005707	0.006023	0.006441	0.007022	0.0057	0.015
0.00544	0.005094	0.005821	0.004345	0.00675	0.003742	0.005124	0.005983	0.005881	0.005	0.00639	0.0055	0.015
0.004532	0.004984	0.005903	0.002917	0.005581	0.004178	0.005206	0.006346	0.006953	0.004453	0.006657	0.0052	0.015
0.004006	0.004241	0.004887	0.001203	0.006201	0.003536	0.004381	0.005479	0.006969	0.003847	0.004824	0.0042	0.015
0.002283	0.002433	0.002737	0.001173	0.004155	0.001798	0.002605	0.003251	0.00456	0.001992	0.002484	0.0024	0.015
0.002295	0.002433	0.002695	0.001049	0.004098	0.001791	0.002637	0.003252	0.004568	0.001976	0.002437	0.0024	0.015
0.002177	0.002317	0.002528	0.000805	0.003886	0.001684	0.002574	0.003125	0.004437	0.001842	0.002265	0.0022	0.015
0.001894	0.002045	0.002207	0.000418	0.003447	0.001445	0.002352	0.002824	0.004087	0.001566	0.002003	0.0020	0.015
0.001387	0.001543	0.001663	0.000418	0.002881	0.001008	0.001876	0.002254	0.003402	0.001092	0.001526	0.0015	0.015
0.000714	0.000851	0.000947	0	0.002155	0.000411	0.001184	0.001465	0.00244	0.000508	0.000865	0.0008	0.015
0	0	0	0	0.000315	0	0	0	0.000444	0	0	0.0000	0.015

After analyzing the building in the ETABS program, we get results of plastic hinge rotation for 2×11 seismic records. Then average of these values taken for the beams. And then, comparison done between the average value with the Collapse prevention rotation limit at 0.06 rad. As we see in the Table 3.20, none of the beams crosses the limit, and so we can say all the beams are safe, and after a large earthquake, the building beam will not fail. As an example, we see in Table 3.20 and Figure 3.27 that is done for beam B2 for all stories of the building. Moreover, none of the beams in a story crosses the limit. However, does not cross the limit of rotation, it is so smaller than the limit. Even the rotation does not cross the controlled damage level of performance, so it means that

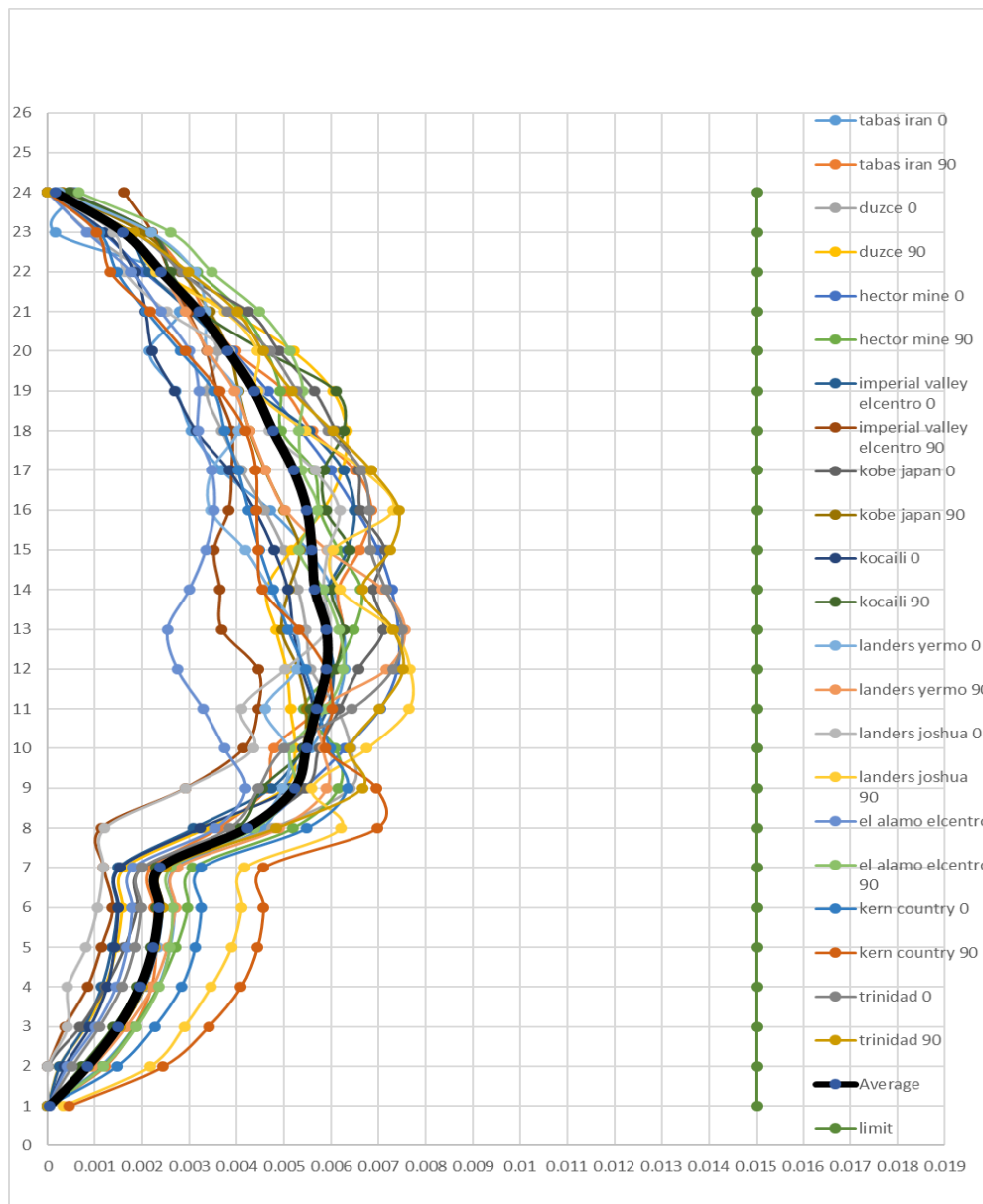


Figure 3.27 : Plastic rotation of hinge in beam B2 in all stories

the beam B2 will damage in the large earthquakes but not significantly. As we see maximum rotations in the beam B2 happens in story 12 of the building.

Now, like as we did before for beam, we are going to do for a column of the building. In code says that the result of the 2×11 seismic analysis record should not cross the plastic hinge rotation limit for columns. So first, as we analyzed the building under each seismic load individually, we had a plastic rotation for every one of it. Then we take the average of 2×11 records for columns. And as we see in before sections, we found some plastic hinge rotation for column top and bottom supports that was the limit for Collapse Prevention limit for the column. If columns do not cross the limits, it means that our columns are safe and will not collapse under large earthquakes which can happen in this area. As we see in the Table 3.22, all the average values are significantly less than the limit. So we can say that the building column is safe and will not collapse and will let the people who are living there to escape and save their life.

Now for shear walls, also we need to check that if it is under the limit or not. So as we mentioned above, we assigned the fiber hinge for our shear walls, so it needs different checking methods. Now we need to compare the outermost part of section strain with limits. Since we have two types of material for our shear walls, so we are comparing the concrete and steel strain with limits as we found some limits for concrete and steel strain in the section above. Now in from ETABS, fiber strain values are derived for 2×11 seismic records.

Moreover, now we are comparing them with the limit. Moreover, as we see for shear wall P1, the average limit of strain for concrete and steel is less than the limit as we see in Table 3.21. So it means that our shear walls under the large earthquake will not collapse. Moreover, it will be safe under these types of seismic motion. Thus, if the column, beam, and shear walls of a building satisfy the limits and do not cross, so it means the building is safe and will not collapse, and our design is in the best way as we want and perform the required performance level.

Table 3.34 : Shear wall P1 strain check for concrete and steel

Shear Wall P1 Strain Check for Concrete and Steel		
No	Name	Strain
1	Tabas iran 0	0.00612
2	tabas 90	0.00651
3	duzce 0	0.00549
4	duzce 90	0.00483
5	hector mine 0	0.00571
6	hector mine 90	0.00612
7	imperial valley elcentro 0	0.00481
8	imperial valley elcentro 90	0.00517
9	kobe japan 0	0.00456
10	Kobe japan 90	0.00602
11	kocaili 0	0.00497
12	kocaili 90	0.00562
13	landers Yermo 0	0.00574
14	landers Yermo 90	0.00583
15	landers Joshua 0	0.00411
16	landers Joshua 90	0.00773
17	El Alamo El-Centro 0	0.00510
18	El Alamo El-Centro 90	0.00641
19	Kern country 0	0.00694
20	Kern country 90	0.00835
21	Trinidad 0	0.00530
22	Trinidad 90	0.00581
Average		0.00578
Limit for Concrete Strain		0.01665
Limit for Steel Strain		0.03200

Table 3.35 : Columns plastic hinge rotation check with limits

Columns Plastic Hinge Rotation Check												
No	1	2	3	4	5	6	7	8	9	10	11	12
Name	Tabas iran 0	tabas 90	duzce 0	duzce 90	hector mine 0	hector mine 90	imperial valley elcentro 0	imperial valley elcentro 90	kobe japan 0	kobe japan 90	kocaili 0	kocaili 90
C1	0.003531	0.006439	0.003705	0.004263	0.002388	0.003524	0.003898	0.006188	0.004167	0.00219	0.004412	0.003458
C4	0.008795	0.002313	0.00083	0.00556	0.002988	0.002638	0.004634	0.007189	0.005833	0.00246	0.002818	0.004566
C5	0.010165	0.004015	0.003064	0.005087	0.003216	0.003677	0.005433	0.001823	0.004517	0.005632	0.00423	0.005395
C6	0.003335	0.003837	0.006048	0.005651	0.001666	0.002825	0.003611	0.003296	0.006628	0.001146	0.003862	0.002536
C2	0.002934	0.007827	0.003594	0.000906	0.00493	0.002732	0.004149	0.004305	0.005132	0.005757	0.004067	0.005914
C3	0.00555	0.003555	0.003066	0.005401	0.005462	0.007001	0.003792	0.006219	0.004285	0.00573	0.003588	0.006101
C11	0.00687	0.003731	0.004155	0.00881	0.00399	0.006861	0.006942	0.005308	0.005408	0.004925	0.002995	0.004959
C12	0.00217	0.003628	0.004866	0.003124	0.00451	0.002977	0.003317	0.010955	0.006468	0.004365	0.002052	0.004909

No	13	14	15	16	17	18	19	20	21	22		
Name	landers yermo 0	landers yermo 90	landers joshua 0	landers joshua 90	El Alamo El-centro 0	El Alamo El-centro 90	Kern country 0	Kern coutry 90	Trinidad 0	Trinidad 90	Average	Limit
C1	0.003529	0.002637	0.003368	0.002182	0.004627	0.007398	0.002585	0.004291	0.006645	0.004311	0.0040	0.0145
C4	0.003461	0.004525	0.003569	0.001735	0.003141	0.003985	0.003132	0.003722	0.006775	0.003339	0.0042	0.0145
C5	0.002668	0.004855	0.004686	0.003213	0.00395	0.00514	0.004791	0.004926	0.001846	0.00813	0.0047	0.0145
C6	0.001305	0.005349	0.004474	0.0029	0.004847	0.006215	0.002984	0.00297	0.003268	0.001718	0.0037	0.0145
C2	0.006362	0.007677	0.006505	0.002447	0.005871	0.006353	0.002145	0.003882	0.003091	0.008161	0.0044	0.0101
C3	0.005293	0.00539	0.001484	0.004988	0.00551	0.00398	0.001928	0.004587	0.007756	0.005628	0.0050	0.0101
C11	0.006587	0.007459	0.004226	0.002072	0.002753	0.005101	0.004735	0.006018	0.00469	0.005735	0.0054	0.0101
C12	0.004017	0.006161	0.005522	0.000887	0.004906	0.002592	0.002701	0.003104	0.006801	0.003339	0.0044	0.0101



4. CONCLUSION

In this case study, structural design of a 24 story tall building is accomplished according to TBDY 2018 code. Design formulas are taken from ACI- 314 and TS500 standards, and its seismic performance is evaluated according to chapter 13 of TBSC 2018 code.

In the strength-based design of the tall building the following parameters such as important factor, seismic design factor, building height factor, type of building, seismic load reduction factor, and other similar factors are used. Analysis of building is carried out using ETABS software. First, all information about structural system is defined in ETABS software and modal analysis is performed. The fundamental periods of tall building are obtained and given in tables. The first period is related to X-direction second one is related to the Y direction, and the third one is related to torsion of tall building. In numerical analysis first 25 modes are considered and it yields more than 95% of mass participation, which is coincident with TBSC 2018. After linear elastic analysis of tall building using ETABS under load combinations given in TBSC 2018 code, and unfavorable internal forces for all members are found. All members are designed according to ACI 318 and TS500 standards by using strength-based design method (DGT), and about 22.4% of base shear are resisted by frame system and the remaining percentage of all load is supported by shear walls. Generally, at least 25% of all loads should be resisted by the frames and the remaining part by shear walls. Because in here, the load supported by frame is about 22.4%, and to increase the strength of the frame and obtain the term redundancy, we put a little bit more reinforcement in frame sections to reach 25% capacity. By this, we increase the strength of frame sections, and if shear walls fail in some cases, then the frame will have the capacity to keep the building stable and let the people escape and prevent the collapse of the building.

We should also mention that in this case study, the slab of the building also designed. For the design of the slab, the SAFE software and stripped based design method is used. Waffle slab type is used because the spans are so long. To avoid punching shear near

columns and shear walls, the drop panels are used, and the drop panel dimension is calculated by formulas given in ACI-314. All reinforcement details of structural members and slab after the design is drawn by AUTOCAD software. Constructive rules and details of members are prepared according to TBDY 2018 code and TS500 standard.

Then the seismic performance of tall building is evaluated under seismic loads and vertical loads. Here we have two steps to evaluate the performance of the building. First, the building analyzed under DD-4 seismic load (service earthquake) for KK operational performance level. In this performance level, beams and column are checked for flexural and shear. For all structural members, the ratio of internal loads (flexural and shear) over their capacity is less than 1.5 and it is coincident with 13.5.5 section of TBSC 2018 code. So the structural system of the building is determined as safe for low seismic loads and performs KK (operational performance level) or IO (Immediate occupancy level).

In the last part the seismic performance of the building is evaluated for life safety performance level considering for controlled damage limits. In this part, the building is analyzed under DD-1 seismic load (maximum credible earthquake) and nonlinear time history analysis is accomplished. The first plastic hinge is assigned for all sections, and then limits for this performance level are calculated for all members. Then all member's plastic rotation and material strains compared with limits given in TBDY 2018 code. All members did not pass this limit, and finally, the building in this coordinate under high seismic loads will be safe and will not collapse. As a result, we see the plastic rotations are so far from limits which states that, the tall building is safe under maximum credible earthquake. So, it is shown that the life safety performance level is satisfied.

The strength based design of buildings is quite important, depending on the structural system capacity of a new design, it may satisfy proposed seismic performance levels easily. Regular structural system in plan and elevation is essential and structural system of a building should have sufficient lateral rigidity in two directions. For the tall building having 24 stories numerically treated here. It can be used for some other occupancy classes due to its high structural capacity.

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APPENDICES

APPENDIX A: Tables

APPENDIX B: Reinforcement Detailing and Graphs





APPENDIX A





Table A.1 : Modal mass participation ratios for building

Modal Participating Mass Ratios							
Case	Mode	Period (sec)	Direction	UX	UY	Sum UX	Sum UY
Modal	1	2.181	UX	0.72420	0.00000	0.7242	0
Modal	2	1.618	UY	0.00000	0.68870	0.7242	0.6887
Modal	3	1.333	RZ	0.00000	0.00000	0.7242	0.6887
Modal	4	0.624	UX	0.12360	0.00000	0.8478	0.6887
Modal	5	0.413	UY	0.00000	0.15400	0.8478	0.8427
Modal	6	0.378	RZ	0.00000	0.00000	0.8478	0.8427
Modal	7	0.297	UX	0.05260	0.00000	0.9003	0.8427
Modal	8	0.182	UZ	0.00000	0.05900	0.9003	0.9017
Modal	9	0.179	UY	0.00000	0.00001	0.9003	0.9017
Modal	10	0.174	UX	0.02970	0.00000	0.9301	0.9017
Modal	11	0.114	UY	0.01850	0.00000	0.9486	0.9017
Modal	12	0.107	UX	0.00000	0.02830	0.9486	0.93
Modal	13	0.105	RZ	0.00000	0.00000	0.9486	0.93
Modal	14	0.093	UY	0.00000	0.00460	0.9486	0.9346
Modal	15	0.082	UX+RZ	0.01210	0.00000	0.9607	0.9346
Modal	16	0.069	UX+RZ	0.00000	0.00002	0.9607	0.9346
Modal	17	0.069	UX	0.00000	0.01850	0.9607	0.9531
Modal	18	0.064	RZ	0.00480	0.00000	0.9655	0.9531
Modal	19	0.064	UY	0.00150	0.00000	0.967	0.9531
Modal	20	0.058	UX	0.00380	0.00000	0.9708	0.9531
Modal	21	0.051	UY	0.00000	0.01210	0.9708	0.9652
Modal	22	0.05	RZ	0.00000	0.00000	0.9708	0.9652
Modal	23	0.048	UX	0.00590	0.00000	0.9767	0.9652
Modal	24	0.04	UX	0.00470	0.00000	0.9814	0.9652
Modal	25	0.039	UY	0.00000	0.00820	0.9814	0.9735

Table A.2 : Reduced elastic design spectrum values for DD-2 earthquake level spectrum

T	Sae	R	SaeR	T	Sae	R	SaeR
0	0.77272	2.5	0.309088	4.7431	0.119837	7	0.01712
0.001	0.792417	2.515294	0.315039	4.9431	0.114989	7	0.016427
0.005	0.871203	2.57647	0.338138	5.1431	0.110517	7	0.015788
0.009	0.949989	2.637646	0.360166	5.3431	0.10638	7	0.015197
0.013	1.028776	2.698822	0.381194	5.5431	0.102542	7	0.014649
0.017	1.107562	2.759998	0.401291	5.7431	0.098971	7	0.014139
0.021	1.186349	2.821174	0.420516	5.9431	0.09564	7	0.013663
0.025	1.265135	2.88235	0.438925	6	0.094733	7	0.013533
0.029	1.343922	2.943526	0.456569	6.05	0.093174	7	0.013311
0.033	1.422708	3.004701	0.473494	6.1	0.091653	7	0.013093
0.037	1.501495	3.065877	0.489744	6.15	0.090169	7	0.012881
0.041	1.580281	3.127053	0.505358	6.2	0.08872	7	0.012674
0.045	1.659067	3.188229	0.520373	6.25	0.087306	7	0.012472
0.049	1.737854	3.249405	0.534822	6.3	0.085926	7	0.012275
0.053	1.81664	3.310581	0.548738	6.35	0.084578	7	0.012083
0.0588	1.930881	3.399286	0.568025	6.4	0.083262	7	0.011895
0.061	1.930881	3.432933	0.562458	6.45	0.081976	7	0.011711
0.065	1.930881	3.494109	0.55261	6.5	0.08072	7	0.011531
0.0686	1.930881	3.549167	0.544038	6.55	0.079492	7	0.011356
0.1186	1.930881	4.313866	0.447599	6.6	0.078292	7	0.011185
0.1686	1.930881	5.078566	0.380202	6.65	0.077119	7	0.011017
0.2186	1.930881	5.843265	0.330446	6.7	0.075972	7	0.010853
0.2686	1.930881	6.607964	0.292205	6.75	0.074851	7	0.010693
0.275	1.930881	6.705845	0.28794	6.8	0.073754	7	0.010536
0.2942	1.930881	6.99949	0.27586	6.85	0.072682	7	0.010383
0.311	1.827653	7	0.261093	6.9	0.071632	7	0.010233
0.3431	1.65666	7	0.236666	6.95	0.070605	7	0.010086
0.5431	1.046584	7	0.149512	7	0.0696	7	0.009943
0.7431	0.764904	7	0.109272	7.05	0.068616	7	0.009802
0.9431	0.602693	7	0.086099	7.1	0.067653	7	0.009665
1.1431	0.497244	7	0.071035	7.15	0.06671	7	0.00953
1.3431	0.4232	7	0.060457	7.2	0.065787	7	0.009398
1.5431	0.368349	7	0.052621	7.25	0.064883	7	0.009269
1.7431	0.326086	7	0.046584	7.3	0.063997	7	0.009142
1.9431	0.292522	7	0.041789	7.35	0.063129	7	0.009018
2.1431	0.265223	7	0.037889	7.4	0.062279	7	0.008897
2.3431	0.242585	7	0.034655	7.45	0.061446	7	0.008778
2.5431	0.223507	7	0.03193	7.5	0.060629	7	0.008661
2.7431	0.207211	7	0.029602	7.55	0.059829	7	0.008547
2.9431	0.19313	7	0.02759	7.6	0.059044	7	0.008435
3.1431	0.180841	7	0.025834	7.65	0.058275	7	0.008325
3.3431	0.170022	7	0.024289	7.7	0.057521	7	0.008217
3.5431	0.160424	7	0.022918	7.75	0.056781	7	0.008112
3.7431	0.151853	7	0.021693	7.8	0.056055	7	0.008008
3.9431	0.144151	7	0.020593	7.85	0.055343	7	0.007906
4.1431	0.137192	7	0.019599	7.9	0.054645	7	0.007806
4.3431	0.130874	7	0.018696	7.95	0.05396	7	0.007709
4.5431	0.125113	7	0.017873	8	0.053288	7	0.007613

Table A.3 : Elastic acceleration spectrum value for DD-4 earthquake level spectrum

DD-4 earthquake level Spectrum					
T (sec)	Sae	T (sec)	Sae	T (sec)	Sae
0	0.1534	2.1431	0.051794	6.75	0.01461728
0.001	0.157375	2.3431	0.047373	6.8	0.01440311
0.005	0.173275	2.5431	0.043648	6.85	0.01419362
0.009	0.189174	2.7431	0.040465	6.9	0.01398866
0.013	0.205074	2.9431	0.037715	6.95	0.01378811
0.017	0.220974	3.1431	0.035315	7	0.01359184
0.021	0.236873	3.3431	0.033203	7.05	0.01339973
0.025	0.252773	3.5431	0.031328	7.1	0.01321166
0.029	0.268673	3.7431	0.029655	7.15	0.01302753
0.033	0.284573	3.9431	0.02815	7.2	0.01284722
0.037	0.300472	4.1431	0.026792	7.25	0.01267063
0.041	0.316372	4.3431	0.025558	7.3	0.01249765
0.045	0.332272	4.5431	0.024433	7.35	0.0123282
0.049	0.348171	4.7431	0.023402	7.4	0.01216216
0.053	0.364071	4.9431	0.022456	7.45	0.01199946
0.0579	0.383548	5.1431	0.021582	7.5	0.01184
0.061	0.3835	5.3431	0.020774	7.55	0.0116837
0.065	0.3835	5.5431	0.020025	7.6	0.01153047
0.0686	0.3835	5.7431	0.019328	7.65	0.01138024
0.1186	0.3835	5.9431	0.018677	7.7	0.01123292
0.1686	0.3835	6	0.0185	7.75	0.01108845
0.2186	0.3835	6.05	0.018195	7.8	0.01094675
0.2686	0.3835	6.1	0.017898	7.85	0.01080774
0.275	0.3835	6.15	0.017609	7.9	0.01067137
0.2894	0.3835	6.2	0.017326	7.95	0.01053756
0.311	0.356913	6.25	0.01705	8	0.01040625
0.3431	0.323521	6.3	0.01678	8.05	0.01027738
0.5431	0.204382	6.35	0.016517	8.1	0.01015089
0.7431	0.149374	6.4	0.01626	8.15	0.01002672
0.9431	0.117697	6.45	0.016009	8.2	0.00990482
1.1431	0.097104	6.5	0.015763	8.25	0.00978512
1.3431	0.082645	6.55	0.015524	8.3	0.00966759
1.5431	0.071933	6.6	0.015289	8.35	0.00955215
1.7431	0.06368	6.65	0.01506		
1.9431	0.057125	6.7	0.014836		

Table A.4 : Elastic acceleration spectrum value for DD-1 earthquake level

DD-1 earthquake level Spectrum					
T (sec)	Sae	T (sec)	Sae	T (sec)	Sae
0	1.23936	2.1431	0.482759	6.75	0.136244
0.001	1.267197	2.3431	0.441552	6.8	0.134247
0.005	1.378545	2.5431	0.406826	6.85	0.132295
0.009	1.489894	2.7431	0.377165	6.9	0.130384
0.013	1.601242	2.9431	0.351534	6.95	0.128515
0.017	1.71259	3.1431	0.329165	7	0.126686
0.021	1.823939	3.3431	0.309473	7.05	0.124895
0.025	1.935287	3.5431	0.292004	7.1	0.123142
0.029	2.046635	3.7431	0.276402	7.15	0.121426
0.033	2.157984	3.9431	0.262382	7.2	0.119745
0.037	2.269332	4.1431	0.249716	7.25	0.118099
0.041	2.38068	4.3431	0.238217	7.3	0.116487
0.045	2.492029	4.5431	0.22773	7.35	0.114908
0.049	2.603377	4.7431	0.218127	7.4	0.11336
0.053	2.714725	4.9431	0.209302	7.45	0.111844
0.0588	2.876181	5.1431	0.201163	7.5	0.110357
0.061	2.937422	5.3431	0.193633	7.55	0.1089
0.0668	3.098877	5.5431	0.186646	7.6	0.107472
0.0686	3.098877	5.7431	0.180147	7.65	0.106072
0.1186	3.098877	5.9431	0.174084	7.7	0.104699
0.1686	3.098877	6	0.172433	7.75	0.103352
0.2186	3.098877	6.05	0.169595	7.8	0.102032
0.2686	3.098877	6.1	0.166826	7.85	0.100736
0.275	3.098877	6.15	0.164125	7.9	0.099465
0.2942	3.098877	6.2	0.161488	7.95	0.098218
0.3339	3.098877	6.25	0.158915	8	0.096994
0.3431	3.015447	6.3	0.156402	8.05	0.095793
0.5431	1.90499	6.35	0.153949	8.1	0.094614
0.7431	1.392276	6.4	0.151553	8.15	0.093456
0.9431	1.09702	6.45	0.149212	8.2	0.09232
1.1431	0.905083	6.5	0.146925	8.25	0.091204
1.3431	0.770307	6.55	0.144691	8.3	0.090109
1.5431	0.670469	6.6	0.142507	8.35	0.089033
1.7431	0.59354	6.65	0.140372		
1.9431	0.532448	6.7	0.138285		

Table A.5 : A1 torsional irregularity for X-X direction

Story	EQ Direction	Story Disp			Relative Story Disp			η_{bi}	Check
		di max	di min	di Avg	Δ_{max}	Δ_{min}	Δ_{avg}		≤ 1.2
24	Direction X	83.69	83.682	83.685	3.118	3.116	3.117	1.000321	OK
23	Direction X	80.57	80.566	80.568	3.175	3.175	3.175	1.000000	OK
22	Direction X	77.395	77.391	77.393	3.244	3.244	3.244	1.000000	OK
21	Direction X	74.151	74.147	74.149	3.335	3.337	3.336	0.999700	OK
20	Direction X	70.816	70.81	70.813	3.441	3.439	3.44	1.000291	OK
19	Direction X	67.375	67.371	67.373	3.554	3.554	3.554	1.000000	OK
18	Direction X	63.821	63.817	63.819	3.671	3.671	3.671	1.000000	OK
17	Direction X	60.15	60.146	60.148	3.788	3.788	3.788	1.000000	OK
16	Direction X	56.362	56.358	56.36	3.899	3.899	3.899	1.000000	OK
15	Direction X	52.463	52.459	52.461	4.001	4.001	4.001	1.000000	OK
14	Direction X	48.462	48.458	48.46	4.089	4.087	4.088	1.000245	OK
13	Direction X	44.373	44.371	44.372	4.159	4.159	4.159	1.000000	OK
12	Direction X	40.214	40.212	40.213	4.204	4.206	4.205	0.999762	OK
11	Direction X	36.01	36.006	36.008	4.225	4.225	4.225	1.000000	OK
10	Direction X	31.785	31.781	31.783	4.212	4.21	4.211	1.000237	OK
9	Direction X	27.573	27.571	27.572	4.161	4.161	4.161	1.000000	OK
8	Direction X	23.412	23.41	23.411	4.063	4.061	4.062	1.000246	OK
7	Direction X	19.349	19.349	19.349	3.91	3.912	3.911	0.999744	OK
6	Direction X	15.439	15.437	15.438	3.693	3.693	3.693	1.000000	OK
5	Direction X	11.746	11.744	11.745	3.396	3.396	3.396	1.000000	OK
4	Direction X	8.35	8.348	8.349	3.001	3.001	3.001	1.000000	OK
3	Direction X	5.349	5.347	5.348	2.484	2.482	2.483	1.000403	OK
2	Direction X	2.865	2.865	2.865	2.012	2.012	2.012	1.000000	OK
1	Direction X	0.853	0.853	0.853	0.853	0.853	0.853	1.000000	OK

Table A.6 : A1 torsional irregularity for Y-Y direction

Story	EQ Direction	Story drift			Relative story drift			η_{bi}	Check
		di (max)	di (min)	di (Avg)	Δ_{max}	Δ_{min}	Δ_{avg}		≤ 1.2
24	Direction Y	60.325	60.293	60.309	2.797	2.797	2.797	1.00000	OK
23	Direction Y	57.528	57.496	57.512	2.817	2.815	2.816	1.00036	OK
22	Direction Y	54.711	54.681	54.696	2.835	2.833	2.834	1.00035	OK
21	Direction Y	51.876	51.848	51.862	2.857	2.857	2.857	1.00000	OK
20	Direction Y	49.019	48.991	49.005	2.879	2.877	2.878	1.00035	OK
19	Direction Y	46.14	46.114	46.127	2.899	2.897	2.898	1.00035	OK
18	Direction Y	43.241	43.217	43.229	2.917	2.915	2.916	1.00034	OK
17	Direction Y	40.324	40.302	40.313	2.93	2.926	2.928	1.00068	OK
16	Direction Y	37.394	37.376	37.385	2.938	2.936	2.937	1.00034	OK
15	Direction Y	34.456	34.44	34.448	2.939	2.939	2.939	1.00000	OK
14	Direction Y	31.517	31.501	31.509	2.933	2.931	2.932	1.00034	OK
13	Direction Y	28.584	28.57	28.577	2.917	2.913	2.915	1.00069	OK
12	Direction Y	25.667	25.657	25.662	2.888	2.886	2.887	1.00035	OK
11	Direction Y	22.779	22.771	22.775	2.845	2.845	2.845	1.00000	OK
10	Direction Y	19.934	19.926	19.93	2.785	2.783	2.784	1.00036	OK
9	Direction Y	17.149	17.143	17.146	2.705	2.703	2.704	1.00037	OK
8	Direction Y	14.444	14.44	14.442	2.599	2.599	2.599	1.00000	OK
7	Direction Y	11.845	11.841	11.843	2.464	2.462	2.463	1.00041	OK
6	Direction Y	9.381	9.379	9.38	2.292	2.292	2.292	1.00000	OK
5	Direction Y	7.089	7.087	7.088	2.076	2.074	2.075	1.00048	OK
4	Direction Y	5.013	5.013	5.013	1.808	1.81	1.809	0.99945	OK
3	Direction Y	3.205	3.203	3.204	1.481	1.479	1.48	1.00068	OK
2	Direction Y	1.724	1.724	1.724	1.195	1.195	1.195	1.00000	OK
1	Direction Y	0.529	0.529	0.529	0.529	0.529	0.529	1.00000	OK

Table A.7 : B2 soft story irregularity for X-X direction

Story	EQ Direction	Story Height	Relative Story Displacement			Limit	Check
		m	$\Delta i(\text{max})/h_i$	$\Delta i(\text{min})/h_i$	$\Delta i(\text{Avg})/h_i$		≤ 2
24	Direction X	3.5	0.000890857	0.000890286	0.000890571	x	OK
23	Direction X	3.5	0.000907143	0.000907143	0.000907143	1.01861	OK
22	Direction X	3.5	0.000926857	0.000926857	0.000926857	1.02173	OK
21	Direction X	3.5	0.000952857	0.000953429	0.000953143	1.02836	OK
20	Direction X	3.5	0.000983143	0.000982571	0.000982857	1.03118	OK
19	Direction X	3.5	0.001015429	0.001015429	0.001015429	1.03314	OK
18	Direction X	3.5	0.001048857	0.001048857	0.001048857	1.03292	OK
17	Direction X	3.5	0.001082286	0.001082286	0.001082286	1.03187	OK
16	Direction X	3.5	0.001114	0.001114	0.001114	1.02930	OK
15	Direction X	3.5	0.001143143	0.001143143	0.001143143	1.02616	OK
14	Direction X	3.5	0.001168286	0.001167714	0.001168	1.02174	OK
13	Direction X	3.5	0.001188286	0.001188286	0.001188286	1.01737	OK
12	Direction X	3.5	0.001201143	0.001201714	0.001201429	1.01106	OK
11	Direction X	3.5	0.001207143	0.001207143	0.001207143	1.00476	OK
10	Direction X	3.5	0.001203429	0.001202857	0.001203143	0.99669	OK
9	Direction X	3.5	0.001188857	0.001188857	0.001188857	0.98813	OK
8	Direction X	3.5	0.001160857	0.001160286	0.001160571	0.97621	OK
7	Direction X	3.5	0.001117143	0.001117714	0.001117429	0.96283	OK
6	Direction X	3.5	0.001055143	0.001055143	0.001055143	0.94426	OK
5	Direction X	3.5	0.000970286	0.000970286	0.000970286	0.91958	OK
4	Direction X	3.5	0.000857429	0.000857429	0.000857429	0.88369	OK
3	Direction X	3.5	0.000709714	0.000709143	0.000709429	0.82739	OK
2	Direction X	4	0.000503	0.000503	0.000503	0.70902	OK
1	Direction X	4	0.00021325	0.00021325	0.00021325	0.42396	OK

Table A.8 : B2 soft story irregularity for Y-Y direction.

Story	EQ Direction	Story Height	Relative Story Displacement			Limit	Check
		m	$\Delta i(\max)/h_i$	$\Delta i(\min)/h_i$	$\Delta i(\text{Avg})/h_i$		≤ 2
24	Direction Y	3.5	0.000799143	0.000799143	0.000799143	x	OK
23	Direction Y	3.5	0.000804857	0.000804286	0.000804571	1.00679	OK
22	Direction Y	3.5	0.00081	0.000809429	0.000809714	1.00639	OK
21	Direction Y	3.5	0.000816286	0.000816286	0.000816286	1.00812	OK
20	Direction Y	3.5	0.000822571	0.000822	0.000822286	1.00735	OK
19	Direction Y	3.5	0.000828286	0.000827714	0.000828	1.00695	OK
18	Direction Y	3.5	0.000833429	0.000832857	0.000833143	1.00621	OK
17	Direction Y	3.5	0.000837143	0.000836	0.000836571	1.00412	OK
16	Direction Y	3.5	0.000839429	0.000838857	0.000839143	1.00307	OK
15	Direction Y	3.5	0.000839714	0.000839714	0.000839714	1.00068	OK
14	Direction Y	3.5	0.000838	0.000837429	0.000837714	0.99762	OK
13	Direction Y	3.5	0.000833429	0.000832286	0.000832857	0.99420	OK
12	Direction Y	3.5	0.000825143	0.000824571	0.000824857	0.99039	OK
11	Direction Y	3.5	0.000812857	0.000812857	0.000812857	0.98545	OK
10	Direction Y	3.5	0.000795714	0.000795143	0.000795429	0.97856	OK
9	Direction Y	3.5	0.000772857	0.000772286	0.000772571	0.97126	OK
8	Direction Y	3.5	0.000742571	0.000742571	0.000742571	0.96117	OK
7	Direction Y	3.5	0.000704	0.000703429	0.000703714	0.94767	OK
6	Direction Y	3.5	0.000654857	0.000654857	0.000654857	0.93057	OK
5	Direction Y	3.5	0.000593143	0.000592571	0.000592857	0.90532	OK
4	Direction Y	3.5	0.000516571	0.000517143	0.000516857	0.87181	OK
3	Direction Y	3.5	0.000423143	0.000422571	0.000422857	0.81813	OK
2	Direction Y	4	0.00029875	0.00029875	0.00029875	0.70650	OK
1	Direction Y	4	0.00013225	0.00013225	0.00013225	0.44268	OK

Table A.9 : Relative story displacement limit control for X-X direction.

Story	EQ Direction	di(max)	$\Delta i(\text{max})$	R	I	δi	hi	λ	Result	Check ≤ 0.008
24	Direction X	83.688	3.118	7	1	21.826	3.5	0.356	0.00222	OK
23	Direction X	80.57	3.175	7	1	22.225	3.5	0.356	0.00226	OK
22	Direction X	77.395	3.244	7	1	22.708	3.5	0.356	0.00231	OK
21	Direction X	74.151	3.335	7	1	23.345	3.5	0.356	0.00237	OK
20	Direction X	70.816	3.441	7	1	24.087	3.5	0.356	0.00245	OK
19	Direction X	67.375	3.554	7	1	24.878	3.5	0.356	0.00253	OK
18	Direction X	63.821	3.671	7	1	25.697	3.5	0.356	0.00261	OK
17	Direction X	60.15	3.788	7	1	26.516	3.5	0.356	0.00270	OK
16	Direction X	56.362	3.899	7	1	27.293	3.5	0.356	0.00278	OK
15	Direction X	52.463	4.001	7	1	28.007	3.5	0.356	0.00285	OK
14	Direction X	48.462	4.089	7	1	28.623	3.5	0.356	0.00291	OK
13	Direction X	44.373	4.159	7	1	29.113	3.5	0.356	0.00296	OK
12	Direction X	40.214	4.204	7	1	29.428	3.5	0.356	0.00299	OK
11	Direction X	36.01	4.225	7	1	29.575	3.5	0.356	0.00301	OK
10	Direction X	31.785	4.212	7	1	29.484	3.5	0.356	0.00300	OK
9	Direction X	27.573	4.161	7	1	29.127	3.5	0.356	0.00296	OK
8	Direction X	23.412	4.063	7	1	28.441	3.5	0.356	0.00289	OK
7	Direction X	19.349	3.91	7	1	27.37	3.5	0.356	0.00278	OK
6	Direction X	15.439	3.693	7	1	25.851	3.5	0.356	0.00263	OK
5	Direction X	11.746	3.396	7	1	23.772	3.5	0.356	0.00242	OK
4	Direction X	8.35	3.001	7	1	21.007	3.5	0.356	0.00214	OK
3	Direction X	5.349	2.484	7	1	17.388	3.5	0.356	0.00177	OK
2	Direction X	2.865	2.012	7	1	14.084	4	0.356	0.00125	OK
1	Direction X	0.853	0.853	7	1	5.971	4	0.356	0.00053	OK

Table A.10 : Relative story displacement limit checking for Y-Y direction.

Story	EQ Direction	di(max)	$\Delta i(\text{max})$	R	I	δi	hi	λ	Result	Check ≤ 0.008
24	Direction Y	60.325	2.797	7	1	19.579	3.5	0.356	0.00199	OK
23	Direction Y	57.528	2.817	7	1	19.719	3.5	0.356	0.00201	OK
22	Direction Y	54.711	2.835	7	1	19.845	3.5	0.356	0.00202	OK
21	Direction Y	51.876	2.857	7	1	19.999	3.5	0.356	0.00203	OK
20	Direction Y	49.019	2.879	7	1	20.153	3.5	0.356	0.00205	OK
19	Direction Y	46.14	2.899	7	1	20.293	3.5	0.356	0.00206	OK
18	Direction Y	43.241	2.917	7	1	20.419	3.5	0.356	0.00208	OK
17	Direction Y	40.324	2.93	7	1	20.51	3.5	0.356	0.00209	OK
16	Direction Y	37.394	2.938	7	1	20.566	3.5	0.356	0.00209	OK
15	Direction Y	34.456	2.939	7	1	20.573	3.5	0.356	0.00209	OK
14	Direction Y	31.517	2.933	7	1	20.531	3.5	0.356	0.00209	OK
13	Direction Y	28.584	2.917	7	1	20.419	3.5	0.356	0.00208	OK
12	Direction Y	25.667	2.888	7	1	20.216	3.5	0.356	0.00206	OK
11	Direction Y	22.779	2.845	7	1	19.915	3.5	0.356	0.00203	OK
10	Direction Y	19.934	2.785	7	1	19.495	3.5	0.356	0.00198	OK
9	Direction Y	17.149	2.705	7	1	18.935	3.5	0.356	0.00193	OK
8	Direction Y	14.444	2.599	7	1	18.193	3.5	0.356	0.00185	OK
7	Direction Y	11.845	2.464	7	1	17.248	3.5	0.356	0.00175	OK
6	Direction Y	9.381	2.292	7	1	16.044	3.5	0.356	0.00163	OK
5	Direction Y	7.089	2.076	7	1	14.532	3.5	0.356	0.00148	OK
4	Direction Y	5.013	1.808	7	1	12.656	3.5	0.356	0.00129	OK
3	Direction Y	3.205	1.481	7	1	10.367	3.5	0.356	0.00105	OK
2	Direction Y	1.724	1.195	7	1	8.365	4	0.356	0.00074	OK
1	Direction Y	0.529	0.529	7	1	3.703	4	0.356	0.00033	OK

Table A.11 : P- Δ effect limit controlling for X-X direction.

Story	EQ Direction	Δ_{iavg}	hi	story weight	V_i	theta	theta max	check
24	Direction X	3.117	3.5	10386.2145	2156.075	0.004290049	0.086	OK
23	Direction X	3.175	3.5	19465.3123	3038.806	0.005810775	0.086	OK
22	Direction X	3.244	3.5	28544.4102	3345.708	0.00790762	0.086	OK
21	Direction X	3.336	3.5	37623.5081	3530.567	0.010157173	0.086	OK
20	Direction X	3.44	3.5	46702.606	3655.554	0.01255678	0.086	OK
19	Direction X	3.554	3.5	55781.7038	3797.063	0.014917407	0.086	OK
18	Direction X	3.671	3.5	64860.8017	4013.333	0.016950926	0.086	OK
17	Direction X	3.788	3.5	73939.8996	4205.715	0.019027465	0.086	OK
16	Direction X	3.899	3.5	83018.9975	4344.771	0.021286086	0.086	OK
15	Direction X	4.001	3.5	92098.0953	4464.022	0.0235844	0.086	OK
14	Direction X	4.088	3.5	101177.193 2	4600.77	0.025685912	0.086	OK
13	Direction X	4.159	3.5	110256.291 1	4763.897	0.027501851	0.086	OK
12	Direction X	4.205	3.5	119391.139	4882.442	0.029378728	0.086	OK
11	Direction X	4.225	3.5	128525.986 9	4955.985	0.03130543	0.086	OK
10	Direction X	4.211	3.5	137660.834 7	5066.867	0.032687982	0.086	OK
9	Direction X	4.161	3.5	146795.682 6	5214.592	0.033467448	0.086	OK
8	Direction X	4.062	3.5	155930.530 5	5360.642	0.03375874	0.086	OK
7	Direction X	3.911	3.5	165065.378 4	5495.204	0.033565407	0.086	OK
6	Direction X	3.693	3.5	174200.226 2	5671.445	0.032409048	0.086	OK
5	Direction X	3.396	3.5	183335.074 1	5991.464	0.029690139	0.086	OK
4	Direction X	3.001	3.5	192469.922	6427.095	0.025677109	0.086	OK
3	Direction X	2.483	3.5	201604.769 9	6911.547	0.020693512	0.086	OK
2	Direction X	2.012	4	211224.242 7	7440.36	0.014279658	0.086	OK
1	Direction X	0.853	4	220843	7736.064	0.006087691	0.086	OK

Table A.12 :P- Δ effect limit controlling for Y-Y direction.

Story	EQ Direction	Δ_{avg}	hi	story weight	Vi	theta	theta max	check
24	Direction Y	2.797	3.5	10386.2145	2809.9692	0.002953794	0.086	OK
23	Direction Y	2.816	3.5	19465.3123	4345.1284	0.00360432	0.086	OK
22	Direction Y	2.834	3.5	28544.4102	5053.3094	0.004573798	0.086	OK
21	Direction Y	2.857	3.5	37623.5081	5380.7605	0.005707656	0.086	OK
20	Direction Y	2.878	3.5	46702.606	5612.1579	0.006842802	0.086	OK
19	Direction Y	2.898	3.5	55781.7038	5786.9688	0.007981251	0.086	OK
18	Direction Y	2.916	3.5	64860.8017	5786.9688	0.009337931	0.086	OK
17	Direction Y	2.928	3.5	73939.8996	6066.7906	0.010195837	0.086	OK
16	Direction Y	2.937	3.5	83018.9975	6244.8206	0.011155613	0.086	OK
15	Direction Y	2.939	3.5	92098.0953	6424.5518	0.012037585	0.086	OK
14	Direction Y	2.932	3.5	101177.1932	6610.6376	0.012821393	0.086	OK
13	Direction Y	2.915	3.5	110256.2911	6806.7987	0.013490591	0.086	OK
12	Direction Y	2.887	3.5	119391.139	6990.2961	0.014088192	0.086	OK
11	Direction Y	2.845	3.5	128525.9869	7172.2175	0.014566383	0.086	OK
10	Direction Y	2.784	3.5	137660.8347	7403.214	0.014790787	0.086	OK
9	Direction Y	2.704	3.5	146795.6826	7704.0085	0.014720927	0.086	OK
8	Direction Y	2.599	3.5	155930.5305	8068.6031	0.014350632	0.086	OK
7	Direction Y	2.463	3.5	165065.3784	8512.9305	0.013644992	0.086	OK
6	Direction Y	2.292	3.5	174200.2262	9041.2673	0.012617287	0.086	OK
5	Direction Y	2.075	3.5	183335.0741	9623.7942	0.011294039	0.086	OK
4	Direction Y	1.809	3.5	192469.922	10258.704	0.009697078	0.086	OK
3	Direction Y	1.48	3.5	201604.7699	10948.23	0.007786648	0.086	OK
2	Direction Y	1.195	4	211224.2427	11589.359	0.005444929	0.086	OK
1	Direction Y	0.529	4	220843	11891.826	0.002456014	0.086	OK

Table A.13 : Beam Rebar and stirrups for all structures.

Story	Name	Section		Rebar			Stirrups	
		height	width	Right	Mid	Left	Mid	Supports
1-10	B1	75	50	12 Φ 22	8 Φ 22	12 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B2	75	50	12 Φ 22	8 Φ 22	12 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B3	75	50	12 Φ 22	8 Φ 22	12 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B4	75	50	10 Φ 22	8 Φ 22	10 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B5	70	60	10 Φ 22	6 Φ 22	10 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B6	60	45	8 Φ 22	5 Φ 22	8 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B7	60	45	8 Φ 22	5 Φ 22	8 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B8	60	45	8 Φ 22	5 Φ 22	8 Φ 22	Φ 10 @ 20	Φ 10 @ 15
1-10	B9	75	50	10 Φ 22	6 Φ 22	10 Φ 22	Φ 10 @ 20	Φ 10 @ 15
11-24	B1	75	45	12 Φ 24	8 Φ 24	12 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B2	75	45	12 Φ 24	8 Φ 24	12 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B3	75	45	12 Φ 24	8 Φ 24	12 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B4	75	45	10 Φ 24	8 Φ 24	10 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B5	70	60	10 Φ 24	6 Φ 24	10 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B6	60	45	8 Φ 24	5 Φ 24	8 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B7	60	45	8 Φ 24	5 Φ 24	8 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B8	60	45	8 Φ 24	5 Φ 24	8 Φ 24	Φ 10 @ 20	Φ 10 @ 15
11-24	B9	75	50	10 Φ 24	6 Φ 24	10 Φ 24	Φ 10 @ 20	Φ 10 @ 15

Table A.14 : Columns rebar and ties for all structures.

Story	Column	Section		Min As (cm ²)	As (cm ²)	Rebar	Ties
		h	b				
1-12	C1	90	90	81	138	26 Φ 26	Φ 12 @ 7
1-12	C3	60	45	27	64	12 Φ 26	Φ 12 @ 10
13-24	C2	80	80	64	105	20 Φ 26	Φ 12 @ 7

Table A.15 : Shear walls rebar and ties for the entire structure.

Section		Longitudinal Reinforcement				Ties End Zones	Ties Center
Story	Name	b (cm)	h (cm)	End zone	Center		
1-5	P1	50	1040	29 Φ 22	62 Φ 18	Φ 14 @ 10	Φ 14 @ 10
1-5	P2	50	600	17 Φ 22	36 Φ 18	Φ 14 @ 10	Φ 14 @ 10
1-5	P3	50	600	17 Φ 22	36 Φ 18	Φ 14 @ 10	Φ 14 @ 10
6-10	P1	50	1040	29 Φ 22	62 Φ 18	Φ 14 @ 20	Φ 14 @ 10
6-10	P2	50	600	11 Φ 22	48 Φ 18	Φ 14 @ 20	Φ 14 @ 10
6-10	P3	50	600	15 Φ 22	82 Φ 18	Φ 14 @ 20	Φ 14 @ 10
11-18	P1	40	1040	29 Φ 18	62 Φ 16	Φ 12 @ 20	Φ 14 @ 10
11-18	P2	40	600	11 Φ 18	48 Φ 16	Φ 12 @ 20	Φ 12 @ 10
11-18	P3	40	600	15 Φ 18	82 Φ 16	Φ 12 @ 20	Φ 12 @ 10
19-25	P1	40	1040	29 Φ 16	62 Φ 12	Φ 12 @ 20	Φ 12 @ 10
19-24	P2	40	600	11 Φ 16	48 Φ 12	Φ 12 @ 20	Φ 12 @ 10
19-24	P3	40	600	15 Φ 16	82 Φ 12	Φ 12 @ 20	Φ 12 @ 10



APPENDIX B



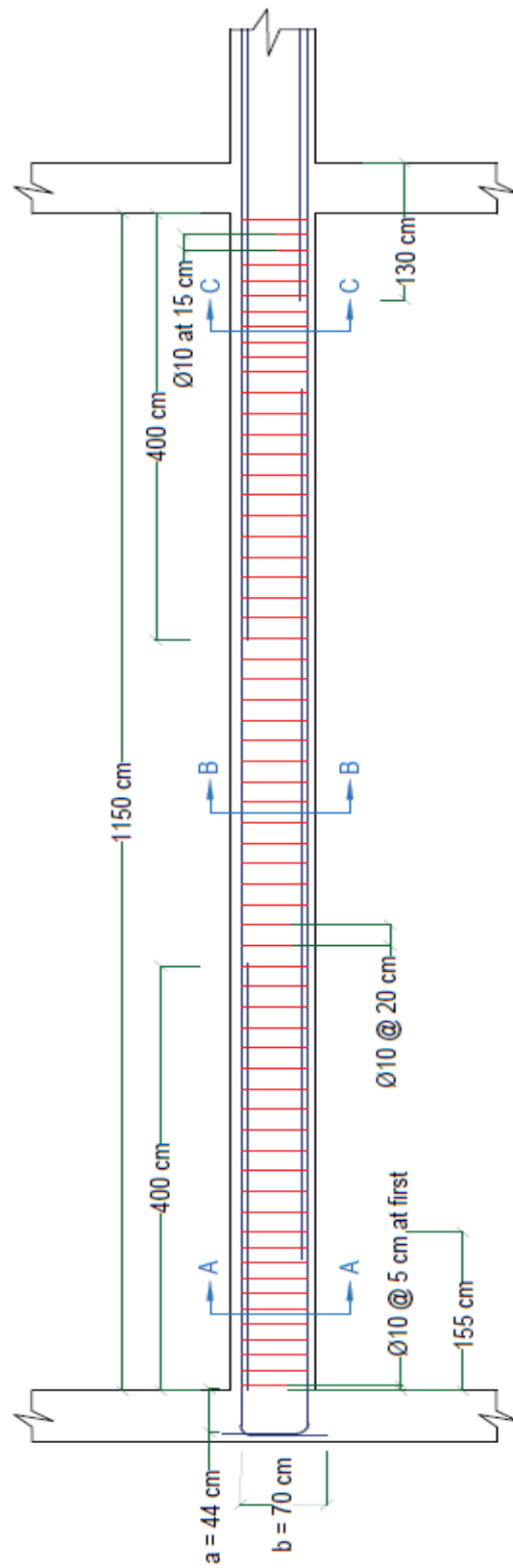
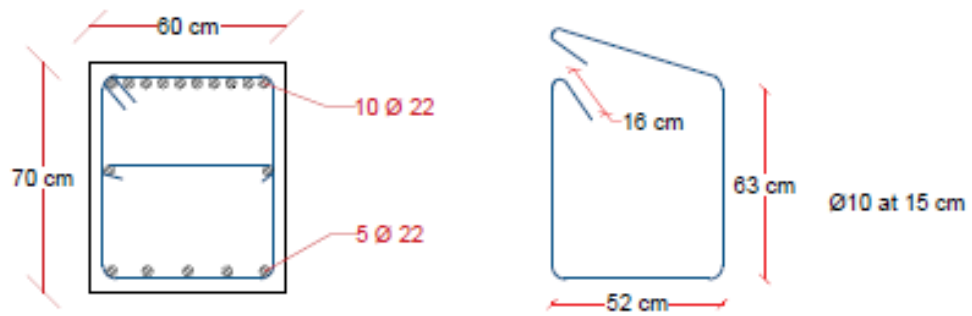


Figure B.1: Beam Longitudinal and Traverse rebar detailing, lapped splice and continuity of longitudinal reinforcement (beam B5 in 8th story)

Section B-B



Section A-A



Section C-C

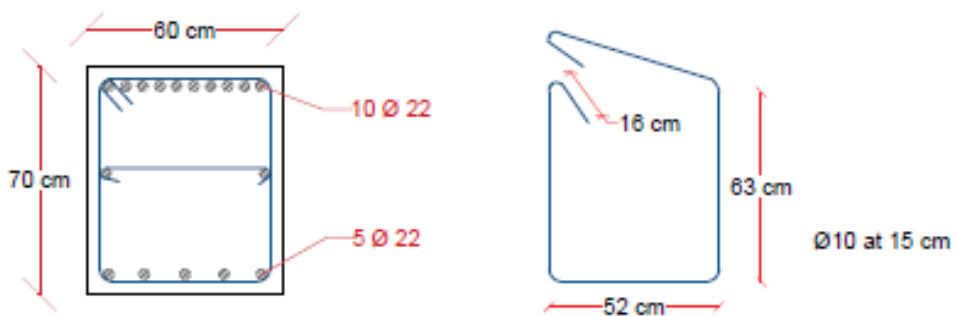


Figure B.2 : Beam sections detailing

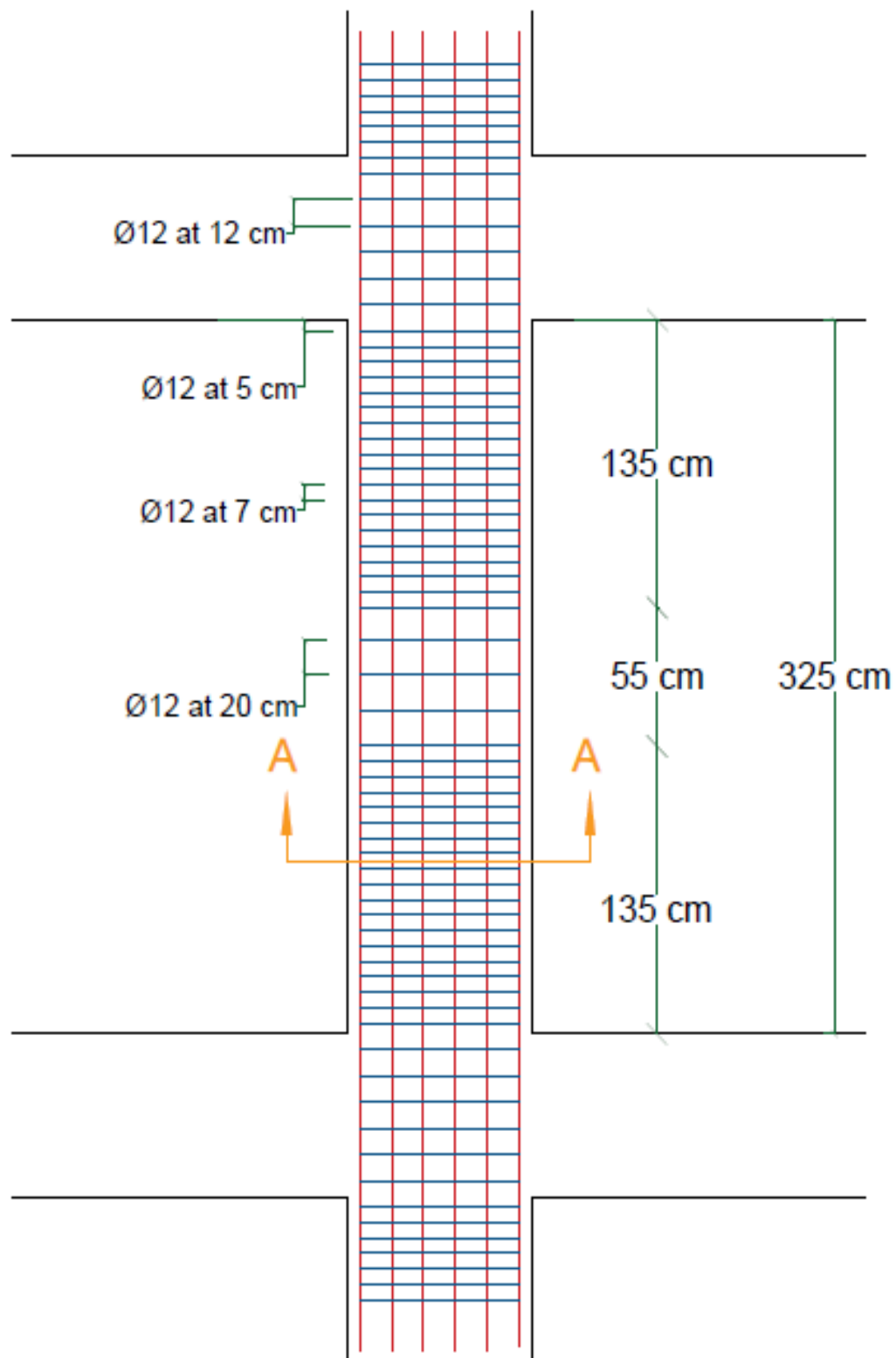


Figure B.3 : Column Rebar detailing (column C1 in the first basement)

Section A-A

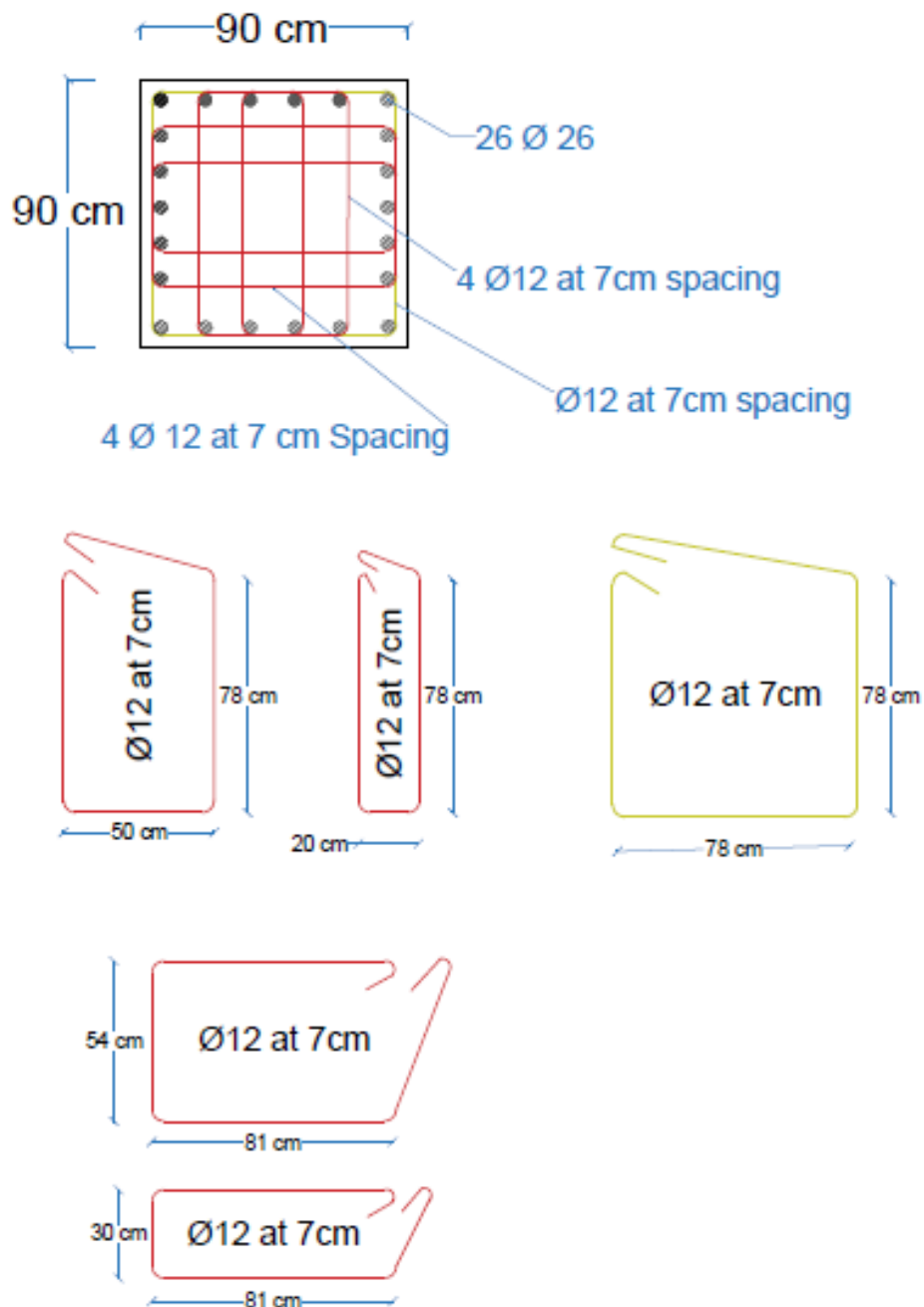


Figure B.4 : Column Section Detailing

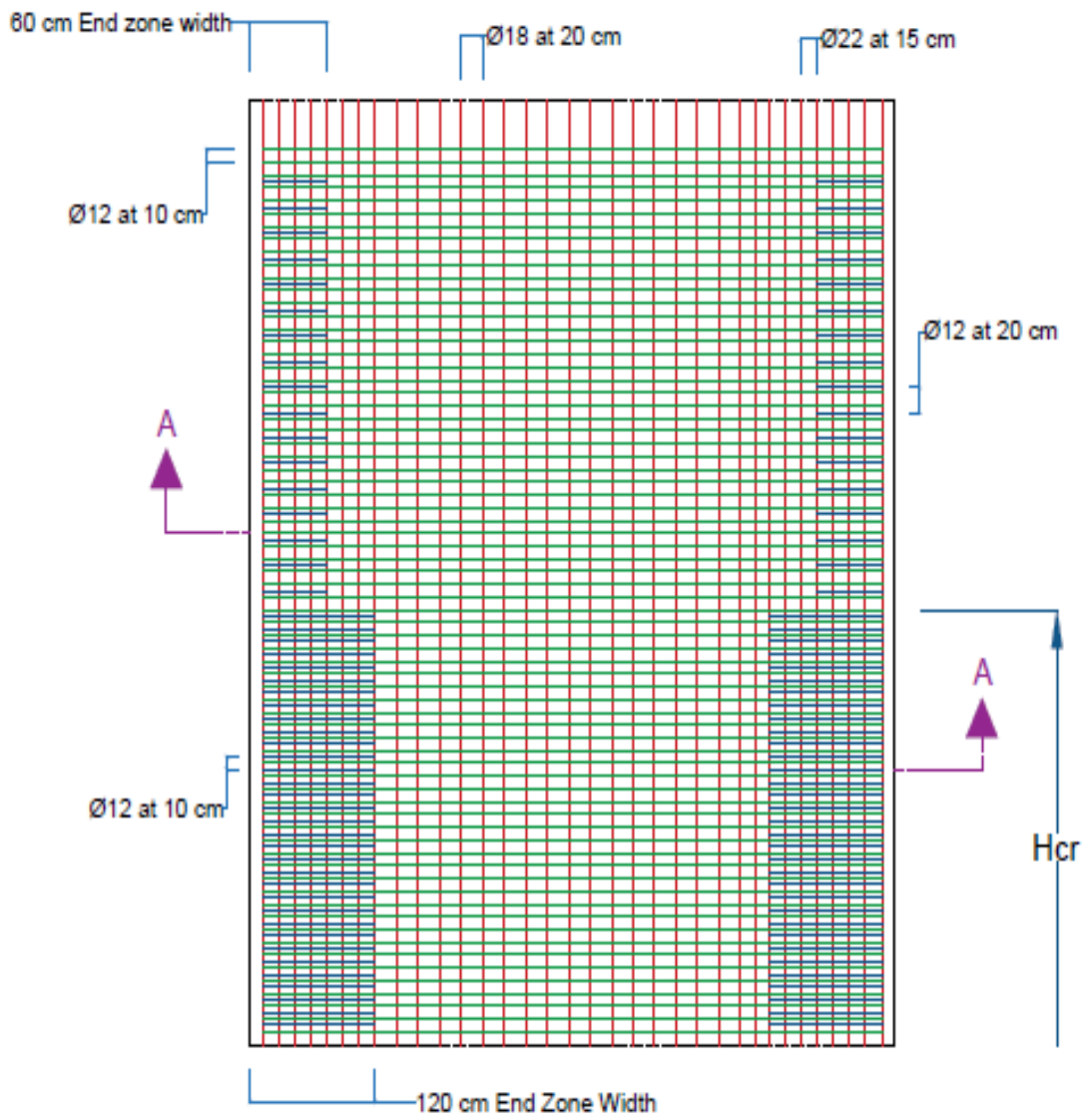
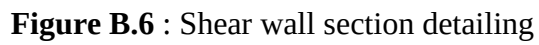


Figure B.5 : Shear walls ties placement (shear of P2 from H_{cr} to its extension)



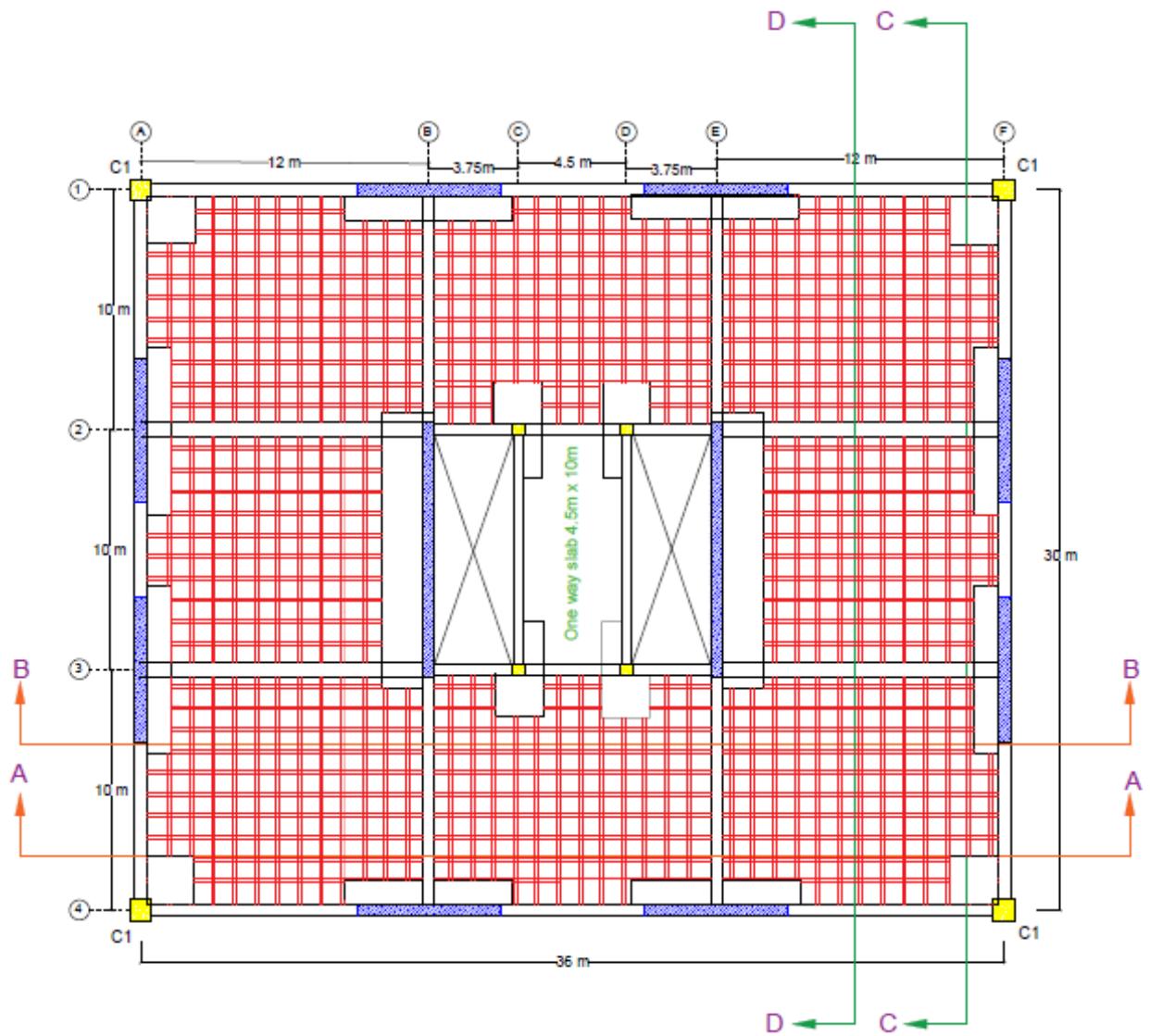


Figure B.7 : Plan view of Waffle Slab

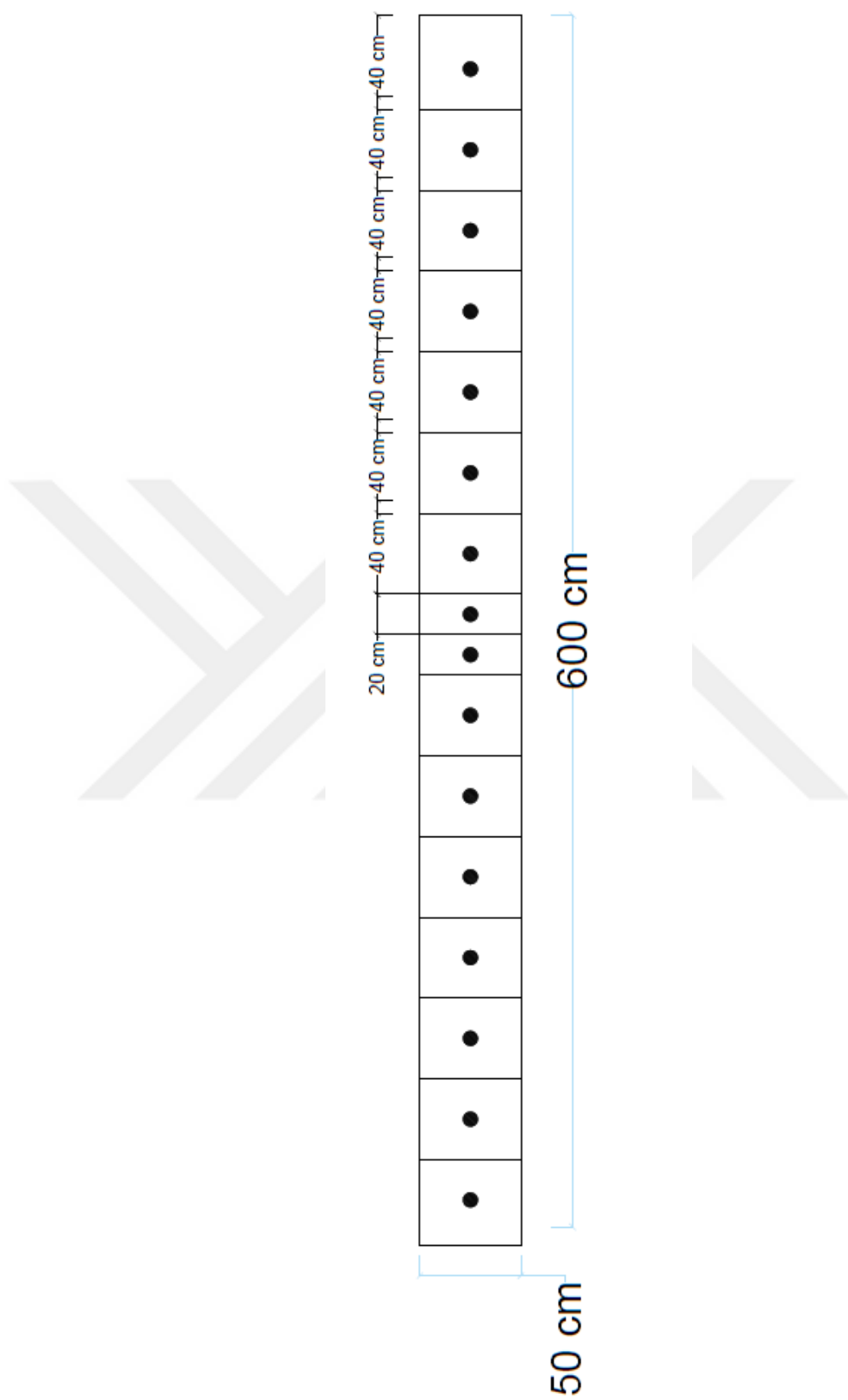


Figure B.8 : Shear wall fiber element detailing

Figure 10 illustrates the reinforcement details of the beam-column joints. The figure shows three cross-sectional views of a beam-column joint. The top view shows a beam with a total width of 11.45 cm. The middle view shows a beam with a total width of 11.45 cm. The bottom view shows a beam with a total width of 11.45 cm. The reinforcement details include top and bottom bars, stirrups, and cross-sections of the beam and column.

Figure B.9: Slab rebar detailing section A-A

Section B-B

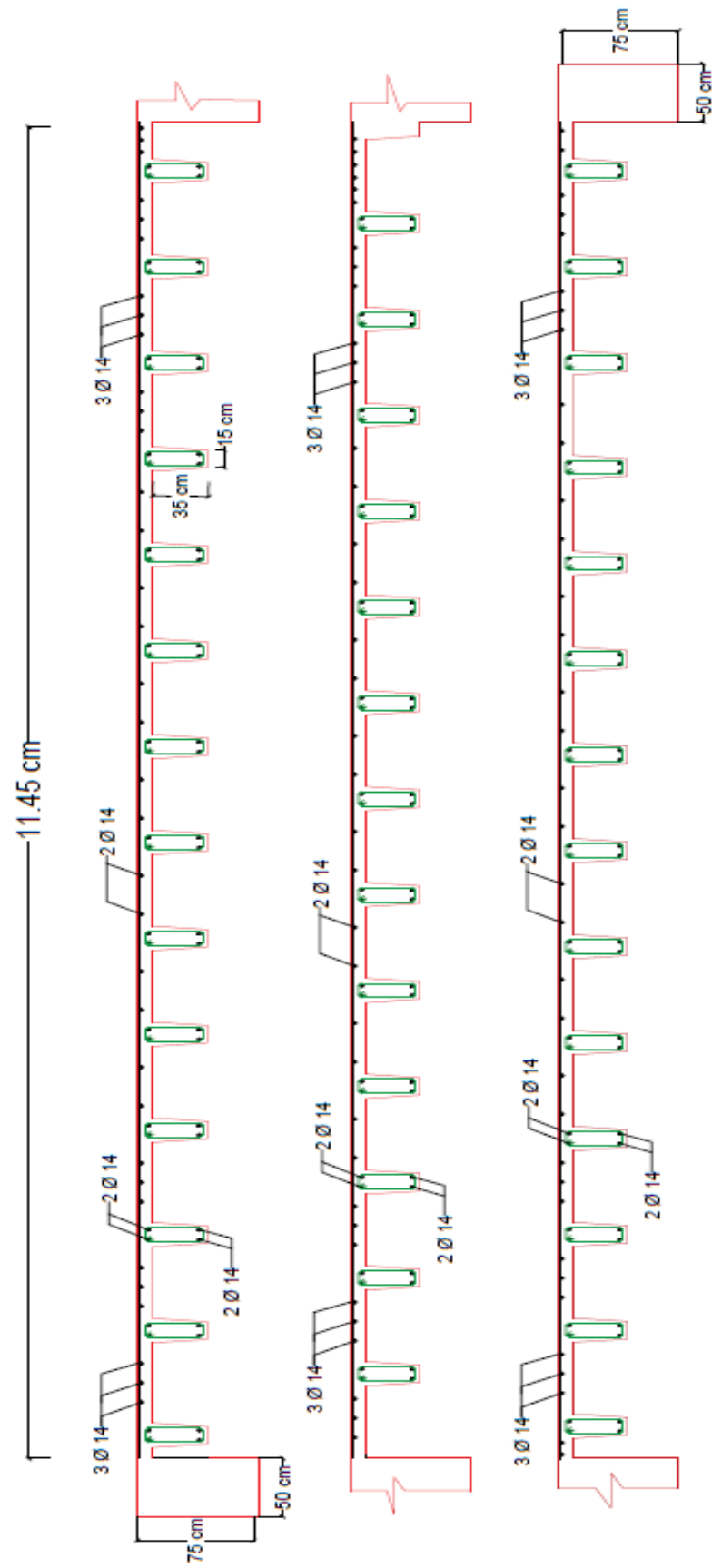


Figure B.10: Slab rebar detailing section B-B

Section C-C

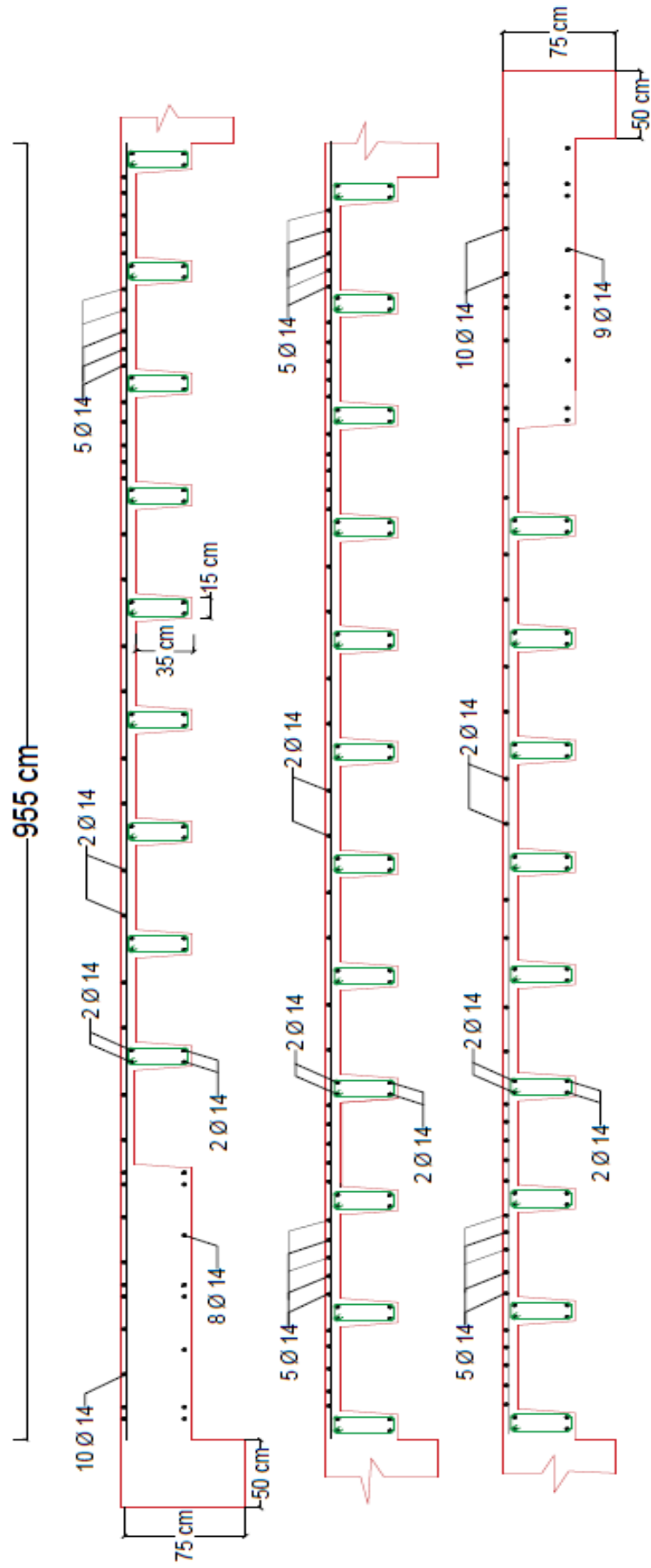


Figure B.11: Slab rebar detailing section C-C

Section D-D

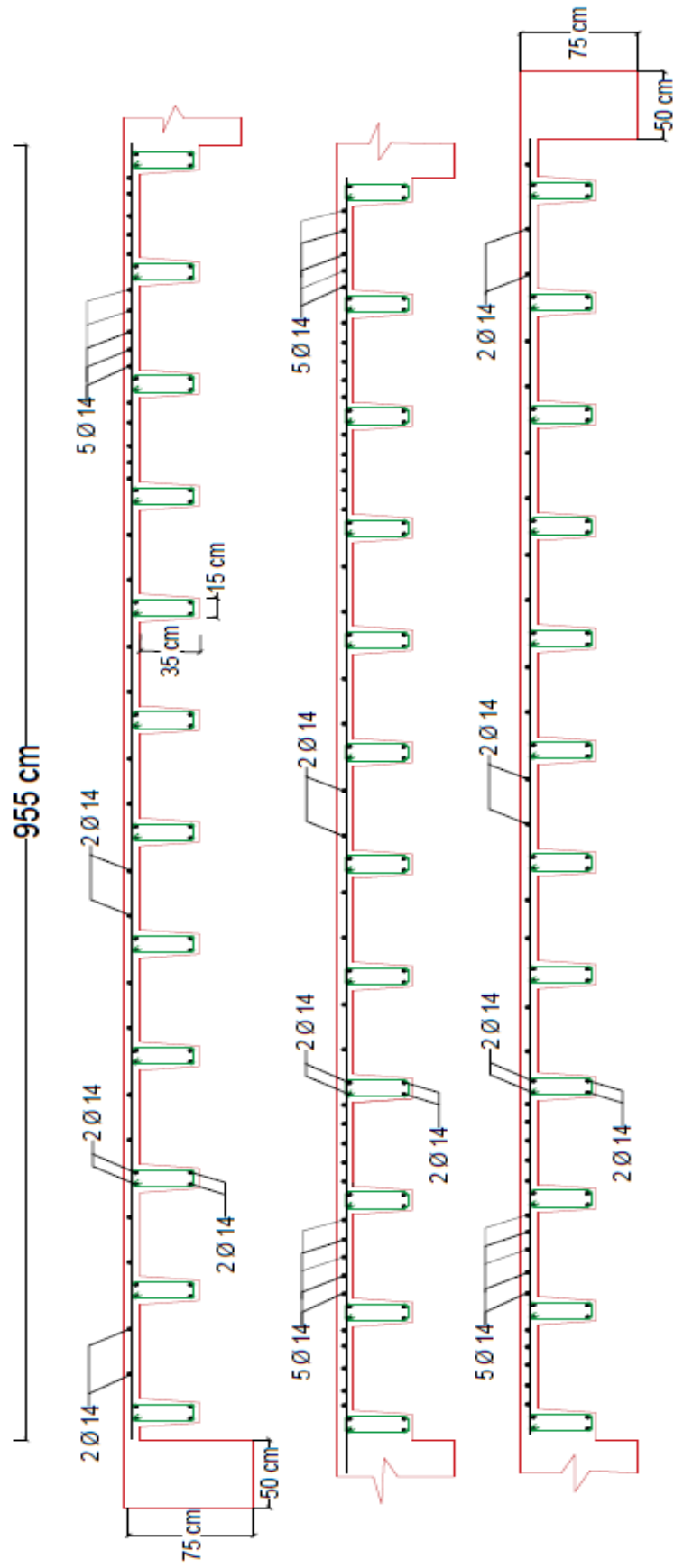
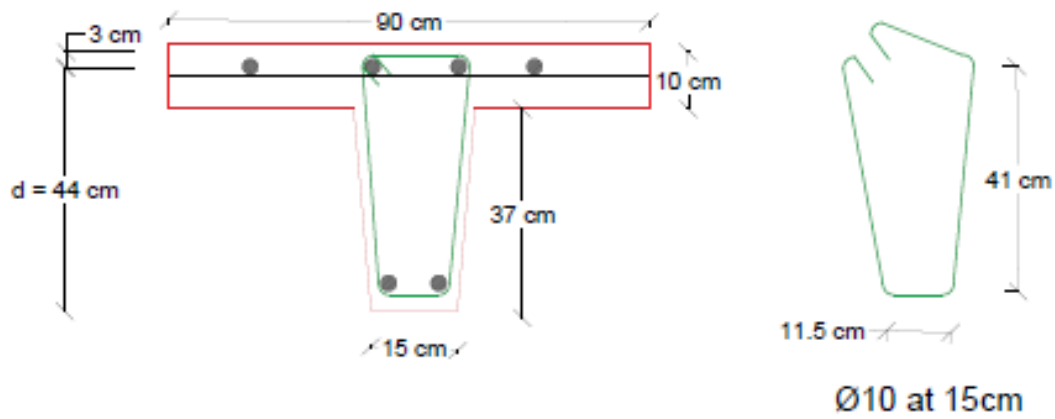


Figure B.12 : Slab rebar detailing section D-D

Middle strip layer A (X Direction)



Middle strip layer B (Y Direction)

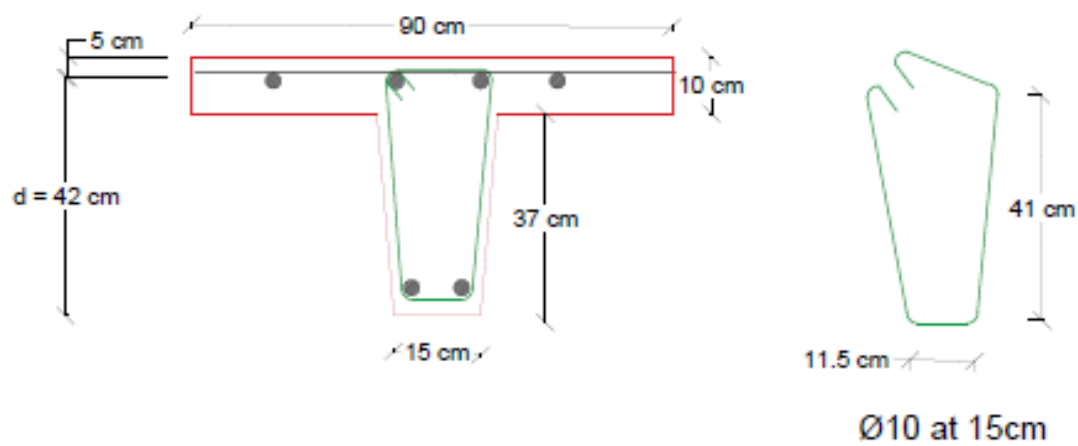
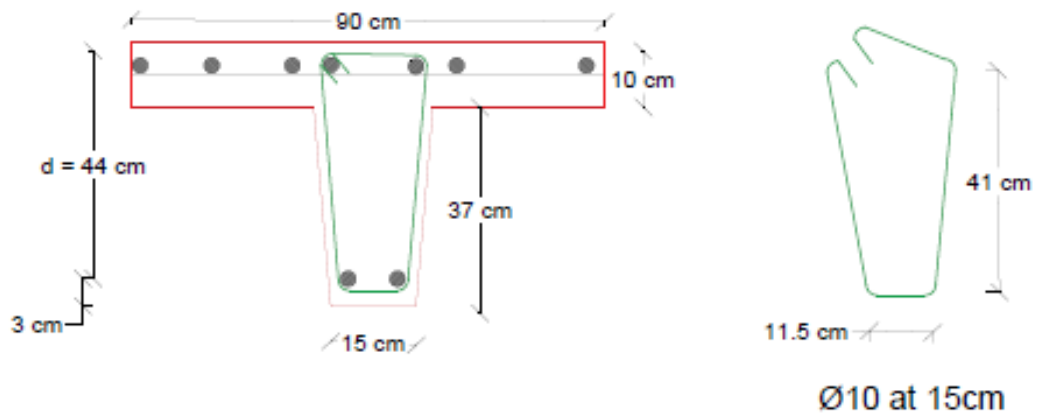


Figure B.13 : Waffle slab ribs section detailing in middle strip Layer A (X direction)
And Layer B (Y direction)

Column strip layer A (X Direction)



Column strip layer B (Y Direction)

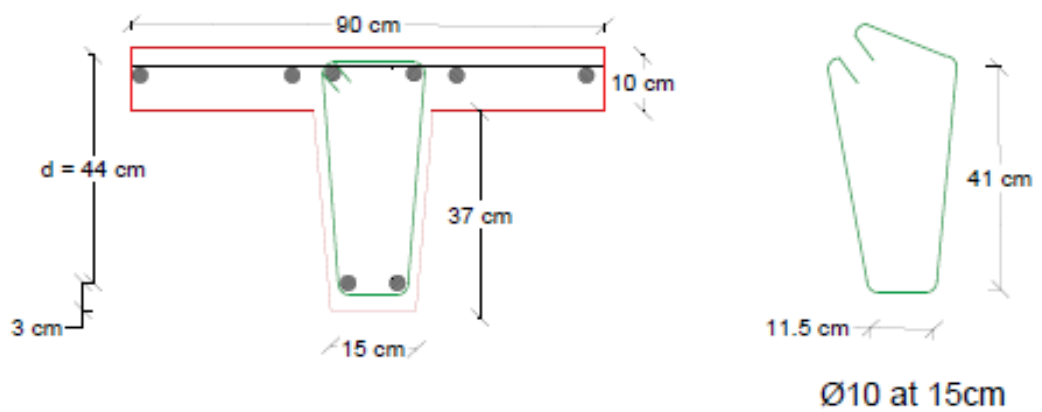
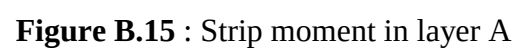


Figure B.14 : Waffle slab ribs section detailing in column strip layer A (X direction) and layer B (Y direction)



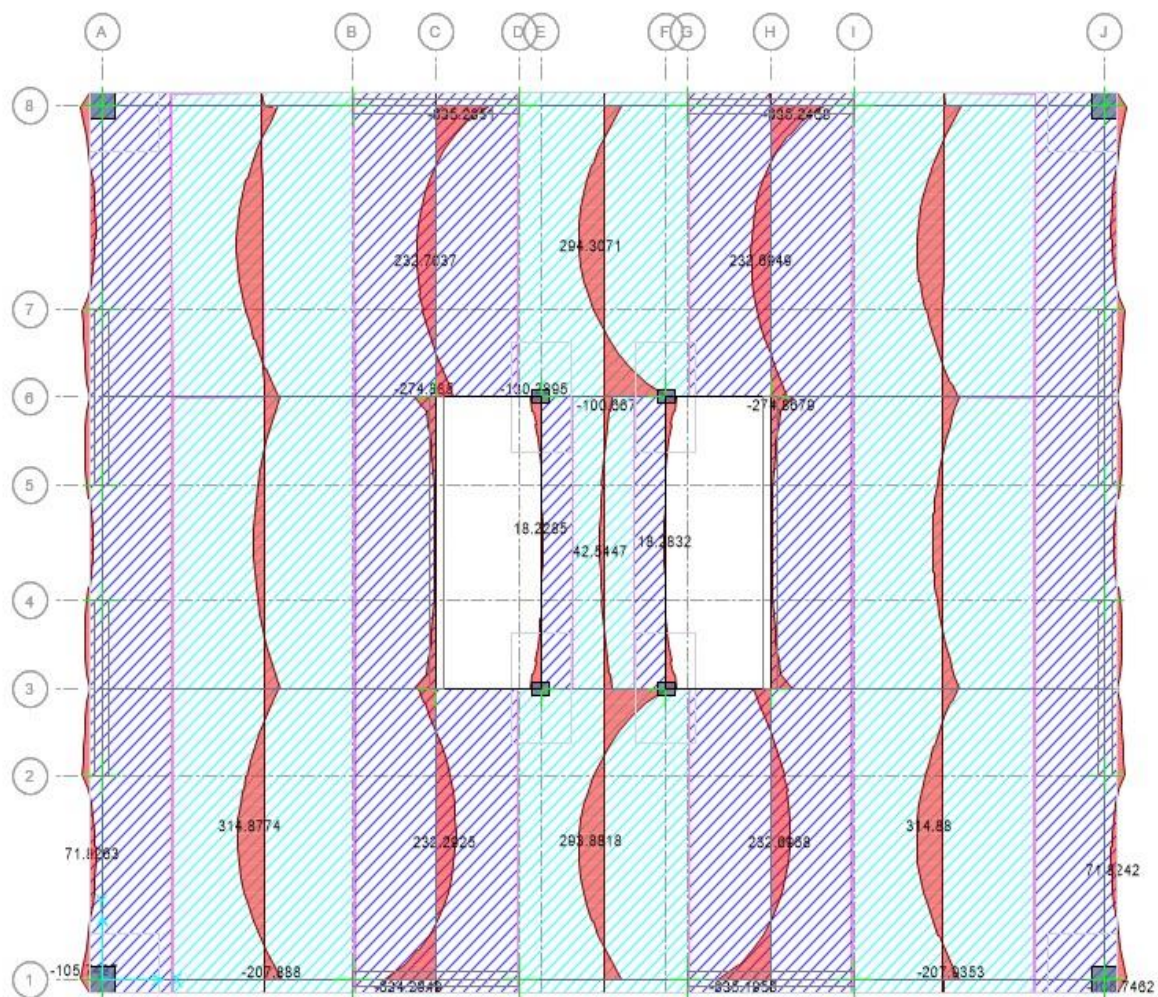


Figure B.16 : Strip moment in layer B



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