

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**THE PERFORMANCE ANALYSIS OF AN EXISTING RC
BUILDING STRENGTHEN WITH DIFFERENT TYPES OF STEEL
BRACES**

ABDIRIZAK YOUSUF AHMED
A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING PROGRAM

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THESIS SUPERVISOR
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GEBZE
2022

**T.C.
GEBZE TEKNİK ÜNİVERSİTESİ
FEN BİLİMLERİ ENSTİTÜSÜ**

**ÇELİK ÇAPRAZLAR İLE
GÜÇLENDİRİLMİŞ MEVCUT
BETONARME YAPININ PERFORMANS
ANALİZİ**

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YÜKSEK LİSANS TEZİ
İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI
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**GEBZE
2022**



YÜKSEK LİSANS JÜRİ ONAY FORMU

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SUMMARY

In recent years, the increasing destructive earthquake risk has led to the need to question the earthquake reliability of the vulnerable existing building stock which consists of buildings that are insufficient in strength and ductility. Therefore, it is of critical importance to determine the seismic performance of the existing buildings to reduce the structural damages in a possible future earthquake and to retrofit the buildings that do not satisfy the minimum performance criteria. Among the conventional strengthening techniques, steel braces can be considered as one of the most efficient solutions for seismic performance upgrading of RC buildings. In this study the seismic behavior of an existing RC frame building strengthened with different types of steel braces is investigated. The bracing systems are considered as X-braced frames, diagonal braced frames, inverted V braced frames. Nonlinear static pushover analyses have been conducted to investigate the structural capacities. Seismic performance level of the buildings has been determined by examining the base shear force – top story displacement behavior obtained from the analysis results and a comparative study has been performed. Finally, it is found that adding braces enhances the global capacity of the buildings in terms of strength, deformation and ductility and significantly contributes to the lateral stiffness of the RC building when compared to X bracing system was the most irresistible compared with the others.

Keywords: Existing RC buildings, Seismic performance, Steel bracing, Strengthening.

ÖZET

Son yıllarda artan tahribatlı deprem riski, dayanım ve süneklik açısından yetersiz yapılardan oluşan mevcut yapı stoğunun depreme karşı güvenilirliğinin sorgulanması ihtiyacını doğurmuştur. Bu nedenle, gelecekteki olası bir depremde yapısal hasarları azaltmak ve minimum performans kriterlerini sağlamayan binaların güçlendirilmesi için mevcut binaların deprem performansının belirlenmesi kritik önem taşımaktadır. Geleneksel güçlendirme teknikleri arasında çelik çaprazlar, betonarme binaların sismik performansının yükseltilmesi için en verimli çözümlerden biri olarak kabul edilmektedir. Bu çalışmada, farklı tipte çelik çaprazlarla güçlendirilmiş betonarme çerçeve taşıyıcı sisteme sahip mevcut bir binanın deprem performansı incelenmiştir. Çaprazlama sistemleri X çaprazlı çerçeveler, çapraz çaprazlı çerçeveler, ters V çaprazlı çerçeveler olarak göz önüne alınmıştır. Yapısal kapasiteleri araştırmak için doğrusal olmayan statik itme analizleri yapılmıştır. Analiz sonuçlarından elde edilen taban kesme kuvveti – üst kat yer değiştirme davranışı incelenerek binaların deprem performans düzeyi belirlenmiş ve karşılaştırmalı bir çalışma yapılmıştır. Son olarak, çaprazların eklenmesinin binaların küresel kapasitesini dayanım, deformasyon ve süneklik açısından geliştirdiği ve geleneksel güçlendirme yöntemleriyle karşılaştırıldığında betonarme yapının yanal rijitliğine önemli ölçüde katkıda bulunduğu bulunmuştur.

Anahtar Kelimeler: Mevcut Betonarme Binalar, Sismik Performans, Çelik Çaprazlama, Güçlendirme.

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LIST of ABBREVIATIONS and ACRONYMS

<u>Abbreviations</u>	<u>Explanation</u>
<u>And Acronyms</u>	
$R_a(T)$	: Earthquake load reduction coefficient
$S_{ae}(T)$	: Elastic spectral acceleration (m/s ²)
$S_{aR}(T_r)$	: Reduced spectral acceleration (m/s ²) for the rth natural vibration mode
Γ_{xN}	: x is the first mode contribution factor in the earthquake direction
$\epsilon_C^{(KH)}$	: Allowable confined concrete unit shortening limit for controlled damage performance level
$\epsilon_C^{(SH)}$	: Limit of permissible confined concrete unit shortening for limited damage performance level
$\epsilon_C^{(G\ddot{O})}$	: Allowable confined concrete unit shortening limit for failure prevention performance level
ϵ_{cu}	: Concrete compressive strain at the outermost fiber of the section
ϵ_s	: Reinforcing steel unit deformation
$\epsilon_s^{(G\ddot{O})}$	: Permissible reinforcing steel strain limit for failure prevention performance level
$\epsilon_s^{(KH)}$	: Permissible reinforcing steel strain limit for controlled damage performance level
$\epsilon_s^{(SH)}$	: Permissible reinforcing steel strain limit for limited damage performance level
ϵ_{su}	: Reinforcement unit elongation corresponding to maximum strength
η_{bi}	: Torsional irregularity coefficient defined on the i th floor
$\theta_p^{(G\ddot{O})}$	: Permissible plastic rotation limit (rad) for anti-collapse performance level
$\theta_p^{(KH)}$	: Permissible plastic rotation limit (rad) for controlled damage performance level
$\theta_p^{(SH)}$	: Permissible plastic rotation limit (rad) for limited damage performance level
θ_p	: Plastic rotation prompt

ρ_s	:	Volumetric ratio of transverse reinforcement present in the section
ρ_{sh}	:	Volumetric ratio of transverse reinforcement in the considered direction
ρ_{sm}	:	Volumetric ratio of transverse reinforcement required in the section
ϕ_p	:	Plastic curvature prompt
ϕ_t	:	Total curvature prompt
ϕ_u	:	Curvature before migration
Φ_{xin}	:	In buildings where floor slabs operate as rigid diaphragms, the horizontal component of the nth mode shape in the x-axis direction at the ith floor
Φ_{xN1}	:	Mode shape amplitude of the first mode in the x earthquake direction at the top of the building (on the Nth floor)
ϕ_y	:	Equivalent yield curvature
Φ_{yin}	:	In buildings where floor slabs operate as rigid diaphragms, the horizontal component of the nth mode shape on the i th floor in the y-axis direction
ω_B	:	The natural angular frequency corresponding to the characteristic period in the acceleration spectrum
ω_{we}	:	Mechanical reinforcement ratio of effective winding reinforcement
g	:	Gravitational acceleration
$A(T)$	:	Spectral acceleration coefficient
$G\ddot{O}$	:	Immigration Prevention Performance Level
I	:	Building importance coefficient
KH	:	Controlled Damage Performance Level
KK	:	Uninterrupted Usage Performance Level
n	:	Live load participation coefficient
R	:	Coefficient of behavior of the carrier system
SH	:	Limited Damage Performance Level
T	:	Building natural vibration period (s)
BKS	:	Building Use Class
BYS	:	Building Height Class
DTS	:	Earthquake Design Class

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1. INTRODUCTION

Earthquakes are one of the most catastrophic natural disasters and effective natural disasters that cause great loss of life and property all over the world. It is known that many major earthquakes have occurred in the world in the last century. The fact that Turkey is located on active earthquake zones due to its geographical location and most of the people live close to these regions means that it is at high risk in terms of earthquakes and causes significant amount of loss of life and property after the earthquakes.

In order to reduce the earthquake risk, it has become a necessity to make the necessary arrangements after the 1999 earthquakes in the earthquake regulations for the evaluation and strengthening of existing buildings in terms of earthquake safety. Many existing structures built according to previous regulations that do not comply with current standards and current regulations. In addition, structures that are not constructed in accordance with the design projects may be insufficient in terms of earthquake performance.

Among the most significant issues of earthquake engineering are improving the earthquake behavior of structures and increasing their performance levels. Post-earthquake damage has been emphasized in terms of earthquake engineering and increasing the earthquake resistance and performance levels of structures. Today, more attention is paid to earthquake resistant building design in countries that are generally located in earthquake zones. Since Turkey is located in an earthquake zone, it is of great importance that the buildings are built resistant to earthquakes and designed according to the standards.

The concept of earthquake resistant structure design reveals a structure designed, analyzed and applied in accordance with the current earthquake regulations and specifications. However, it does not provide the security of the building against earthquakes in all cases. The main principle of earthquake resistant structure design is to allow certain levels of damage that may occur in structures to prevent loss of life and property according to the intensity of the earthquake.

The purpose of earthquake regulations is not to prevent damage to structures for any situation, but to ensure life safety. In the event of an earthquake, the structure may

suffer structural and non-structural damage, but as long as the structure does not collapse, the regulations achieve their purpose. Since the earthquake regulations issued to make the buildings resistant to earthquakes cannot meet the requirements over time, they need to be constantly developed and renewed.

Turkish earthquake regulation foresees a design based on earthquake resistance in new buildings to be built. First of all, the safety of the buildings to be used after the earthquake and their performance during the earthquake should be known. In order to increase the earthquake safety of existing structures to the targeted level, repairs and reinforcement are required.

1.1. Scope

The significance of this study, the seismic behavior of an existing RC frame building strengthened with different types of steel braces is investigated. The bracing systems are regarded as X-braced frames, diagonal braced frames, and inverted V braced frames. The performance of various types of concentrically bracing systems with respect to un-braced buildings are investigated in depth in this study.

In the scope of the study, the selection of the exact type and size of columns and beams are taken into account in order to consider the equivalent qualities in the comparison of various bracing systems. The reason of the comparison is to understand which is the most affectable and resistible bracing and protectible during the seismic activities as well.

1.2. Thesis Outline

The present report begins by introducing the general purpose for the thesis work and the motivations that lead to explore the intended topic. The scope of this work is clearly stated in the first chapter to convey the essential concept of work. That has been accomplished in this study.

The second chapter addresses an overview about the retrofitting techniques considering on steel bracing system.

In the third chapter, describes design stages of the existing reinforced concrete structures that were employed in this study.

In the fourth chapter, the concept of performance-based design of retrofitting an existing RC building is explained. It provides the analysis methodology, which design is based on the nonlinear static pushover analysis.

The context of this research is described in the fifth chapter, along with the key aspects of numerical modeling in the ETABs software. This chapter also highlights a numerical example to clarify the stages pursued in the modelling process and the numerical results that followed

This study concludes with chapter six, which covers the significant closing statements gathered from this research study.

2. STRENGTHENING OF EXISTING RC BUILDINGS

The prevention of unexpected earthquake hazards has become more vital in recent centuries. Earthquake hazards protection involves the decreasing seismic possibility through the rehabilitation and retrofitting of the remaining buildings to enhance the safety of the seismic problems. There are various standard seismic retrofitting approaches for RC structures. These approaches can be classed primarily into two categories; (a) conventional techniques that focus on enhancing the structure's ductility, stiffness, and strength, and (b) innovative techniques for response modification which aim to diminish the impact of seismic forces on structures.

The conventional approach involves procedures like adding RC shear walls to the structure and jacketing RC columns. The essential advantage of these strategies is that they can be quickly designed and executed utilizing the conventional construction process. However, there are significant technical and practical drawbacks to conventional approaches. Strengthening the RC columns or adding shear walls may result in increases in the structure's weight, which generate more powerful earthquake forces. Furthermore, these approaches need extensive demolition and building work, limiting the structure's usage throughout the rehabilitation phase.

Innovative techniques for response modification include establishing seismic isolation devices at the building's base or hysteretic, frictional, and fluid viscous dampers in the building's structural frame system are examples of techniques. These devices are designed to adjust a structure's seismic response to mitigate the effects of seismic forces occurring on the structure. Response modification-based retrofitting technologies have substantial benefits compared to conventional techniques of seismic retrofitting. Response modification approaches are quite successful in mitigating the negative impacts of earthquakes.

The require for strengthening, repair of buildings and civil engineering works may emerge when they have been harmed in such a way that they are no longer fit for their ordinary utilize. In such situation, the structure cannot meet the next order of the same action with the accepted reliability or other accidental acts and consequently, the risk of loss of lives, as well as further structural and content damage, would be

unacceptable. Retrofitting refers to a strengthening intervention capable of restoring an appropriate level of safety against such events [1].

There may also be a need for strengthening or retrofitting for buildings or structures that have not been damaged previously. In most cases, the level of risk to be accepted for future damage is limited primarily by the need to reduce the risk of loss of life. However, when the structure is of considerable importance due to its function, the acceptance of damage is lower. Therefore, advanced techniques for strengthening and retrofitting are most commonly adopted for strategic buildings.

Seismic evaluation and retrofitting of existing structures are a complicated and tough topic that is constantly evolving in social, political, economic, technical, and scientific aspects [2].

2.1. Techniques for Innovative Seismic Retrofitting

Techniques for innovative seismic retrofitting include those that modify the seismic response of the structure as well as those based on enhancing ductility and strength of the RC members. In general, response modification devices mitigate the effects of seismic forces acting on structures. Examples of techniques for seismic retrofitting based upon response modification are seismic base isolation and damping devices employed within the building's structural method. Carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) applications are two examples of innovative seismic retrofitting approaches focused on increasing RC member strength and ductility. Techniques for innovative seismic retrofitting are more successful than conventional techniques of seismic retrofitting for improving building structural performance. Overall, these procedures will be most useful for the rehabilitation of structures whose owners want to be improved seismic performance and can afford the additional expenses involved in the manufacture and implementation of seismic isolators, energy dissipation devices and also, CFRP or GFRP material. Seismic isolation and energy dissipation techniques are more sophisticated concepts that need more extensive design and comprehensive research than conventional techniques rehabilitation options [3].

2.1.1. Techniques for Seismic Base Isolation

Seismic isolation is a response modification approach based upon the deployment of devices known as isolators, which are often positioned between columns and foundations at the building's base. The building is dissociated from horizontal elements of the ground motion by placing seismic isolators, which have extremely low horizontal stiffness, at the building's base. Therefore, when the building is due to seismic forces, most of the deformation happens in the isolators instead of in the building, as illustrated in Figure 2.1 which minimizes the seismic stress and displacement demand on the building's structural elements [4].

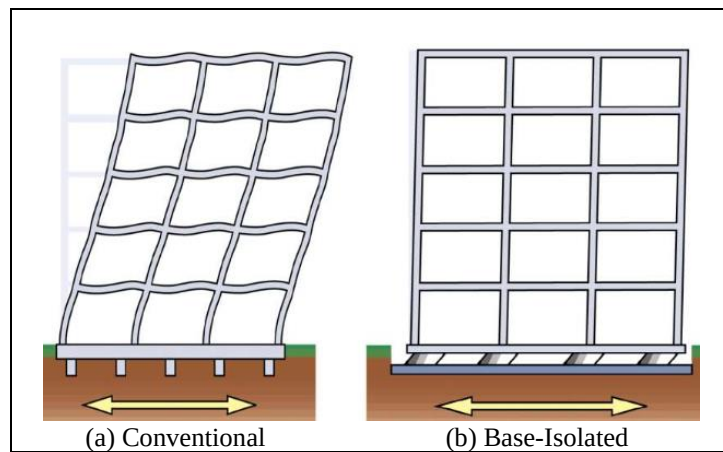


Figure 2.1: Behavior of base isolated and fixed base buildings.

2.1.2. Fiber Reinforced Polymer

Advanced compound materials such as carbon fiber reinforced polymer (CFRP) is a significantly stronger and lighter material than steel. Analytical and experimental outcomes have displayed that wrapping structural elements (beams, columns, and walls) with CFRP sheets enhance their ductility and strength without adding stiffness to the components [5]. CFRP's high modulus of elasticity and strength make it perfect for use as confinement for RC columns to improve their axial strength and ductility. Figure 2.2 illustrates a close-up view of CFRP.

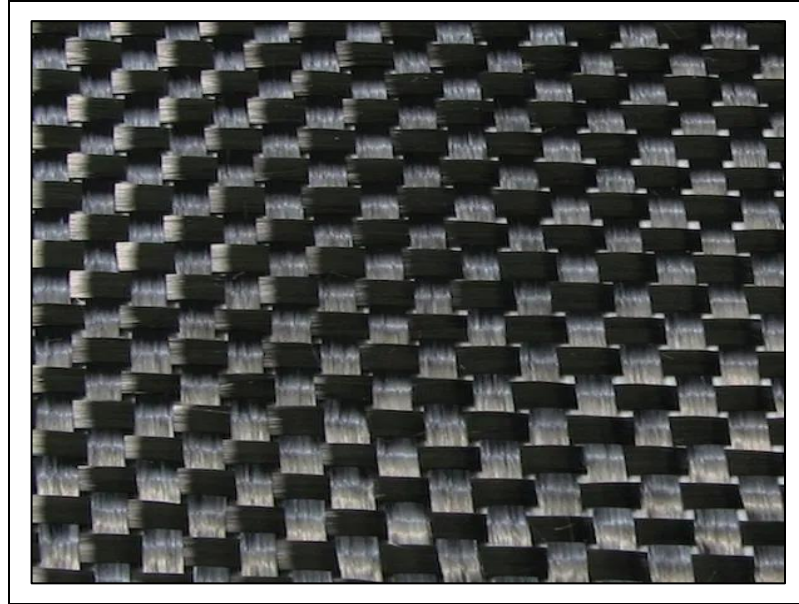


Figure 2.2: Close up picture of CFRP.

2.2. Techniques for Conventional Retrofitting

The conventional seismic design technique attempts to dissipate energy during earthquake excitations through the inelastic cyclic deformation of a building's structural components. In this technique, the energy dissipation is usually focused on the beam ends where the weak beam strong columns approach is applied [6]. Buildings with an insufficient seismic capability are usually unable to dissipate such energy. Thus, conventional seismic rehabilitation approaches primarily aim to increase the ductility and strength of a building's structural system. conventional rehabilitation of structures can be provided by local and global retrofitting of the lateral load conveying system [7]. Global retrofitting methods are typically utilized when retrofitting a few elements of the building that may not be suitable stand-alone solutions, especially when the structure lacks the stiffness required to meet code-imposed drift requirements. Local retrofitting is often employed when the number of elements to be retrofitted is restricted, and the drift criteria of the structure are delighted. In such circumstances, enhancing the structure's lateral stiffness by presenting shear walls or steel braces in the structural system provides a realistic solution [8]. In particular cases, both global and local retrofitting procedures are employed in parallel to improve a building's seismic performance.

2.2.1. Techniques for Local Retrofitting

Strengthening of insufficient columns is one of the commonly accepted approaches utilized to improve the seismic performance of moment-resisting RC frames [9]. The conventional techniques for strengthening columns, like concrete jacketing and steel jacketing are obtainable in the literature. Figure 2.3 illustrates the authentic and RC jacketed column section. Jacketing with RC layers enhances the columns' axial and shear load capability. This approach can also enhance flexural capacity, but it's not an easy procedure due to the requirement of ensuring continuity of the longitudinal bars. Forming a sufficient connection between the old and renewed concrete is an essential aspect of column retrofitting with concrete jacketing [10].

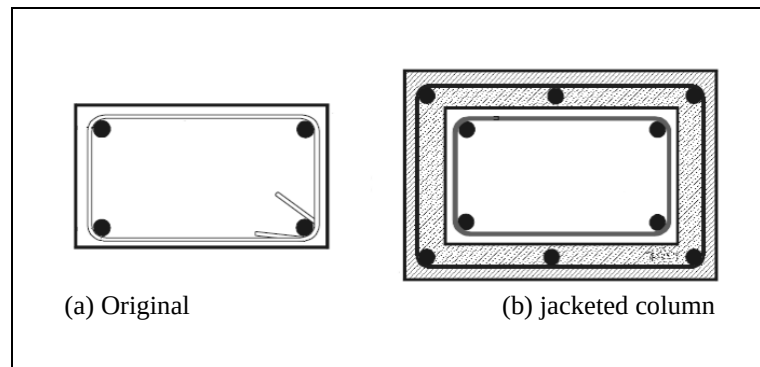


Figure 2.3: R.C. jacketed column sections.

In summary, construction long periods during seismic retrofitting, the complexity of building procedures, and increasing member sizes through column jacketing are some of the major drawbacks of local seismic retrofitting approaches [2].

2.2.2. Techniques for Global Retrofitting

Generally, global standard retrofitting is often applied when the overall structural lateral load resisting mechanism is deemed to be inadequate [11]. In addition to steel bracings, shear walls, infill walls, and wall-thickening are examples of global conventional seismic retrofitting procedures [7], [11]. RC shear walls are employed to mitigate stiffness eccentricities in the building or to improve a building's capacity to carry lateral loads. The newly added shear walls are often positioned on the structure's

outside perimeter, minimizing inner interference [7]. However, in some circumstances, the existence of numerous windows surrounding the structure may be a challenge when retrofitting shear wall installations.

2.2.3. Techniques for Steel Bracing

Steel bracing is one of the ways that utilized the protection against seismic hazards due to lateral loads caused by the earthquake loads. So, lateral loads might be wind loads or seismic loads. Bracing members typically carry axial loads as a result of seismic activity, with internal compression and tension. Steel braces are amongst the most often used and potentially beneficial technical methods for enhancing the seismic safety of existing RC structures. Steel bracing methods are widely employed in steel structures, and numerous models exist to characterize their response to cyclic actions caused by seismic excitations. Steel braced frames are those that mainly develop earthquake resistance through axial forces in their components. Also, it's used to rescue steel or RC structures from seismic loads.

However, the usage of steel bracing systems in the seismic retrofitting of existing RC buildings is likewise an appealing technology, as it is generally characterized by excellent architectural and functional compatibility concerning the original purpose of the existing structure. Thus, a recent series of research targeted developing design criteria for reinforcing RC frames using viscous dissipation steel bracings. In principle, steel bracing in RC frames can be designed using the well-established capacity design philosophy originally formulated for designing new structures. So far, followed two fundamental alternative approaches are formulating design standards for steel bracing systems in existing RC frames. They are generally known as force-based and displacement-based techniques [12].

Steel bracing systems are divided into two categories: a) eccentric braced frame (EBFs); and b) concentric braced frame (CBFs). Thus, this study is concerned with CBFs system. The bracing systems are considered as X-braced frames, diagonal braced frames, and inverted V braced frames.

2.2.3.1. Eccentric Braced Frame

Eccentric braced frame (EBFs) are widely utilized as a lateral load resistant system, especially in high seismic provinces, due to their high ductility, stable and dependable yield mechanisms. Bending and shear in an active link are used to transfer the actions to the braces. This link prevents the bracing from buckling. One of the most significant members of this bracing system is the active link, and this member must be designed to be elastic at low loads and inelastic when the structure is overloaded. Thus, the system consumes a large amount of energy. EBFs can be employed in a variety of forms, depending on the location of the link beam. They have four major components: a link beam, a non-link beam, bracing, and a column [13].

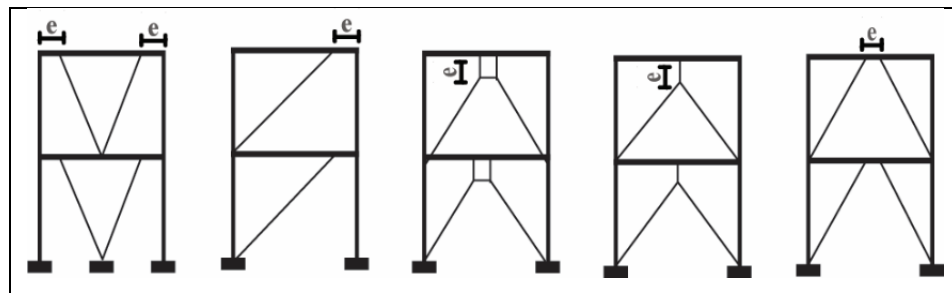


Figure 2.4: Eccentric Braced Frame.

2.2.3.2. Concentric Braced Frame

Concentric braced frame (CBFs) are a type of bracing employed to preserve the structure from the lateral loads. Since they have high strength and stiffness, making them effective in resisting lateral loads. When subjected to less favorable seismic response, they have a low drift capacity and a high acceleration. In order to enhance inelastic drift capacity a unique class of CBFs is proportioned and detailed. The usage of CBFs as structural seismic-force-resisting systems has a long history. Based on research performed during the last twenty years or so, modern seismic codes now include provisions to design ductile concentric braced frames called special concentrically bracing (SCBF). CBFs come in a variety of shapes and sizes, including Chevron, diametric, and cross [14], [15].

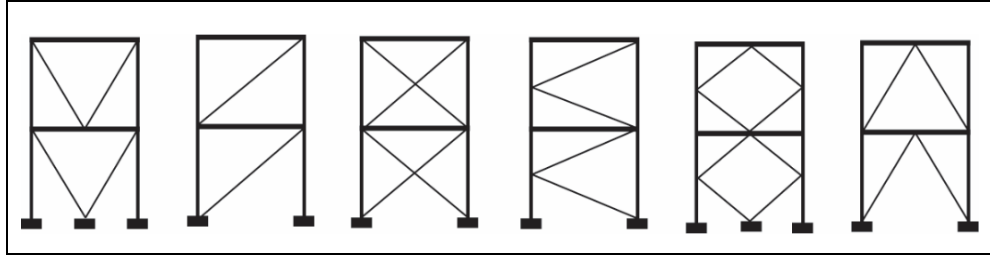


Figure 2.5: Concentric Braced Frame.

2.3. Assessment and Retrofitting

Seismic assessment of an existing structure is the detailed examination to establish the features of the structure as it stands. It attempts to identify specific construction weaknesses and inadequacies. Ersoy (1998) reminds data gathered during the evaluation step is utilized to analyze the structure's seismic performance. As a consequence of this assessment, the structural engineer determines whether to repair or strengthen the structure [2].

Structural retrofitting can be described as the procedure to bring the structural system or particular of the structural members to a set level of performance. Depending on the condition of the structure and the intended use, rehabilitation can be categorized as: (I) strengthening; or (II) repair. Structural strengthening is the process of enhancing the existing capacity (in terms of strength, ductility, or stiffness) of an undamaged structure or to a certain high-performance level of a structural member. Structural repair is the rehabilitation of a damaged structure or of a structural member with the objective of improving capability back to its pre-damage state [1], [2], [16].

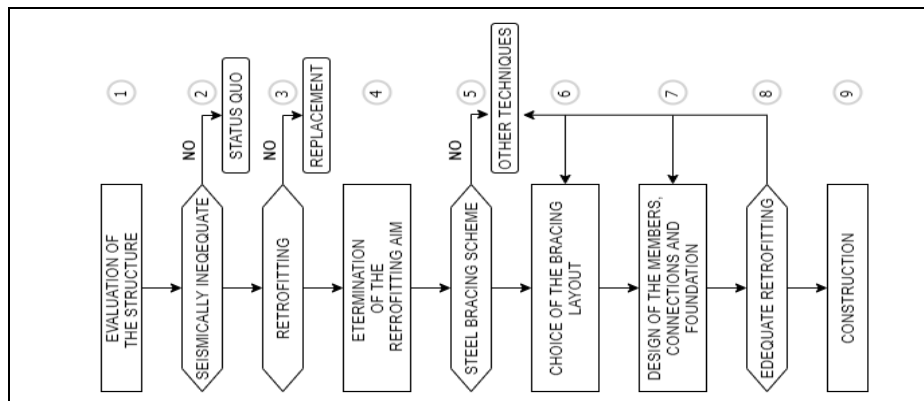


Figure 2.6: Bracing Procedure.

2.3.1. Braces Modeling

Braces must be modelled as columns with a potential plastic hinge located in the middle of the brace to account for both primary axial effects and secondary flexural effects. It is necessary to model the nonlinear axial load–axial deformation behavior of braces at the plastic hinge. Braces loaded in compression, the parameter shall represent total elastic and plastic axial deformation. If rigid plastic hinges are used, only plastic axial deformation may be considered. The reduction in strength of a brace after buckling shall be included in the model. For braces in tension, the parameters shall be the axial deformation at development of the expected tensile yield load in the brace.

Modern seismic regulations for steel place a focus on section compactness, component slenderness, and seismic specifying of connections to guarantee ductile behavior and acceptable performance. Thus, when modeling inelastic deformation capacities for non-ductile connections and components of older existing frames, the modeling parameters that are applicable to ductile detailing and compact sections as presented in this section should be used with caution [17].

2.3.2. Bracing Layout

The influence of retrofitting on the flow of forces through the structure must be considered when laying out the bracing system. During the process, it is critical to avoid the creation of new unfavorable weak connections. From a structural perspective, it may be preferable to brace as many bays of the frame as necessary. So that improvements in strength and stiffness are distributed equally. However, financial and functional reasons may limit the number of braced bays [16].

The designer must choose a bracing pattern while laying out the bracing system. Three possible patterns are presented during this study are:- (I) Diagonal Bracing System: this type of bracing consists of just one component connecting two diagonal joints. (II) X-Bracing System: this is a type of bracing occurs when two bracing members cross towards the middle of their length. (III) Inverted V Bracing System: entails the two members meeting at the upper horizontal member's center point.

2.3.3. Cross Section Conditions

Head width/thickness, body height/thickness and diameter/wall thickness ratios of the cross-sections of the elements in systems to be designed with high and limited ductility levels should not exceed the values given in Figure 2.7 [18].

Eleman Tanımı	Narinlik Oranı	Sınır değerler		Açıklama
		Süneklik Düzeyi Yüksek Elemanlar, λ_{hd}	Süneklik Düzeyi Sınırlı Elemanlar, λ_{md}	
Dikdörtgen kutu kesitler	b/t			
Yapma dikdörtgen kutu kesitler ve I kesitinden kutu şeklinde teşkil edilmiş kesitlerin başlıkları	b/t	$0.55 \sqrt{\frac{E}{F_y}}$	$0.64 \sqrt{\frac{E}{F_y}}^{[a]}$	
I kesitinden kutu şeklinde teşkil edilmiş kesitlerin yan levhaları ve çapraz eleman olarak kullanılacak yapma kutu kesitler	h/t			
Çapraz eleman olarak kullanılacak I veya yapma I kesitlerin gövdeleri	h/t_w	$1.49 \sqrt{\frac{E}{F_y}}$	$1.49 \sqrt{\frac{E}{F_y}}$	
Kiriş veya kolon olarak kullanılacak I veya yapma I kesitlerin gövdeleri ^[b]	h/t_w	$C_a \leq 0.125$ ise $2.45 \sqrt{\frac{E}{F_y}} (1 - 0.93 C_a)$	$C_a \leq 0.125$ ise $3.76 \sqrt{\frac{E}{F_y}} (1 - 2.75 C_a)$	
Kiriş veya kolon olarak kullanılacak I profilinden kutu şeklinde teşkil edilen enkesitlerin yan levhaları	h/t	$C_a > 0.125$ ise $0.77 \sqrt{\frac{E}{F_y}} (2.93 - C_a) \geq 1.49 \sqrt{\frac{E}{F_y}}$	$C_a > 0.125$ ise $1.12 \sqrt{\frac{E}{F_y}} (2.33 - C_a) \geq 1.49 \sqrt{\frac{E}{F_y}}$	
Kiriş veya kolon olarak kullanılacak yapma kutu enkesitlerin gövdeleri	h/t	$C_a = \frac{\Omega_c P_u}{F_y A}$, $\Omega_c = 1.67$ (GKT) $C_a = \frac{P_u}{\phi_c (F_y A)}$, $\phi_c = 0.90$ (YDKT)		
Boru enkesitli elemanlar	D/t	$0.038 \frac{E}{F_y}$	$0.044 \frac{E}{F_y}^{[c]}$	

Figure 2.7: Cross Section Conditions.

2.4. Performance Analysis with Regard to Nonlinear Analysis Methods

The definition of performance analysis considering nonlinear analysis methods is regulated in TBDY-2018. The purpose of the nonlinear analysis methods to be used for the determination of the structural performance of existing or reinforced buildings under the influence of earthquakes and for the strengthening calculations is to calculate the plastic deformation and plastic rotation demands for ductile behavior and the internal force demands for brittle behavior for a given earthquake. Following that, these demand sizes will be compared to strain and internal force capabilities, and structural performance will be evaluated at the section and building levels.

2.4.1. Nonlinear Static Pushover Analysis

The nonlinear analysis method for buildings by gradually increasing the horizontal loads under constant vertical loads is called the "Static Pushover Analysis Method". Since this method more realistically represents the behavior of the building during an earthquake, it allows for more accurate calculations. In the static pushing method, the deformation behavior of all the elements of the building is defined. In this calculation method, the plasticity capacity of the material outside the elasticity limits is also used.

The nonlinear analysis can't be regarded apart from plastic hinges. Plastic hinges can be described as a form of the cross-section that represents the performance after elastic limits. The plastic hinges occur when an element reaches the elastic limit, and it can continue to resist against exposed load with another formation (plastic form). For example, if a beam (having sufficient strength against the shear) that influenced larger cases of the moment, the cross-section continues to lengthen up to the moment carrying capacity is reached. If the moment continues to be increased from this boundary condition, the cross-section continues to rotate (shape changes continue), provided that the moment carried by the section is still constant. In this case, the material remains elastic close to the neutral axis, plasticization's occur on the outer parts which are away from the neutral axis (the most elongation and shortening occurred in outer parts).

In other words, plasticization means that the deformations in the outer fibers occur as elasto-plastic. It is useful to remind that, in the elastic behavior the material returned to its original state when force is removed, in elasto-plastic behavior the material does not return to its original state, permanent shape changes occur when force is removed. It can be said that the plasticization is formed intensively on the beam support region and at the lower ends of the columns. One thing to keep in mind is that if a plastic joint is formed, the damage is also caused in that part.

2.5. Method of Internal Steel Bracing

The internal steel bracing method is provided for the unfilled position surrounding the beams and columns of the RC model structure. The aim of steel bracing trusses is to enhance structural strength and stiffness examined by several previous researchers. Following the investigation, they obtain more strategies for improving seismic resistance of the existing building. The testing machine's geometric capacity, which is 58 x 58 cm, placed a size restriction on the model frames that were chosen. The models were designed with a square cross-section with a side length of 10cm [19].

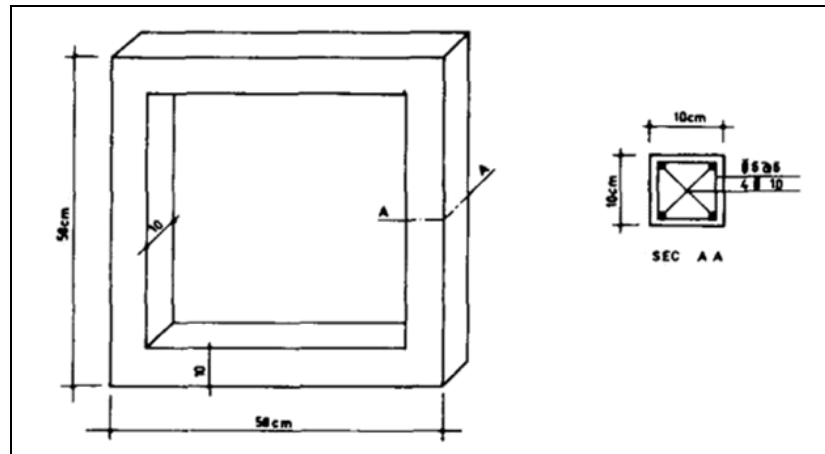


Figure 2.8: RC test model.

Researchers recommend using steel bracing in internal membership with bracing trusses. Figure 2.8 depicts the examination software used to assess the degree of benefit of the steel bracing diagonal in preparation for enhancing the lateral load's

influence on the existing RC building model and detecting the comparative performance of the steel bracing expansion and compressions. This study has chosen the most frequent technique of bracing, which becomes the X bracing system. The chosen frame is regarded as an RC building without steel braces and with steel trusses diagonal for tension. RC structure with steel bracing trusses for compression to decrease failure structure in both tension and compression for the X bracing system. The two trusses are joined together by an iron bolt at the place where they intersect. After attaching the model's components, the welding of the steel-bracing of the edges where column and beam's related each other to the same location both sides precast in the model's joints begins. Following connecting the model's components, it is the welding of the steel-bracing of the edges where column and beam related each other to the identical place both sides precast in the model's joints [19].

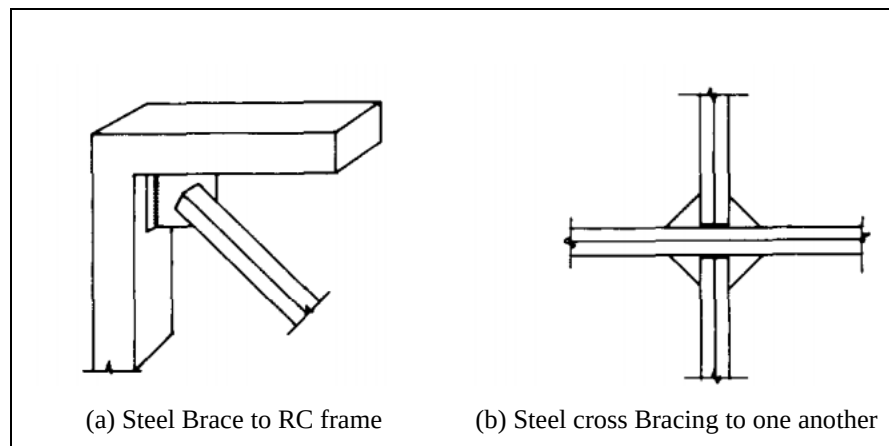


Figure 2.9: Detail of Connections.

The researcher investigated shear capacity and building strength employing cyclic applied to the model. According to the analysis, the greatest load that the modal can sustain without bracing is 4 ton. The model with the diagonal tension braced has carried 9 ton. The diagonal compression braced model was 10 ton, whereas the x braced model was 12.5 ton [20].

The essential findings during this X bracing investigation, the amount of loading in each cable or truss supported is not identical. The fundamental performance of expansion bracing was observed when compared to compression bracing. Consequently, the tension brace carries a greater load than compression bracing. The model increased the shear capacity through one bracing 2.5 times. The rigidity of the

steel bracing (X bracing) was assessed four times higher than the model without bracing [20].

These researchers also investigated the applications of internal bracing, with the intention of studying the seismic behavior of the RC structure model. This researcher has tested and evaluated the methods of the concentric bracing system when used internally for new building construction. There is sample of RC construction-sized 2:5 frames illustrated in the Figure 3.0 [20].

Following the test, they found that the bracing model can withstand a higher lateral load than the moment-resisted model. Therefore, the braced model has a higher load carrying capacity than the moment resist model. Before the bracing member buckled, it had a higher lateral rigidity than the moment-resisting frame. Eventually compression of the lateral-load model's bracing buckling released [20].

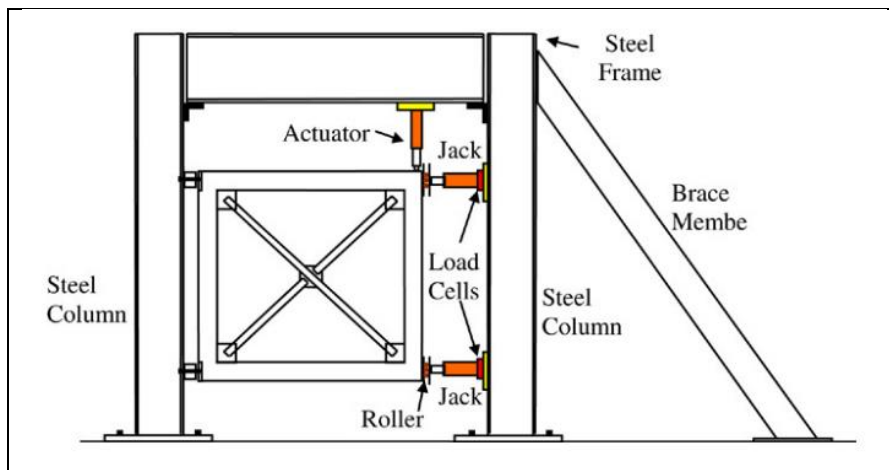


Figure 2.10: Test Setup Drawing.

According to a new study on the drift story, the braced model had less drift than the moment resisted model, and the objectives of this were the earlier rigidity of the bracing model. The evaluation of the story drift, the higher energy reduced through the braced model, was more than the moment resists model [20].

2.6. Rectangular Hollow Sections' Seismic Behavior (RHS)

The RHS, either hot-rolled or fabricated by welding four plates, are utilized in bridge piers and buildings. The RHS are formed by welding four plates or hot rolling,

and they are utilized in constructions with a small width to thickness ratio of element plates [21]. RHS's ultimate strengths are excellent, as well as its post-local buckling performance.

Some tests are done for instance RHS and I-section considering with similar the hysteretic loop [22], [23].

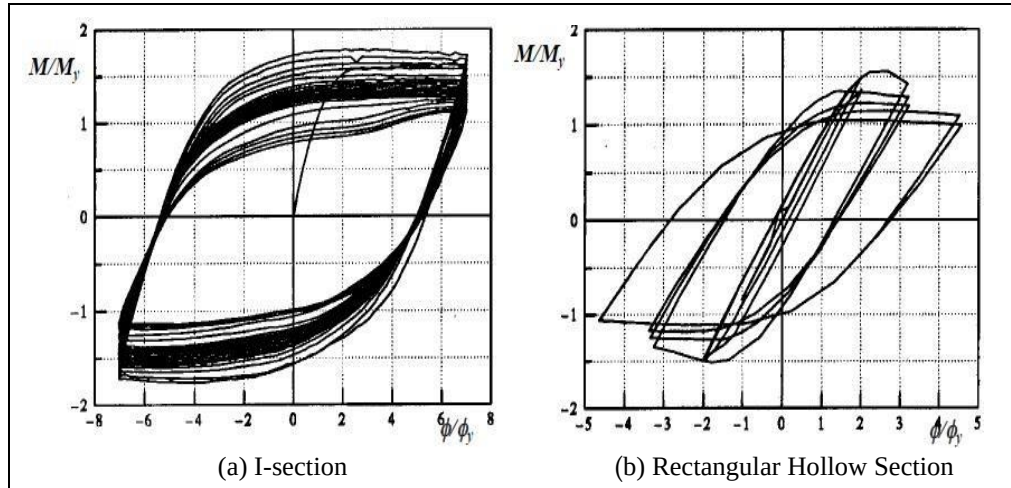


Figure 2.11: Hysteretic Loop Behavior.

Failure phases in I sections of steel structures begin with cracks in the web-flange connection in welded sections and welded components in built-up sections and end failing with local buckling in the flanges. I sections, on the other hand, are excellent sections with great ductility and energy dissipation capacities [23]. The degradation owing to local buckling and the gradual stabilization is the response ranges that can be noticed. Whenever I sections get more compact, the middle range will be decreased, and the tendency of cracking will rise. So, highly compact sections and rigid connections will not provide the required rotations. The damage in I sections may be modeled using a specialized method known as low cycle fatigue [22].

The RHS degradation strength with cycling is significant in the case of high width to thickness ratios under rising amplitudes. Thus, this necessitates damage accumulation estimations that take into account deformation and low cycle fatigue damage [22]. For that reason, this study emphasizes the significance of the RHS Figure 5.18.

3. DESIGN STAGES OF THE REINFORCED CONCRETE BUILDING

The sample building to be designed and analyzed is located in Gebze. The numerical model is a three-dimensional reinforced concrete dormitory building with a frame structural system. The structure is symmetrical in both X and Y axis direction.

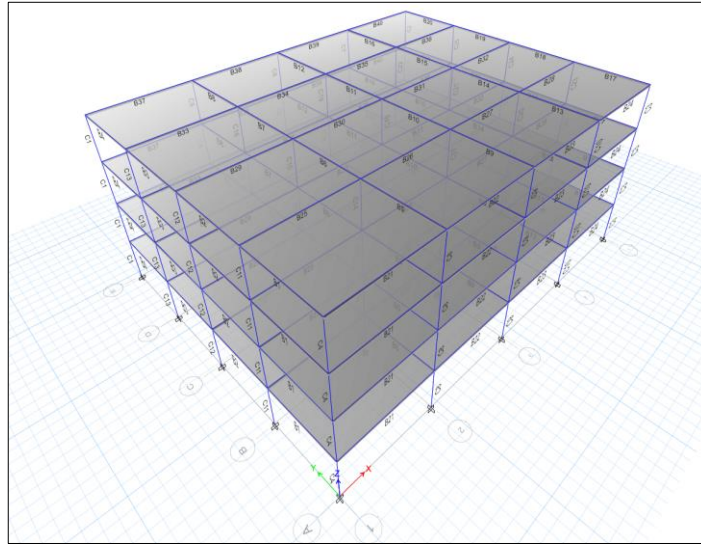


Figure 3.1: 3D View of the Building.

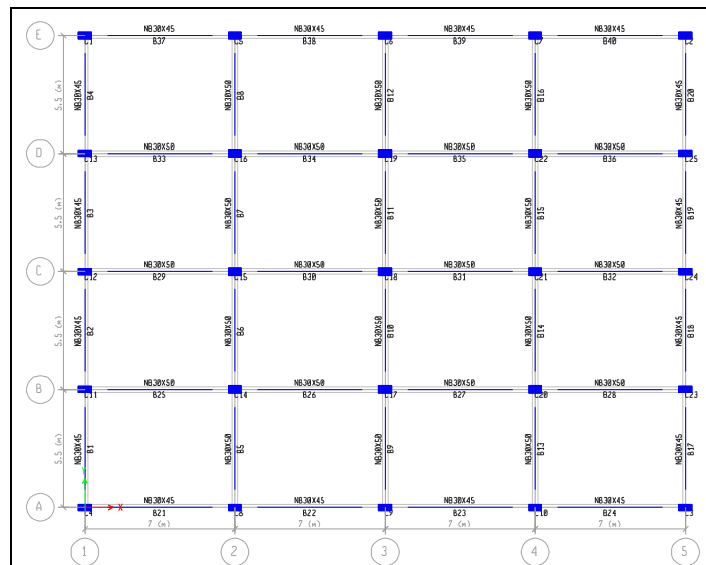


Figure 3.2: Normal Floor Plan of the Building.

3.1. Material Information

The concrete is assumed to perform elastic linear. The concrete grade used for structural members in the study is C20 with a cylindrical pressure resistance of 20MPa was chosen for 28 days, and the steel grade is S420 with minimum characteristic yield strength of 420MPa.

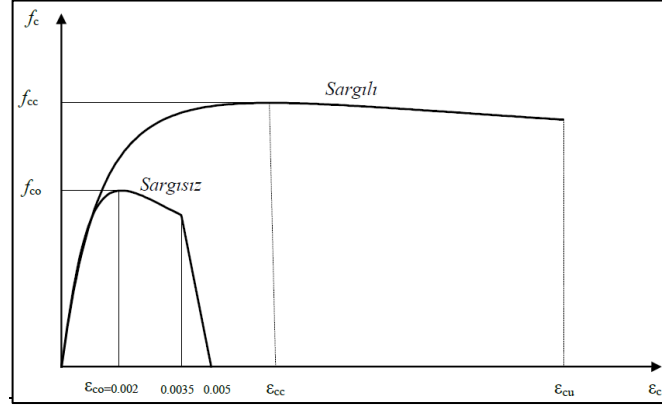


Figure 3.3: Confined Unconfined concrete models.

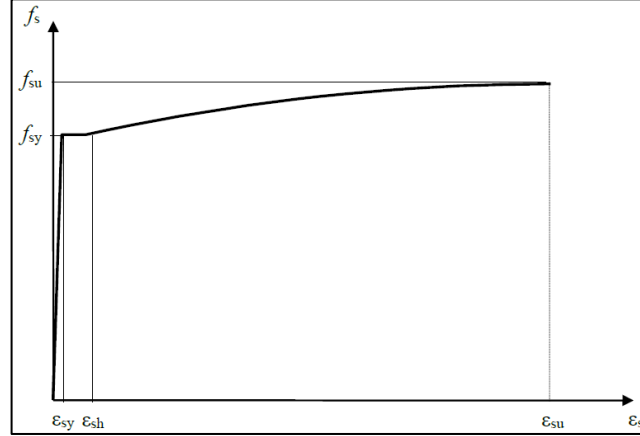


Figure 3.4: Steel model.

3.2. Load Information

The fixed load values used in the calculations are as follows. Self-weight of reinforced concrete element: $\gamma = 25.0 \text{ kN} / \text{m}^3$. Cover load: $g = 2.50 \text{ kN} / \text{m}^2$. Interior walls: $g = 4.5 \text{ kN} / \text{m}^2$. Exterior walls: $g = 5.5 \text{ kN} / \text{m}^2$. Roofing load: $g = 2.0 \text{ kN} / \text{m}^2$.

According to TS498, live loads are defined to the structure as follows. Dormitory:
 $q = 3.0 \text{ kN / m}^2$. Roof Floor: $q = 1.5 \text{ kN / m}^2$.

3.3. Earthquake Ground Motion Spectrum

The designed dormitory building is located in Gebze. In order to obtain the horizontal elastic spectra at the DD-2 ground motion level, the map spectral acceleration coefficient S_s and the map spectral acceleration coefficient S_1 values for the 1.0 second period were examined on the AFAD site.

Rapor Başlığı:	Location		
Deprem Yer Hareketi Düzeyi	DD-2	50 yılda aşılma olasılığı %10 (tekrarlanma periyodu 475 yıl) olan deprem yer hareketi düzeyi	
Yerel Zemin Sınıfı	ZC	Çok sıkı kum, çakıl ve sert kil tabakaları veya ayrılmış, çok çatlaklı zayıf kayalar	
Enlem:	40.789965°		
Boylam	29.492587°		
Çıktılar			
$S_S = 1.361$	$S_1 = 0.373$	$PGA=0.562$	$PGV=42.657$

Figure 3.5: Site Selection.

Map spectral acceleration coefficients based on the DD-2 earthquake ground motion level, the short period spectral acceleration coefficient S_s and 1.0 spectral acceleration coefficient S_1 are taken from the earthquake hazard map and the values of (TBDY 2.3.1) are given below.

$$S_s = 1.361$$

$$S_1 = 0.373$$

TBDY 2018 Local for the period shorter than Table 2.1 and for the period of 1.0 second from Table 2.2 Soil effect coefficients were taken and given below.

$$F_s = 1.200$$

$$F_1 = 1.200$$

Map spectral acceleration coefficients S_s and S_1 were converted into design spectral acceleration coefficients S_{ds} and S_{d1} according to TABLE 2.1. The horizontal elastic design spectrum was created according to article 2.3.4 of TBDY (2018).

$$\begin{aligned} S_{ds} &= S_s F_s \\ S_{d1} &= S_1 F_1 \end{aligned} \quad (3.1)$$

Horizontal design spectrum corner periods are calculated as follows;

$$\begin{aligned} TA &= 0.2 \frac{S_{d1}}{S_{ds}} \\ TB &= \frac{S_{d1}}{S_{ds}} \end{aligned} \quad (3.2)$$

Horizontal elastic design spectral acceleration values ($S_{ae}(T)$) according to the natural vibration period of the structure exchange is expressed below.

Table 3.1: Ground impact coefficients, (F_s).

Soil Class	Ground Impact Coefficient for short period zone, (F_s).					
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	Site-specific ground behavior analysis will be carried out					

Table 3.2: Ground impact coefficients, (F_1).

Soil Class	Ground Impact Coefficient for the 1.0 second period, (F_1).					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Site-specific ground behavior analysis will be carried out					

3.4. Pre-dimensioning of Building Structural System

Dimensioning of slabs; Article 7.11.2 of TBDY (2018) states that the requirements given in TS 500 for floor thicknesses apply. In the design principles section 11.4.2 of TS 500, the minimum value for the slab thickness working in two directions is given as follows. Laying thickness 12 cm was selected.

$$h \geq \frac{l_{sn}}{15 + \frac{20}{m}} \left[1 - \frac{\alpha_s}{4} \right] \quad (3.3)$$
$$h \geq 80mm$$

Beam Sizing; In accordance with Article 7.4.1 of TBDY (2018), the frame width of the moment-transmitting beams shall be at least 25cm. Beam height should not be less than 3 times the floor thickness. In the current project, 30cm / 50cm, 30cm /45cm beams were used.

Column Sizing; Since the Earthquake Loads were not in the pre-dimensioning phase, the following verification was performed. The maximum normal force condition that the columns can bear under increased loads according to the description in 7.4.1 of TS 500 is given below.

$$N_d \leq 0.9f_{cd}A_c \quad (3.4)$$

According to TBDY (2018) clause 7.3.1.1, the minimum cross-sectional size of rectangular columns shall not be less than 300mm. According to TBDY (2018) clause 7.3.1.2, the axial compressive force in the columns shall be the following condition considering the G, Q vertical loads and E earthquake load. must provide.

$$N_{dm} \leq 0.9f_{ck}A_c \quad (3.5)$$

35cm / 60cm, 60cm / 30cm columns were used in the project. Under vertical loads, including the earthquake effect, the axial force level remains between 21% and 3%.

3.5. General Principles for Design Under Earthquake Effect

The completed building will be used as a residence. TBDY (2018) Figure 3.6 shows the Building Use Class and Building Importance Coefficient. (Building Use Class, BKS = 3 and Building Importance Coefficient, I = 1).

Bina Kullanım Sınıfı	Binanın Kullanım Amacı	Bina Önem Katsayısı (I)
BKS = 1	Deprem sonrası kullanımı gereken binalar, insanların uzun süreli ve yoğun olarak bulunduğu binalar, değerli eşyanın saklandığı binalar ve tehlikeli madde içeren binalar a) Deprem sonrasında hemen kullanılması gerekli binalar (Hastaneler, dispanserler, sağlık ocakları, itfaiye bina ve tesisleri, PTT ve diğer haberleşme tesisleri, ulaşım istasyonları ve terminaleri, enerji üretim ve dağıtım tesisleri, vilayet, kaymakamlık ve belediye yönetim binaları, ilk yardım ve afet planlama istasyonları) b) Okullar, diğer eğitim bina ve tesisleri, yurt ve yatakhaneler, askeri kışlalar, cezaevleri, vb. c) Müzeler d) Toksik, patlayıcı, parlayıcı, vb. özellikleri olan maddelerin bulunduğu veya depolandığı binalar	1.5
BKS = 2	İnsanların kısa süreli ve yoğun olarak bulunduğu binalar Alışveriş merkezleri, spor tesisleri, sinema, tiyatro, konser salonları, ibadethaneler, vb.	1.2
BKS = 3	Diğer binalar BKS=1 ve BKS=2 için verilen tanımlara girmeyen diğer binalar (Konutlar, işyerleri, oteller, bina türü endüstri yapıları, vb.)	1.0

Figure 3.6: Building Utilization Classes and Building Coefficients.

Short Period Design Spectral Acceleration Coefficient of Building DD-2 Earthquake Ground Motion Level $S_{ds} = 1.633$. According to the data of building usage class BKS = 3 TBDY Figure 3.7 is used, earthquake design class DTS = 1.

DD-2 Deprem Yer Hareketi Düzeyinde Kısa Periyot Tasarım Spektral İvme Katsayısı (S_{Ds})	Bina Kullanım Sınıfı	
	BKS = 1	BKS = 2, 3
$S_{Ds} < 0.33$	DTS = 4a	DTS = 4
$0.33 \leq S_{Ds} < 0.50$	DTS = 3a	DTS = 3
$0.50 \leq S_{Ds} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{Ds}$	DTS = 1a	DTS = 1

Figure 3.7: Earthquake Design Classes.

Building Height and Building Height Classes, TBDY (2018) According to Figure 3.8, when $DTS = 1$ and building height $H = 3.2M$; Building Height Class is $BYS = 6$.

Bina Yükseklik Sınıfı	Bina Yükseklik Sınıfları ve Deprem Tasarım Sınıflarına Göre Tanımlanan Bina Yükseklik Aralıkları [m]		
	$DTS = 1, 1a, 2, 2a$	$DTS = 3, 3a$	$DTS = 4, 4a$
$BYS = 1$	$H_N > 70$	$H_N > 91$	$H_N > 105$
$BYS = 2$	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
$BYS = 3$	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
$BYS = 4$	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
$BYS = 5$	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
$BYS = 6$	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
$BYS = 7$	$7 < H_N \leq 10.5$	$10.5 < H_N \leq 17.5$	
$BYS = 8$	$H_N \leq 7$	$H_N \leq 10.5$	

Figure 3.8: Building height ranges are defined according to building height classes and earthquake design classes.

Building Performance Target; Building Height Classes Defined by Building Height Classes and Earthquake Design Classes (Figure 3.8). TBDY (2018) In-situ cast reinforced concrete for DD-2 Earthquake Ground Movement Level according to Figure 3.7 buildings must meet the Performance Controlled Damage Performance Target and Approach should be designed with Performance Targets and Assessment / Design Approaches for New Buildings or Existing Buildings According to Earthquake Design Classes (Figure 3.9) New On-Site Bulk Reinforced Concrete, Precast Concrete and Steel Buildings (Except Tall Buildings - $BMS \geq 2$).

Deprem Yer H. Düzeyi	$DTS = 1, 1a^{(1)}, 2, 2a^{(1)}, 3, 3a, 4, 4a$		$DTS = 1a^{(2)}, 2a^{(2)}$	
	Normal Performans Hedefi	Değerlendirme/Tasarım Yaklaşımı	İleri Performans Hedefi	Değerlendirme/Tasarım Yaklaşımı
DD-3	—	—	SH	ŞGDT
DD-2	KH	DGT ⁽⁵⁾	KH	DGT ^(3,4)
DD-1	—	—	KH	ŞGDT

Figure 3.9: New On-Site Bulk Reinforced Concrete, Precast Concrete and Steel Buildings (Except Tall Buildings - $BMS \geq 2$).

Accounting and Design of the Building Under the Effect of Earthquake; Determination of R and D Coefficients According to Structural System Type TBDY

(2018) According to Figure 3.10, the model is a cast-in-situ concrete building and belongs to the class of buildings where all earthquake effects are met with high-ductility reinforced concrete frames. Therefore, the Structural System Behavior Coefficient $R = 8$, the strength coefficient $D = 3$.

Bina Taşıyıcı Sistemi	Taşıyıcı Sistem Davranış Katsayısı R	Dayanım Fazlalığı Katsayısı D	İzin Verilen Bina Yükseklik Sınıfları BYS
A. YERİNDE DÖKME BETONARME BİNA TAŞIYICI SİSTEMLERİ			
A1. Süneklilik Düzeyi Yüksek Taşıyıcı Sistemler			
A11. Deprem etkilerinin tamamının moment aktaran <i>süneklilik düzeyi yüksek</i> betonarme çerçevelerle karşılandığı binalar	8	3	$BYS \geq 3$
A12. Deprem etkilerinin tamamının <i>süneklilik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdelerle karşılandığı binalar	7	2.5	$BYS \geq 2$
A13. Deprem etkilerinin tamamının <i>süneklilik düzeyi yüksek</i> boşluksuz betonarme perdelerle karşılandığı binalar	6	2.5	$BYS \geq 2$
A14. Deprem etkilerinin moment aktaran <i>süneklilik düzeyi yüksek</i> betonarme çerçeveler ile <i>süneklilik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	8	2.5	$BYS \geq 2$
A15. Deprem etkilerinin moment aktaran <i>süneklilik düzeyi yüksek</i> betonarme çerçeveler ile <i>süneklilik düzeyi yüksek</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	7	2.5	$BYS \geq 2$
A16. Deprem etkilerinin tamamının çatı düzeyindeki bağlantıları mafsallı olan ve yüksekliği 12 m'yi geçmeyen <i>süneklilik düzeyi yüksek</i> betonarme kolonlar tarafından karşılandığı tek katlı binalar	3	2	–
A2. Süneklilik Düzeyi Karma Taşıyıcı Sistemler (Bkz. 4.3.4.1, 4.3.4.6)			
A21. Deprem etkilerinin moment aktaran <i>süneklilik düzeyi sınırlı</i> betonarme çerçeveler ile <i>süneklilik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	6	2.5	$BYS \geq 4$
A22. Deprem etkilerinin moment aktaran <i>süneklilik düzeyi sınırlı</i> betonarme çerçeveler ile <i>süneklilik düzeyi yüksek</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	5	2.5	$BYS \geq 4$

Figure 3.10: Carrier System Behavior Coefficient, Strength Excess Coefficient and Allowed Building Height Classes for Building Carrier Systems.

Earthquake Load Reduction Coefficient and Reduced Design Acceleration Spectrum; TBDY (2018) 4.2.1.2 Earthquake Load Reduction Coefficient $R_a(T)$ is defined as follows.

$$\begin{aligned}
 R_a(T) &= \frac{R}{I} \\
 R_a(T) &= D + \left(\frac{R}{I} - D \right) \frac{T}{T_B} \\
 S_{aR}(T) &= \frac{S_{ae}(T)}{R_a(T)}
 \end{aligned}
 \quad
 \begin{aligned}
 &T > T_B \\
 &T \leq T_B
 \end{aligned}
 \quad (3.6)$$

Building Important Factor, $I=1$; Coefficient of excess strength, $R=8$; Coefficient of excess strength, $D=3$.

3.6. Modeling of Structural System and Calculations

The mathematical model of the building will be constructed in three dimensions and the earthquake effect in two horizontal directions perpendicular to each other will be considered (TBDY4.5.1.1 and 4.51.2). Columns will be modeled as rod-finite elements (TBDY4.5.2.1). Rigid diaphragm will be assigned to the slabs. (TBDY4.5.6.3). Live load participation coefficient $n = 0.3$ is taken.

Binanın Kullanım Amacı	n
Depo, antrepo, vb.	0.80
Okul, öğrenci yurdu, spor tesisi, sinema, tiyatro, konser salonu, ibadethane, lokanta, mağaza, vb.	0.60
Konut, işyeri, otel, hastane, otopark, vb.	0.30

Figure 3.11: Live Load Mass Participation Coefficient.

The building floor masses were calculated by adding the coefficient of moving load to the live load and adding it with self-loading. In the table, masses of 4 floors are given.

Table 3.3: Building Floor Masses.

Floor	Floor Mass [kg]	Floor Weight [W_i (kN)]	Floor Mass [mi (ton)]
Floor4	567845.01	5678.4501	578.8430275
Floor3	626364.99	6263.6499	638.496422
Floor2	626364.99	6263.6499	638.496422
Floor1	626365.03	6263.6503	638.4964628
TOTAL		24469.4002	2494.332334

Effective cross-sectional stiffness of building carrier elements is taken from Table 4.2 of TBDY (2018). Table 3.5. Effective Section Stiffness Factors of Reinforced Concrete Bearing System Components.

Table 3.4: Effective section stiffness multipliers.

Betonarme Taşıyıcı Sistem Elemanı	Etkin Kesit Rijitliği Çarpanı	
<i>Perde – Döşeme (Düzlem İçi)</i>	<i>Eksenel</i>	<i>Kayma</i>
Perde	0.5	0.5
Bodrum perdesi	0.8	0.5
Döşeme	0.3	0.3
<i>Perde – Döşeme (Düzlem Dışı)</i>	<i>Eğilme</i>	<i>Kesme</i>
Perde	0.3	1.0
Bodrum perdesi	0.5	1.0
Döşeme	0.3	1.0
<i>Çubuk eleman</i>	<i>Eğilme</i>	<i>Kesme</i>
Bağ kirişi	0.2	1.0
Çerçeve kirişi	0.4	1.0
Çerçeve kolonu	0.7	1.0
Perde (eşdeğer çubuk)	0.5	0.5

3.7. Selecting the Analysis Method

TBDY (2018) Figure 3.12 describes the buildings where the equivalent Earthquake Load method can be used. Since the building's building height class is $BYS = 6$, the earthquake design class is $DTS = 1$, the calculation with the equivalent earthquake load is suitable.

Bina Türü	İzin Verilen Bina Yükseklik Sınıfı	
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a, 4, 4a
Her bir katta burulma düzensizliği katsayısının $\eta_b \leq 2.0$ koşulunu sağladığı ve ayrıca B2 türü düzensizliğinin olmadığı binalar	$BYS \geq 4$	$BYS \geq 5$
Diğer tüm binalar	$BYS \geq 5$	$BYS \geq 6$

Figure 3.12: Buildings where Equivalent Earthquake Load Method can be Applied.

Earthquake Calculation with Equal Earthquake Load Method; The base shear force formula TBDY (2018) is specified.

$$V_w = m_t S_{aR}(T_p) \geq 0.04 m I S_{ds} g \quad (3.7)$$

TBDY (2018) The natural vibration period $T_p(x)$ of the building, calculated according to 4.7.3.2, shall not be more than 1.4 times the T_{PA} period given in Article 4.7.3.4 of TBDY. $C_t = 0.1$ should be taken only in the model with reinforced concrete building system.

$$T_{PA} = C_t H_n^{3/4} \quad (3.8)$$

Design and Calculation for $R = 8$; After determining the building vibration periods, the reduced design spectral accelerations of the structure were calculated during these periods.

DETERMINATION OF FORCES BY THE EQUIVALENT EARTHQUAKE LOAD METHOD							
Floor	Floor Mass [kg]	Floor Weight [Wi (kN)]	Floor Mass [mi (ton)]	PARAMETRELER			
Floor4	567845.01	5678.4501	578.8430275				
Floor3	626364.99	6263.6499	638.496422	T	X-AXIS	Y-AXIS	
Floor2	626364.99	6263.6499	638.496422	SD1	0.559	0.559	
Floor1	626365.03	6263.6503	638.4964628	Sae	0.43740	0.52936	
TOPLAM		24469.4002	2494.332334	Ra	5.33333	5.33333	
				SaR	0.08201	0.09925	
				Vt	2006.807	2428.692	
X Yönü - Son kat Fiktif Yük							
Vtx	621.1718496	Delta FN	18.63515549				
Y Yönü - Son kat Fiktif Yük							
Vty	833.3353544	Delta FN	25.00006063				
Floor	hi(m)	Hi(m)	Floor Mass [mi (ton)]	mi*Hi	mi*Hi/ $\sum$ mi*Hi	X Fte(kN)	Y Fte(kN)
Floor4	3.3	13.2	578.843	7640.728	0.376706802	245.6148266	329.5054641
Floor3	3.3	9.9	638.496	6321.1146	0.311646596	187.7785095	251.9149425
Floor2	3.3	6.6	638.496	4214.0764	0.207764397	125.185673	167.943295
Floor1	3.3	3.3	638.496	2107.0383	0.103882205	62.59284049	83.97165285
			TOPLAM	20282.957	1	621.1718496	833.3353544

Figure 3.13: Determination of Forces by the Equivalent Earthquake Load Method.

According to Article 4.7.2 of TBDY (2018), equivalent earthquake loads were effected to floors in proportion to the mass and height of the building in X and Y direction. $\pm 5\%$ additional eccentricity was considered. The forces acting on the plies are indicated in the Figure 3.12 below.

KAT	Hi(m)	X	Y
		Fte(kN)	Fte(kN)
KAT4	13.2	245.6148266	329.5054641
KAT3	9.9	187.7785095	251.9149425
KAT2	6.6	125.185673	167.943295
KAT1	3.3	62.59284049	83.97165285

Figure 3.14: Forces Affecting the Floors According to Equivalent Earthquake Load Method.

3.8. Regular Building Under Earthquake Effect

Definition of Irregular Buildings due to the negativity in the behavior of earthquake design and construction regarding the identification of irregular buildings that should be avoided with considering related conditions of article 3.6.2.1 in TBDY (2018).

3.8.1. A1 - Torsional irregularity

Torsional irregularity A1 in TBDY (2018) 3.6.1 is explained in Figure 3.15.

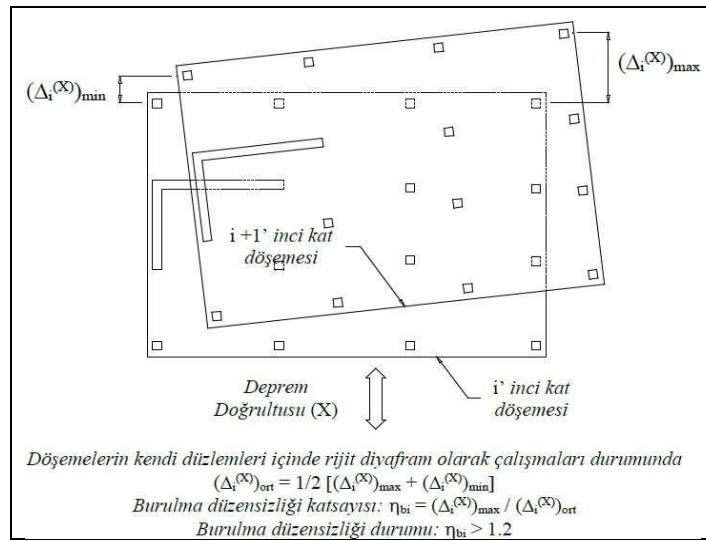


Figure 3.15: A1 - Torsional irregularity.

In the case of buildings, the torsional irregularity value does not exceed the limit of 1.2. It is indicated that the type of A1 irregularity does not exist in the building. The torsion coefficients in the X and Y directions were calculated as 1.05 for the maximum floor. Since the values founded are less than the limit of $\eta_{bi} > 1.4$ there are no torsional in both X and Y directions.

3.8.2. B2 – Stiffness Irregularity Between Neighboring Floor (Soft Floors)

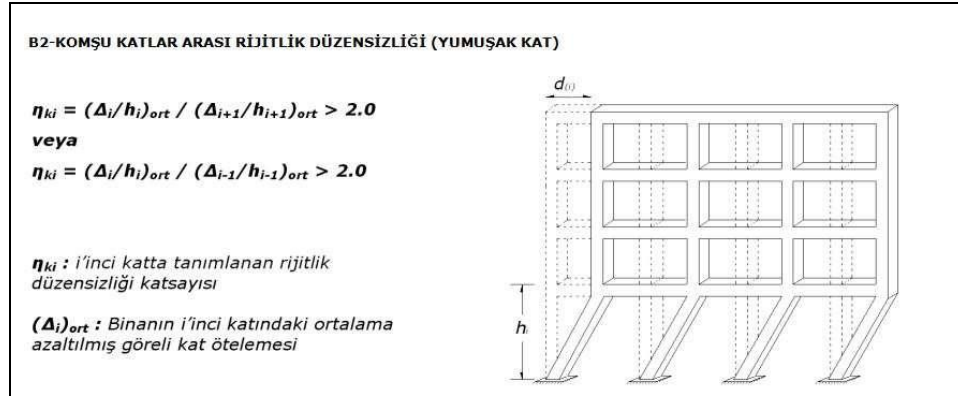


Figure 3.16: B2 – Stiffness Irregularity.

B2 - Stiffness Irregularity Factor is characterized as the ratio of the average relative story drift at any i^{th} floor to the intermediate relative story drift at the floor adjacently above or below, which is greater than 2.0. Since the ratio for direction X, and Y are not greater than 2, B2 type irregularity was not observed.

3.8.3. Evaluation of Relative Floor Transfer

Control of Relative Float Ratios, in addition to the capacity of the carrier elements under earthquake loads to be sufficient to meet the forces acting on the section; It is stated that the relative displacements of the floors should be limited in modern earthquake codes in order to ensure that the damage to the non-bearing elements is kept at a minimum level and to ensure that the second-order effects are limited.

In TBDY 2018 4.9.1, the determination of the effective relative story drift for any earthquake direction is shown mathematically.

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

$$\delta_i^{(x)} = \frac{R}{I} \Delta_i^{(x)} \quad (3.9)$$

For the earthquake direction X and Y, the reduced relative floor shifts at the column ends under the increased additional eccentricity were calculated and the

effective relative floor shifts were calculated. The maximum value of the effective relative floor offset must meet the requirement of TBDY (2018) 4.9.1.3.a and TBDY (2018) 4.9.1.3.b on each floor.

$$\lambda \frac{\delta_{imax}^{(x)}}{h_i} \leq 0.008K \quad (3.10)$$

$\lambda = 0.40$, $\kappa = 1$, this condition was checked on the basis of the corner end columns marked in the Figure 3.17 and it was found to be met.

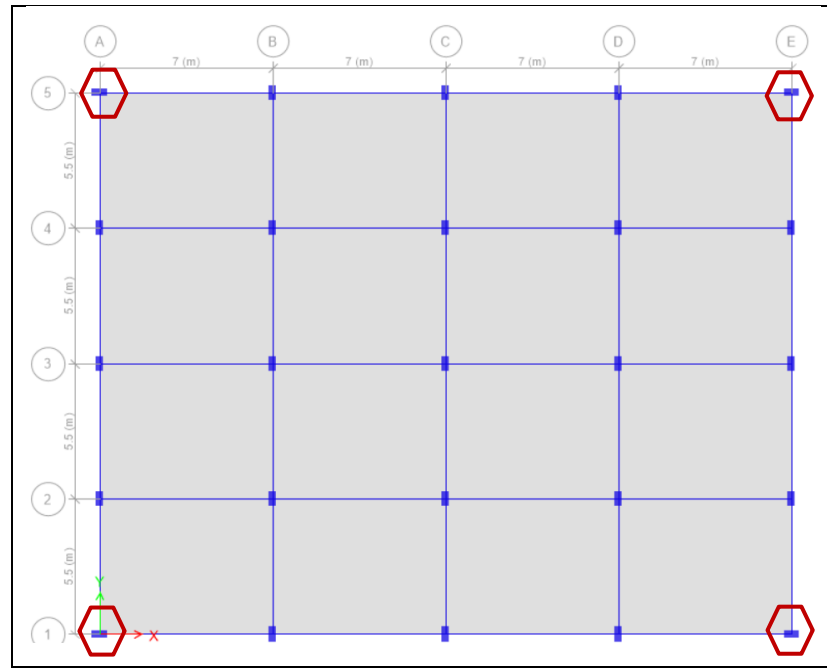


Figure 3.17: Columns for Controlling Relative Floor Shifts.

The coefficient defined in TBDY 4.9.1.4 was calculated as 0.360 for the X direction and 0.358 for the Y direction, and the κ coefficient was taken as 1 since it is a reinforced concrete building. the largest effective relative story drift ratio was calculated with the (Y) earthquake direction as approximately 0.40%. It has been shown that it satisfies the boundary condition given in the TBDY (4.34a) equation.

3.8.4. Second Order Control

TBDY (2018) was used to control the second order effects in the building by using following equation. Reduced relative offsets were calculated taking into account the additional e-eccentricity in the X and Y directions, which met second order control the required limits.

$$\theta_{Ili}^{(x)} = \frac{\left(\Delta_i^{(x)}\right)_{ort} \sum_{K=i}^N w_k}{V_i^{(x)} h_i} \quad (3.11)$$

$$\theta_{IImax}^{(x)} \leq 0.12 \frac{D}{C_h R}$$

3.9. Design of Reinforced Concrete Element

Beam Design; For the calculation of transverse and longitudinal reinforcement of beams with high ductility level conditions are found in article 7.4 of the TBDY (2018) 1. On the floor B / 3-4 B35X50cm dimension B15 beam is calculated. Envelope; The results of the combination. Combinations within the envelope combination are formed.

$$(1.4G + 1.6Q); (G + Q); (G + Q + 0.3EX + EY); (G + Q + EX + 0.3EY).$$

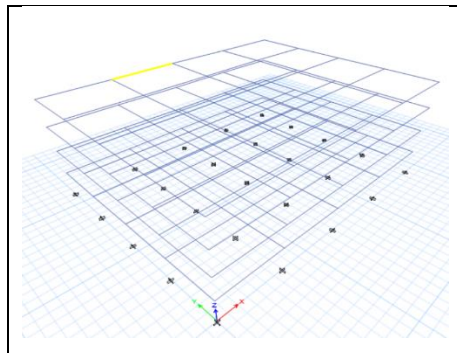


Figure 3.18: Analysis Model Beam.

Beam force Details; The moment values for beam B23 are values of design moment in the opening ($M_d = 128.74 \text{ kN.m}$), and Design moment at support: ($M_d = -187.28 \text{ kN.m}$) founded using the envelope combination to design.

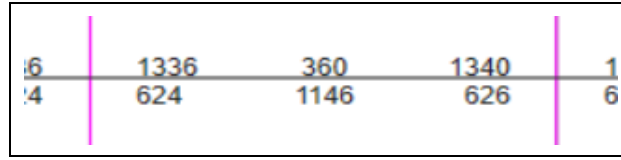


Figure 3.19: Longitudinal Bars in Beam B31.

Table 3.5: Longitudinal Bars in Beam B31 Arrangement.

B31	Left	Middle	Right
	4Φ22 4Φ16	4Φ12 4Φ20	4Φ22 4Φ16

Column Design; Ground floor C18 column 350mmX600mm is designed according to item 7.3 of TBDY (2018). Column design is based on TS500 principles.

$$\frac{N_{dm}}{A_c f_{ck}} = \leq 0.40 (TBDY 7.3.1.2) \quad (3.12)$$

The result of the vertical combinations was the result of the axial force in the column, the axial force level was checked according to TBDY (2018) item 7.3.1.2.

12 Φ22 longitudinal reinforcement is placed on the column as in the Figure 3.20, @10/100 mm transverse reinforcement is also added to the cross section below is the reinforcement details of the column.

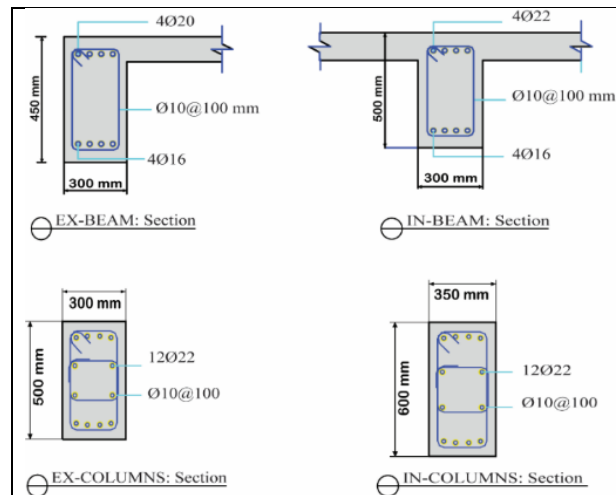


Figure 3.20: Reinforcement details.

4. PERFORMANCE ANALYSIS ACCORDING TO TBDY-2018 EARTHQUAKE CODE REGULATIONS

Performance-based earthquake engineering is the process of designing and evaluating structures to meet specific performance criteria. The notion behind this approach is based on anticipating the performance of structures with measurable dependability under usual and extraordinary loading. This performance is debated with the client in order to arrive at acceptable trade-offs based on life-cycle costs rather than building expenses alone [19].

PBEE is an admirable concept with a long way to go in terms of implementation. PBEE is restricted by legal and professional constraints, but whether it can fulfill on its promises or not remains an open question. It will be a losing cause if it is targeted at predicting structural performance in quantifiable formats [19].

4.1. Performance Evaluation of Existing Buildings According to TBDY-2018

4.1.1. Section Damage Conditions

Three damage states and damage limits are defined for ductile elements at the section level. These are Immediate Occupancy (SH), Life Safety (KH) and Pre-Collapse Prevention (GÖ) states and their limit values. Immediate occupancy defines a limited amount of inelastic behavior in the relevant section, life safety defines inelastic behavior in which the section strength can be achieved safely, and collapse prevention describes an advanced inelastic behavior in the section. This classification is not valid for brittle damaged elements.

4.1.2. Section Damage Areas

Elements whose critical sections do not reach SH are in the Limited Damage Zone. Elements between SH and KH are located in the Significant Damage Zone,

elements between KH and GÖ are located in the Advanced Damage Zone, and elements exceeding GÖ are located in the Collapse Zone.

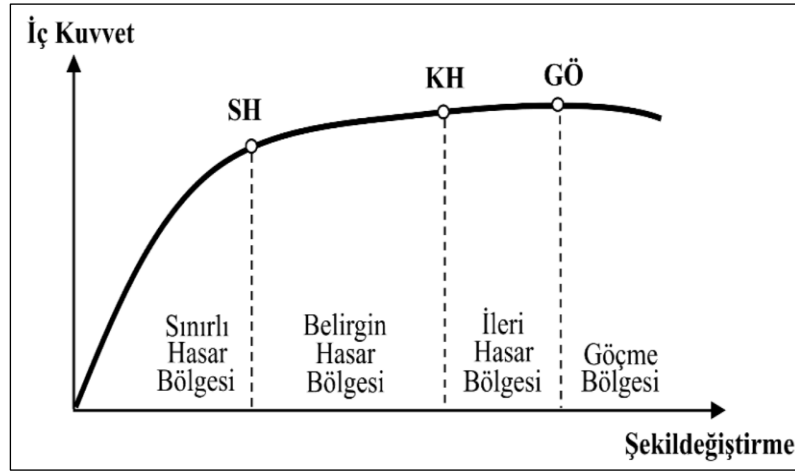


Figure 4.1: Sectional Damage Areas.

4.2. Horizontal Elastic Design Spectrum

The horizontal elastic design spectrum determined according to the soil conditions is used while defining the earthquake effect. When calculating this value, the building's period (T), Horizontal Design Corner Periods (T_A , T_B) and Design Spectral Acceleration Coefficients (S_{DS} , S_{D1}) taken from the Updated Earthquake Hazard Map is used.

$$\begin{aligned}
 S_{ae}(T) &= \left(0.4 + 0.6 \frac{T}{T_A}\right) \times S_{DS} & 0 \leq T \leq T_A \\
 S_{ae}(T) &= S_{DS} & T_A \leq T \leq T_B \\
 S_{ae}(T) &= \frac{S_{DS}}{T} & T_B \leq T \leq T_L \\
 S_{ae}(T) &= \frac{S_{D1} \times T_L}{T^2} & T_L \leq T
 \end{aligned} \tag{4.1}$$

Horizontal Design Spectrum Corner Periods (T_A , T_B) are determined by the values taken from the earthquake hazard map. These values form the plateau in the spectrum graph.

$$T_A = 0.2 \times \frac{S_{D1}}{S_{DS}} \quad T_B = \frac{S_{D1}}{S_{DS}} \quad (4.2)$$

It is obtained by multiplying the Design Spectral Acceleration Coefficients (S_{D1} , S_{DS}), Map Spectral Acceleration Coefficients (S_s , S_1), and Local Ground Effect Coefficients (F_s , F_1). Along with the help of the coordinate values entered in the Turkey Earthquake Hazard Map, which was updated with TBDY-2018 [2], the short-period map spectral acceleration coefficient (S_s) and the map spectral acceleration coefficient (S_1) for the 1.0 second period is obtained. Local soil effect coefficients (F_s , F_1) were given depending on the local soil class. (Table 3.1, 2.2)

Table 4.1: Knowledge Level Coefficient for Buildings.

Knowledge level	Knowledge coefficient
Limited	0.75
comprehensive	1.00

In order to find the pre-failure stiffness of the section close to reality, the effective section stiffness assignment has been introduced. In this regulation, only the axial load is taken into account for the calculation of the effective section stiffness. Along with TBDY-2018 [18], this situation has been found with a calculation that also takes into account the amount of axial load and reinforcement, as suggested in [11]. The effective section stiffnesses of the cracked section of the reinforced concrete elements under the effect of bending are used [18]. These values will be taken from the table below for linearly modeled elements. For elements that are not linearly modeled, these values are calculated with the help of the formulas given in (4.3) and (4.4).

$$(EI)_e = \frac{M_y}{\theta_y} \times \frac{L_s}{3} \quad (4.3)$$

$$\theta_y = \frac{\phi_y \times L_s}{3} + 0.0015 \times \eta \left(1 + 1.5 \frac{h}{L_s} \right) + \frac{\phi_y \times d_b \times f_{ye}}{8 \times \sqrt{f_{ce}}} \quad (4.4)$$

Stacked Plastic Behavior (Plastic Joint) Model is defined in order to obtain the plastic rotation and stress values formed in the columns and beams. This definition is for the ends of clear openings in columns and beams [18].

In the fixed single-mode push method, the earthquake load increments acting on the floors in the (k)th push step in the earthquake direction (X) are considered, and the floor modal effective masses are calculated according to the constant mode shape determined in the first step after the non-earthquake loadings and never changed during the push calculation expressed in terms [18].

$$\Delta f_{ix1}^{(X,k)} = m_{ix1}^{(X,k)} \times \Delta a_1^{(X,k)} ; \Delta f_{iy1}^{(X,k)} = m_{iy1}^{(X,k)} \times \Delta a_1^{(X,k)} \quad (4.5)$$

Modal acceleration increment ($\Delta a_1^{(X,k)}$), It is calculated from the yield condition of a new plastic hinge formed at the end of each pushing step. The obtained modal acceleration increment is added to the modal acceleration value found at the end of the previous step to obtain the cumulative modal acceleration ($a_1^{(X,k)}$) at the end of the (K)th step. Alternatively, it can be found by dividing the base shear force ($V_{tx1}^{(X,k)}$) in the earthquake direction by the base shear force modal effective mass ($m_{tx1}^{(X,1)}$) [18].

$$m_{ix1}^{(X,k)} = m_i \times \Phi_{ix1}^{(1)} \times \Gamma_1^{(X,1)} ; m_{iy1}^{(X,k)} = m_i \times \Phi_{iy1}^{(1)} \times \Gamma_1^{(X,1)} \quad (4.6)$$

Modal acceleration increment ($\Delta a_1^{(X,k)}$), It is calculated from the yield condition of a new plastic hinge formed at the end of each pushing step. The obtained modal acceleration increment is added to the modal acceleration value found at the end of the previous step to obtain the cumulative modal acceleration ($a_1^{(X,k)}$) at the end of the (K)th step. Alternatively, it can be found by dividing the base shear force ($V_{tx1}^{(X,k)}$) in the earthquake direction by the base shear force modal effective mass ($m_{tx1}^{(X,1)}$) [18].

$$a_1^{(X,k)} = \frac{V_{tx1}^{(X,k)}}{m_{tx1}^{(X,1)}} \quad (4.7)$$

Modal displacement ($d_1^{(X,k)}$), It can be calculated from the horizontal displacement obtained in the x-direction at any (i)th floor in the push calculation. Generally, the peak displacement at the (N)th floor can be taken in the push calculation [18].

$$d_1^{(X,k)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1} \times \Gamma_{x1}} \quad (4.8)$$

The push curve is drawn as a relationship between the base shear force and the peak displacement in the push calculation. Then, by making the above transformations of this curve, the modal capacity diagram is obtained as the modal acceleration and modal displacement relationship [18]. (Figure 4.2).

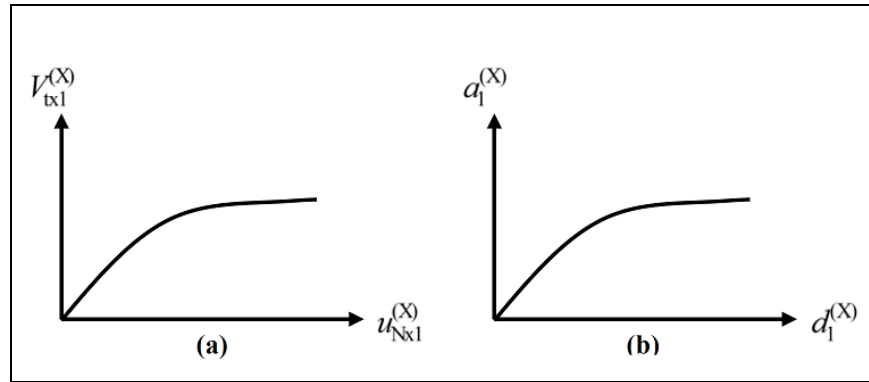


Figure 4.2: Coordinate Change.

The enormous displacement obtained from the modal capacitance diagram is defined as nonlinear spectral displacement [18].

$$d_{1,max}^{(X)} = S_{di}(T_1) \quad (4.9)$$

$$S_{di}(T_1) = C_R \times S_{de}(T_1) \quad (4.10)$$

$$C_R = \frac{\mu(R_y, T_1)}{R_y} \quad (4.11)$$

The spectral displacement ratio (CR) is found by dividing the earthquake's ductility demand ($\mu(R_y, T_1)$) by the Yield Strength Reduction Coefficient (R_y) [18].

- i) When the period (T_1) of the building is greater than the corner period (T_B), the ductility demand of the earthquake is taken equal to the yield strength reduction coefficient (R_y) according to the equal displacement rule. ($T_1 \geq T_B$)

$$\mu(R_y, T_1) = R_y \quad (4.12)$$

$$C_R = \frac{\mu(R_y, T_1)}{R_y} = 1 \quad (4.13)$$

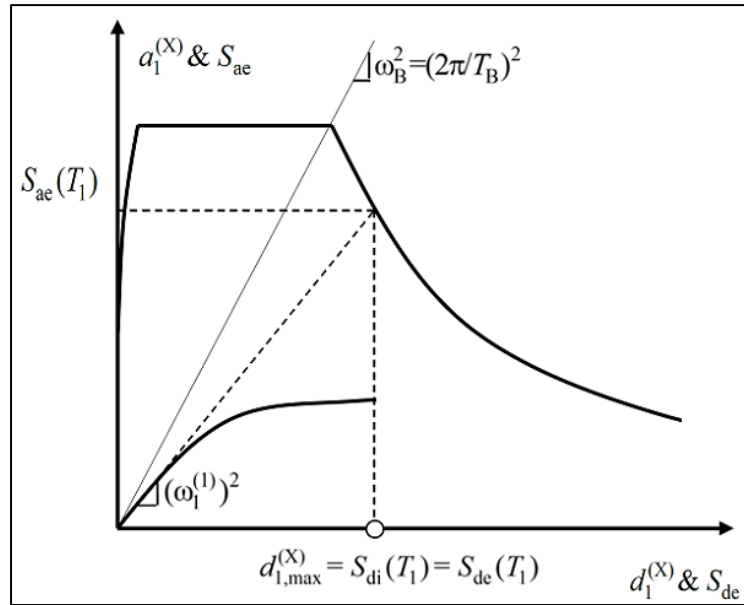


Figure 4.3: Case $T_1 \geq T_B$.

- ii) In case the period (T_1) of the building is smaller than the corner period (T_B), the ductility demand of the earthquake is calculated with the following equation. ($T_1 \leq T_B$).

$$A) \mu(R_y, T_1) = 1 + (R_y - 1) \times \frac{T_B}{T_1} \quad (4.14)$$

$$C_R = \frac{1 + (R_y - 1) \times \frac{T_B}{T_1}}{R_y} \geq 1 \quad (4.15)$$

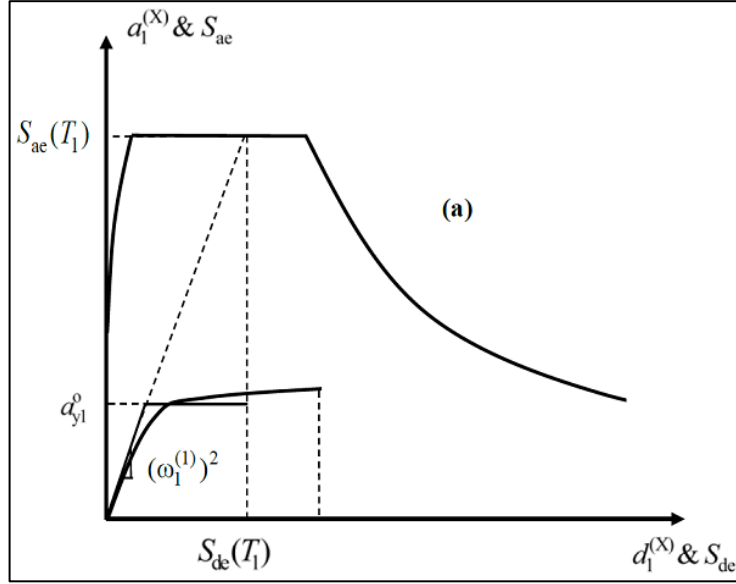


Figure 4.4: Case $T_1 \leq T_B$.

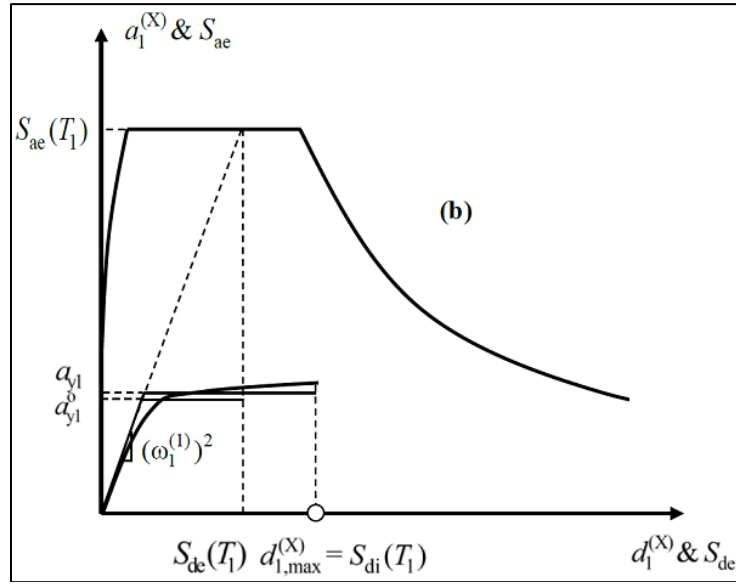


Figure 4.5: Case $T_1 \leq T_B$.

Plastic deformation and plastic rotation requests are received from the joints after the building is pushed as much as the displacement (u_{Nx1x}) demand found. These values are compared with the plastic rotation and deformation limit values [18].

4.2.1. Collapse Prevention (GÖ) performance Level

The allowable limit value for concrete unit shortening calculated based on the distributed plastic behavior model is expressed in TBDY 2018 5.8.1.1 with a relation for rectangular columns and beams. This relation is shown in Equation 4.18.

$$\text{For concrete} \rightarrow \varepsilon_c^{(GÖ)} = 0.0035 + 0.04 \times \sqrt{\omega_{we}} \leq 0.018 \quad (4.16)$$

While ω_{we} in Equation 4.18 represents the ratio of effective wrapping reinforcement to mechanical reinforcement, the value of 0.0035 indicates unit shortening of unwrapped concrete, also called shell concrete. The mechanical reinforcement ratio ω_{we} of the effective winding reinforcement is calculated as shown in equation 4.19.

$$\omega_{we} = \alpha_{se} \rho_{sh,min} \frac{f_{ywe}}{f_{ce}} \quad (4.17)$$

In Equation 4.19, f_{ce} corresponds to the average (expected) compressive strength of concrete, f_{ywe} corresponds to the average (expected) yield strength of transverse reinforcement, $\rho_{sh,min}$ corresponds to the smaller of the volumetric transverse reinforcement ratios in two horizontal directions for a rectangular section, and α_{se} corresponds to the efficiency coefficient of the dressing reinforcement. The calculation of the volumetric reinforcement ratio in two horizontal directions for a rectangular section is shown in equation 4.21.

$$\alpha_{se} = \left(1 - \frac{\sum a_i^2}{6 \times b_o \times h_o}\right) \left(1 - \frac{s}{2 \times b_o}\right) \left(1 - \frac{s}{2 \times h_o}\right) \quad (4.18)$$

$$\rho_{sh} = \frac{A_{sh}}{b_k \times s} \quad (4.19)$$

If the symbols in Equation 4.20 and 4.21 are examined; a_i is the distance between the axes of the longitudinal reinforcements wrapped by a cross tie or a stirrup, the

wrapped concrete dimensions measured from the b_o and h_o winding reinforcement axes, s the transverse reinforcement spacing, b_k the distance between the outermost transverse reinforcement axes, in other words the core size in the vertical direction, and A_{sh} is taken into account. represents the transverse reinforcement area in the received direction.

For reinforcing steel, the unit strain limit value at the performance level for the prevention of migration is defined in TBDY 2018 5.8.1.1, depending on the unit elongation corresponding to the tensile strength of the reinforcing steel. This relation is given in equation 4.22.

$$\text{for steel} \rightarrow \varepsilon_s^{(G\ddot{O})} = 0.4 \times \varepsilon_{su} \quad (4.20)$$

For plastic hinge rotations calculated based on the stacked plastic behavior model, the limit value allowed in the prevention of moving performance level is shown in equation 4.23.

$$\theta_p^{(G\ddot{O})} = \frac{2}{3} \times \left[(\phi_u - \phi_y) L_p \left(1 - 0.5 \frac{L_p}{L_s} \right) + 4.5 \phi_u d_b \right] \quad (4.21)$$

Here, ϕ_u represents the pre-failure curvature, ϕ_y the yield curvature, L_p the plastic hinge length, L_s the shear opening, and d_b the mean diameter of the reinforcing steels clamped at the joint.

4.2.2. Life Safety (KH) Performance Level

In TBDY 2018 5.8.1.3, the concrete and reinforcing steel total unit strain limit values and plastic rotation limit values allowed for the controlled damage performance level are defined depending on the permissible limit values for the prevention of migration. These definitions are expressed in the following Equations 3.24, 3.25, 3.26.

$$\begin{aligned} \text{For concrete} &\rightarrow \varepsilon_c^{(KH)} = 0.75 \times \varepsilon_c^{(G\ddot{O})} \\ \text{For Steel} &\rightarrow \varepsilon_s^{(KH)} = 0.75 \times \varepsilon_s^{(G\ddot{O})} \\ \text{Plastic Hinge Rotation} &\rightarrow \theta_p^{(KH)} = 0.75 \times \theta_p^{(G\ddot{O})} \end{aligned} \quad (4.22)$$

4.2.3. Immediate Occupancy (SH) Performance Level

In TB DY 2018 5.8.1.4, the concrete and reinforcing steel total strain limit values and plastic rotation limit values allowed for limited damage performance level are defined as shown in the following Equations 3.25.

$$\begin{aligned} \text{For concrete} &\rightarrow \epsilon_c^{(SH)} = 0.0025 \\ \text{For Steel} &\rightarrow \epsilon_s^{(SH)} = 0.0075 \\ \text{Plastic Hinge Rotation} &\rightarrow \theta_p^{(SH)} = 0 \end{aligned} \quad (4.23)$$

4.3. Flow Chart of Analysis

The flow chart of the analysis performed (Figure 4.6, Figure 4.7, Figure 4.8, Figure 4.9) is illustrated on the following pages.

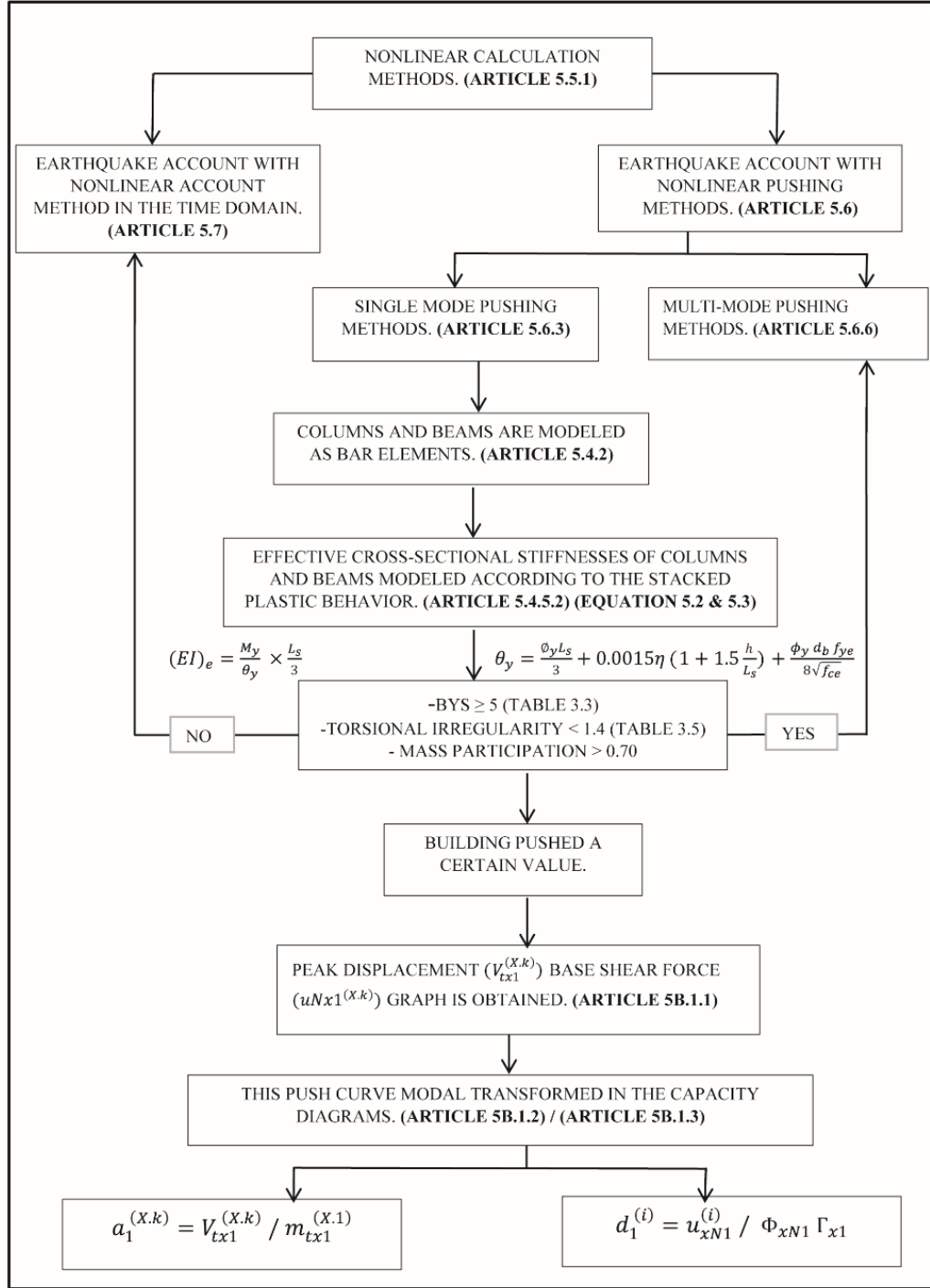


Figure 4.6: Flow Chart of Analysis 1.

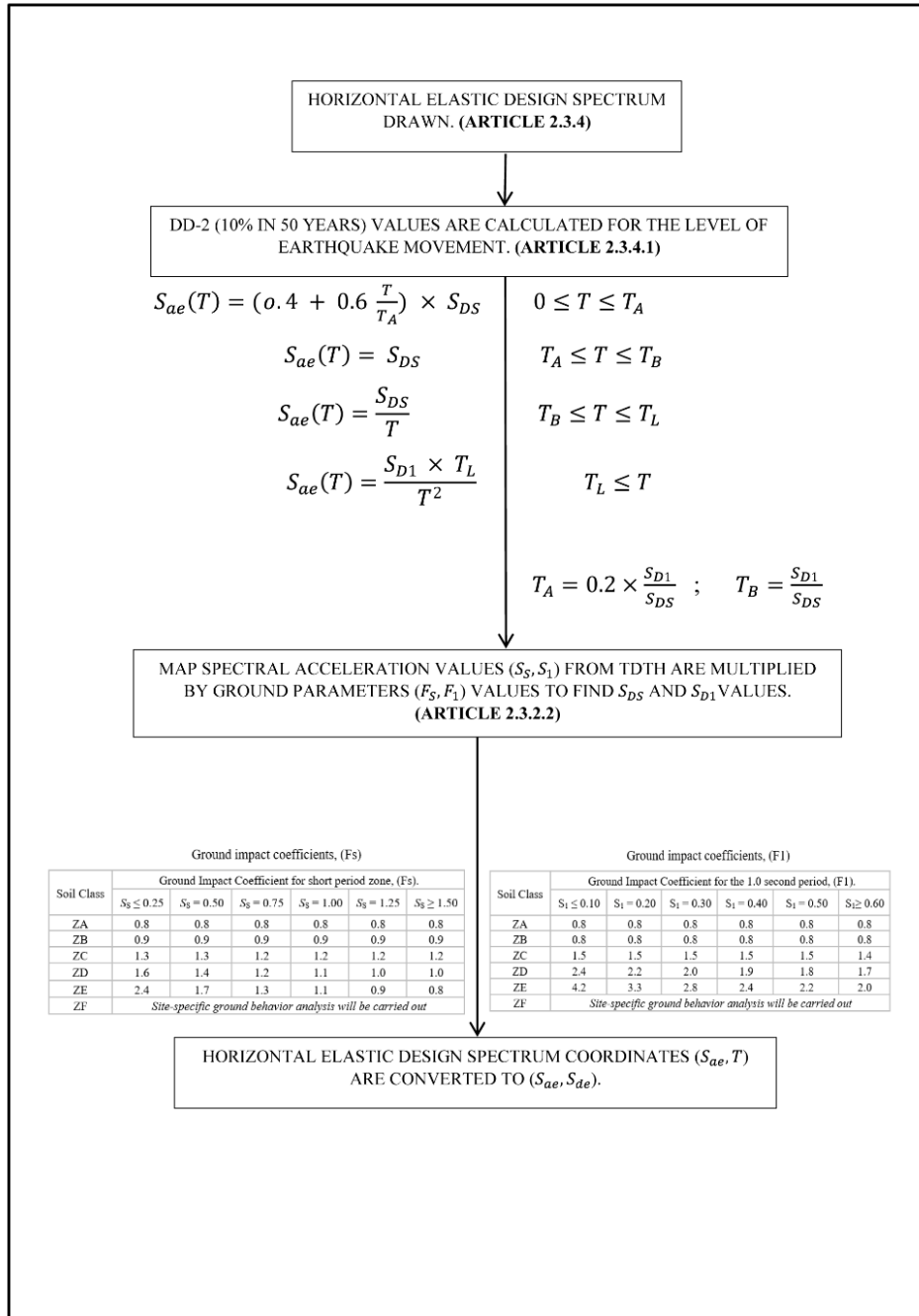


Figure 4.7: Flow Chart of Analysis 2.

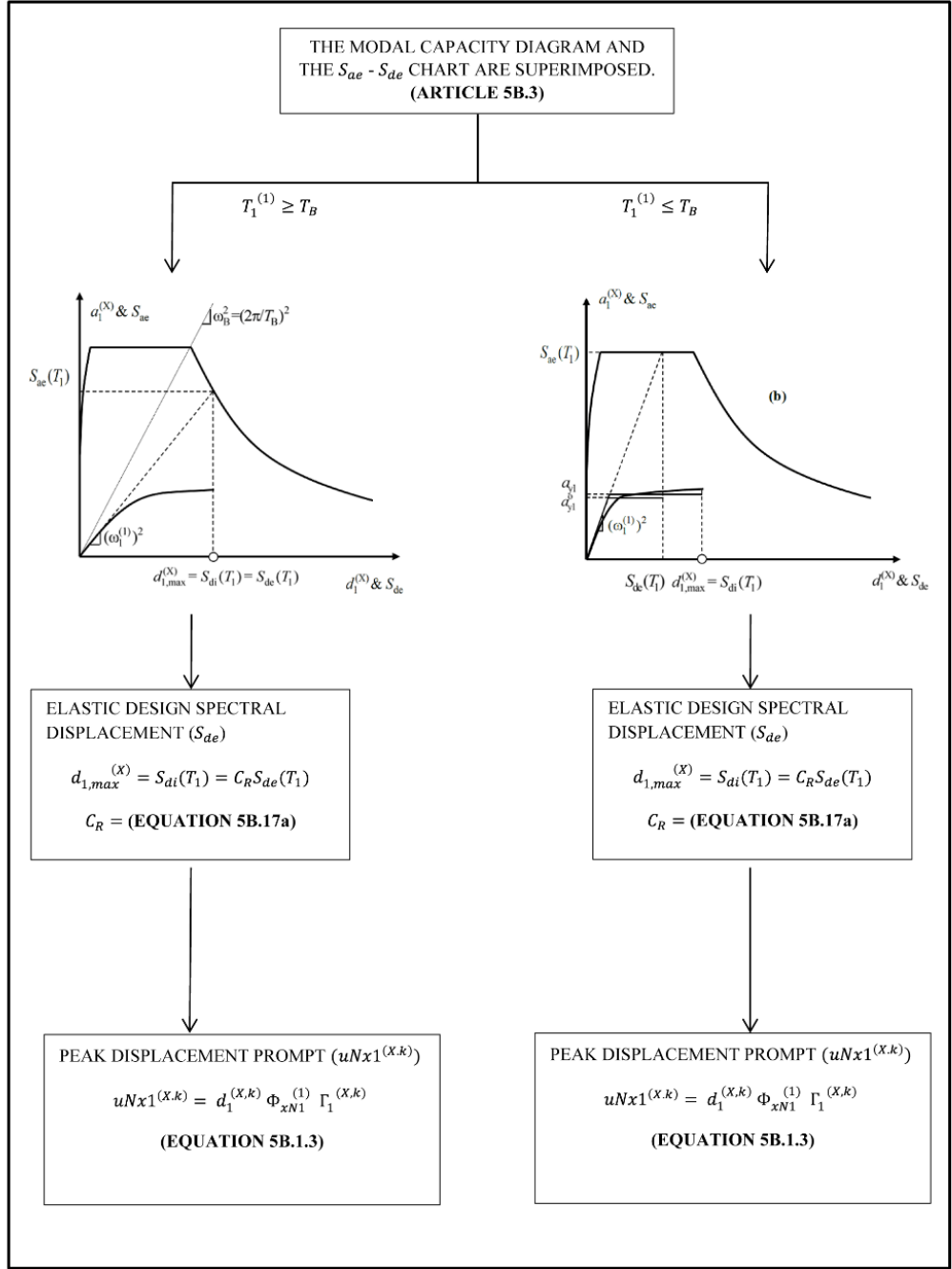


Figure 4.8: Flow Chart of Analysis 3.

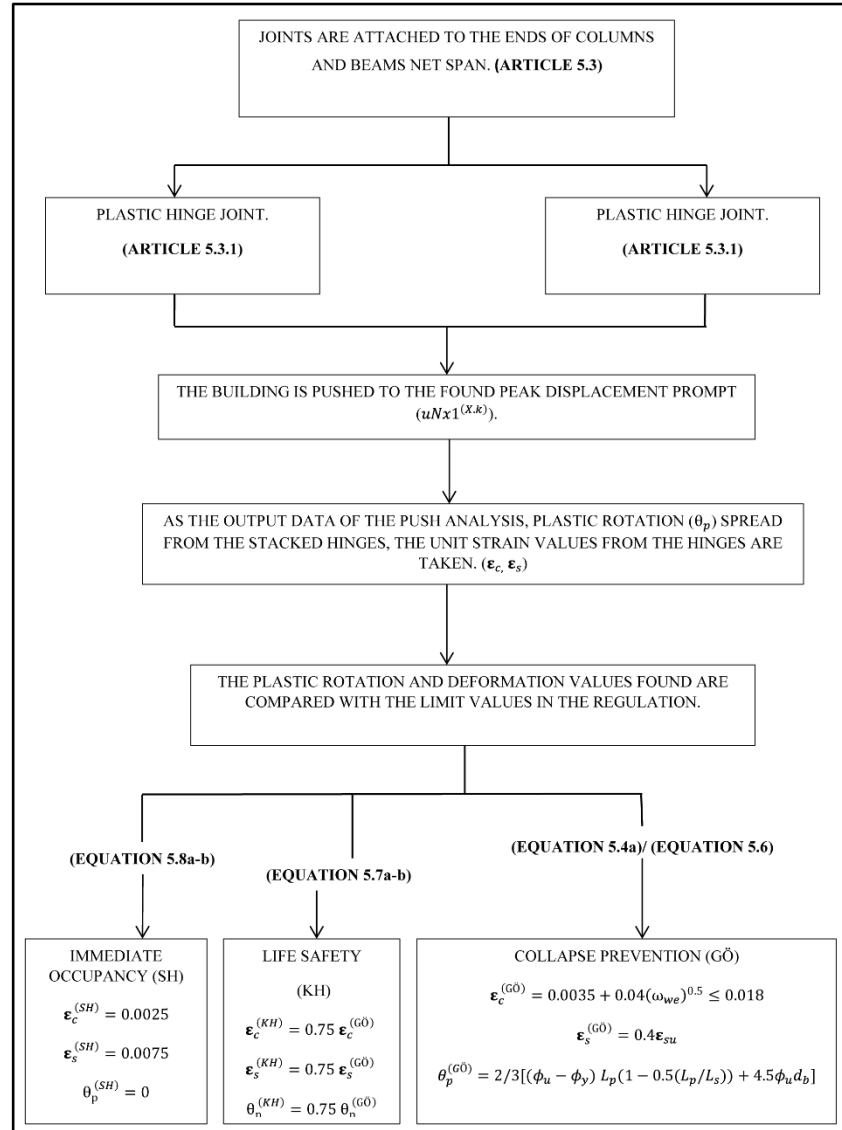


Figure 4.9: Flow Chart of Analysis 4.

5. NUMERICAL MODELING AND RESULTS

5.1. Elevation of Structural Model

In The frame system illustrated in figure 5.1. (a) represents an existing structure with four story and four bays. There is no irregularity in the structure. This structure was strengthened in three different ways by using different types of braced frames and applying reinforcement shapes are given in figure 5.1. The study's goal is to determine the most appropriate braced frame from (b) diagonal braced frames, (c) inverted braced frames, (d) X-braced frames according to (a) existing building.

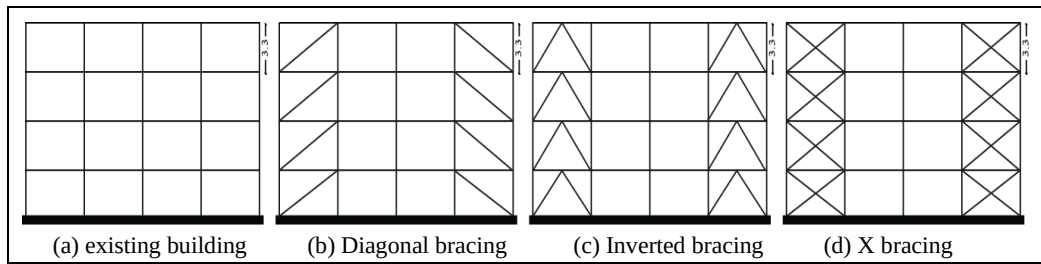


Figure 5.1: Elevation of Structural Model in X direction for Numerical Study.

All braced frames strengthened at edge bays, but the only difference is how was the set bracing. Nonlinear analysis was used to compare the results of these three alternative strengthening frames and the existing building. The capacity values obtained from the results were used for comparison.

5.2. Identification of Column, Beam and Braces Plastic Hinges

After the properties of C20 concrete and S420 reinforcing steels are defined in the analysis model, plastic hinges will be assigned to columns, beams and braces.

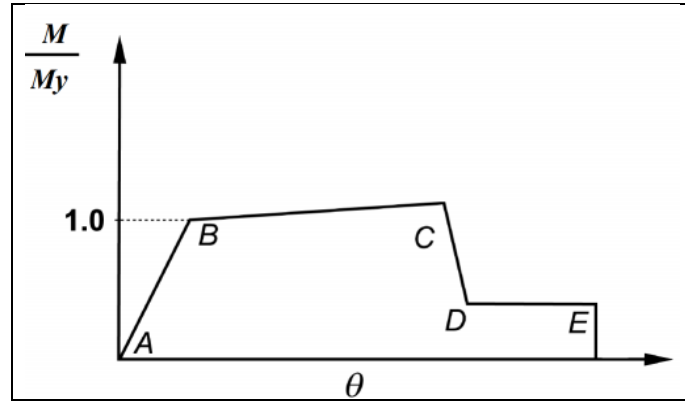


Figure 5.2: Typical moment-rotation relationship for a RC section.

In the graph in Figure 5.2, it can be said that linear behavior occurs in the cross-section between point A and point B, permanent deformations occur in the section between point B and point C, and the section reaches the moment of power depletion at point C. After this point, the capacity of the section can be reduced a little more and the amount of permanent deformation can be increased. Moment and rotation values at points A, B and C were obtained by moment-curvature analysis for column and beam sections, and FEMA - 356 was used for points D and E. Data entry cannot be made at point A for columns and beams to be modeled with the stacked plastic behavior in the ETABS environment. At point B, the moment value is taken into account as the yield moment value of the section, and the plastic rotation value as zero. Therefore, permanent deformation will not occur at point B. The winding effect on the beams is not taken into account.

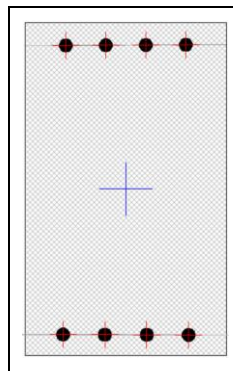


Figure 5.3: Beam Section (B30X50).

Section top reinforcements: 4 ϕ 22, Section sub-reinforcements: 4 ϕ 16. The values obtained as a result of the moment curvature analysis performed in the SAP2000 environment of the beam section with type B30X50 are given in Table 5.1.

Table 5.1: Moment Curvature Analysis for Beam Section B30X50.

Interaction Surface	ϕ_y (rad/m)	ϕ_u (rad/m)	M_y (KNm)	M_u (KNm)	ϕ_p (rad/m)	L_p (m)	θ_p (rad)	M_u / M_y
0°	0.007288	0.1088	226.2419	262.146	0.101512	0.25	0.025378	1.158698

The graphical representation of the values given in Table 5.3 on the points A, B and C defined in Figure 5.2 are given in Figures 5.4.

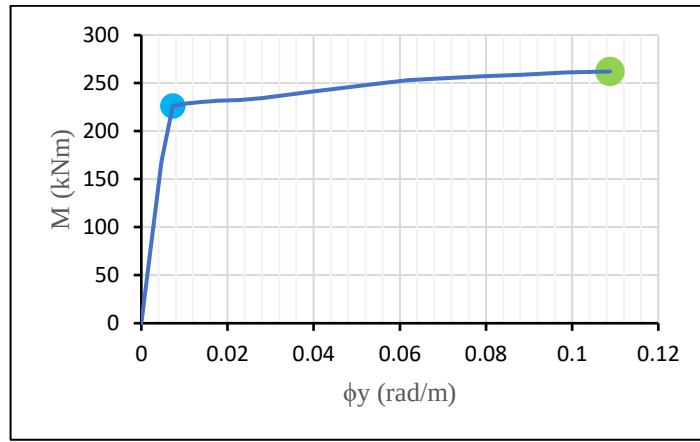


Figure 5.4: Beam Section (B30X50) moment curvature diagram (0° interaction surface).

These processes were also carried out for all beam types and the necessary moment-rotation data for plastic hinge definition were obtained in ETABS environment. The selection of the joint type for beams in the ETABS environment is shown in figure 5.5.

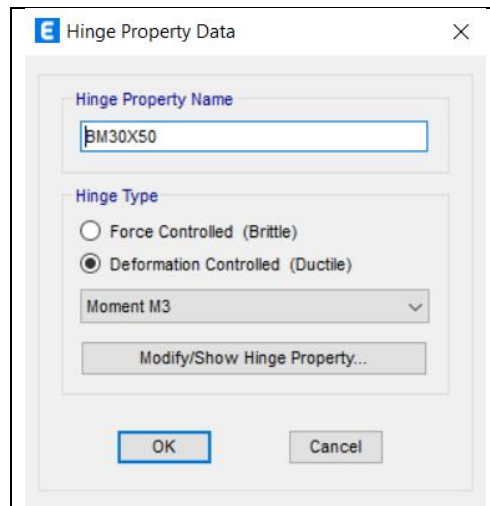


Figure 5.5: Selection of M3 type plastic joint for the beam.

The definition of moment-rotation values obtained for beam types in ETABS environment is shown in Figure 5.6.

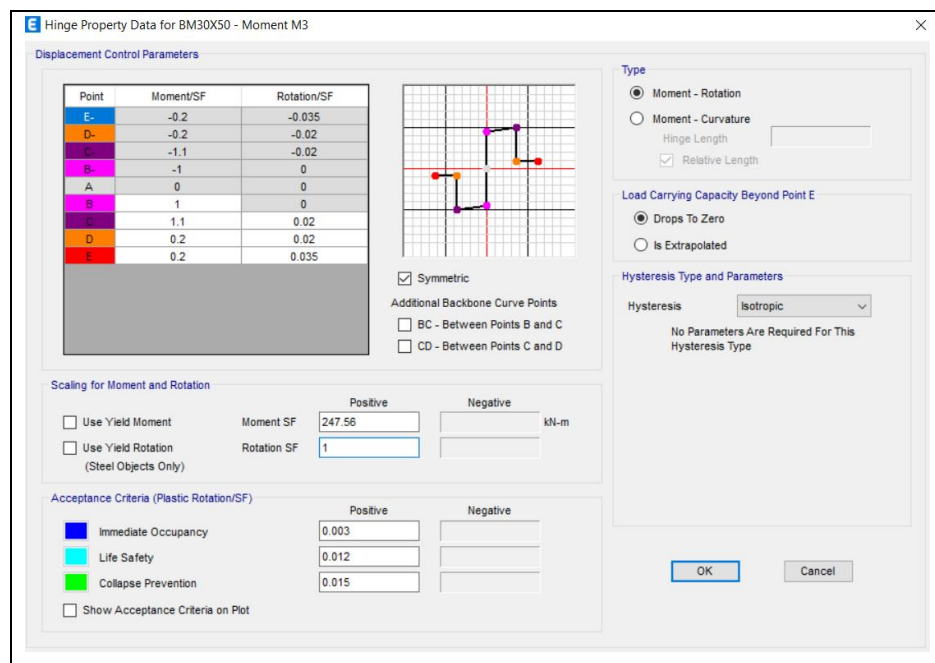


Figure 5.6: Identification of beam plastic joint information in ETABS environment.

The assignment of the stacked plastic hinges at the ends of the beams with defined plastic hinge information to both ends of the beams is shown in Figure 5.7.

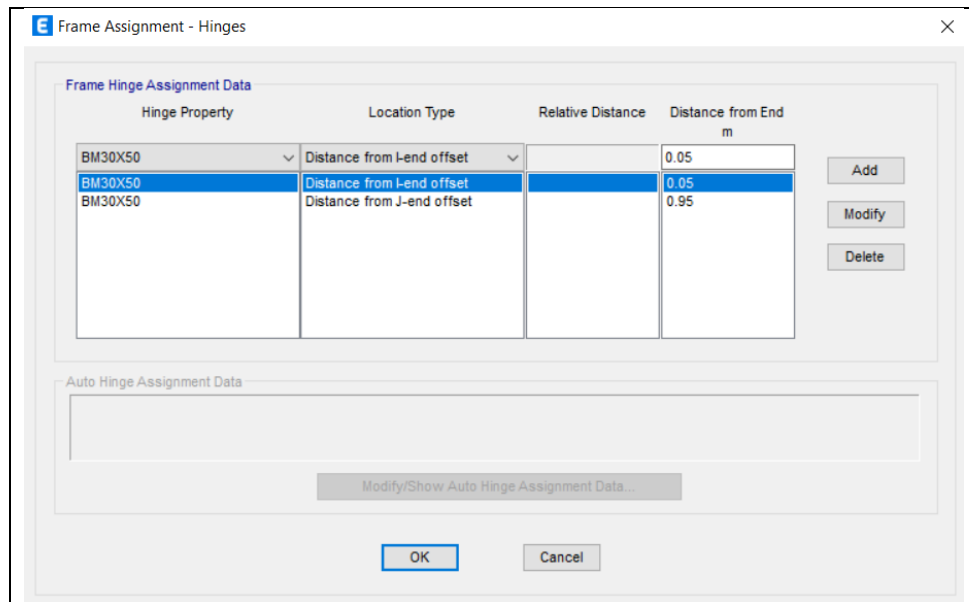


Figure 5.7: Assignment of plastic hinges to the beam ends.

Column sections were also defined in the SAP2000 environment and moment curvature analyzes were performed. As an example, obtaining the plastic hinge data of the C30X50 rectangular column and its definition in the ETABS environment will be shown.

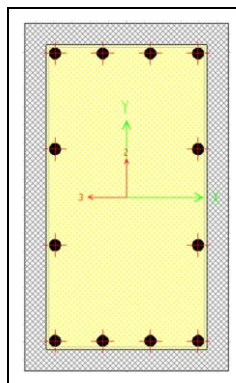


Figure 5.8: Column Section C30X50.

Column longitudinal reinforcements: $12\phi 20$, Column transverse reinforcements: $2\phi 10/100$ both directions. The winding effect for the columns is taken into account in the SAP2000 environment as shown in figure 5.9.

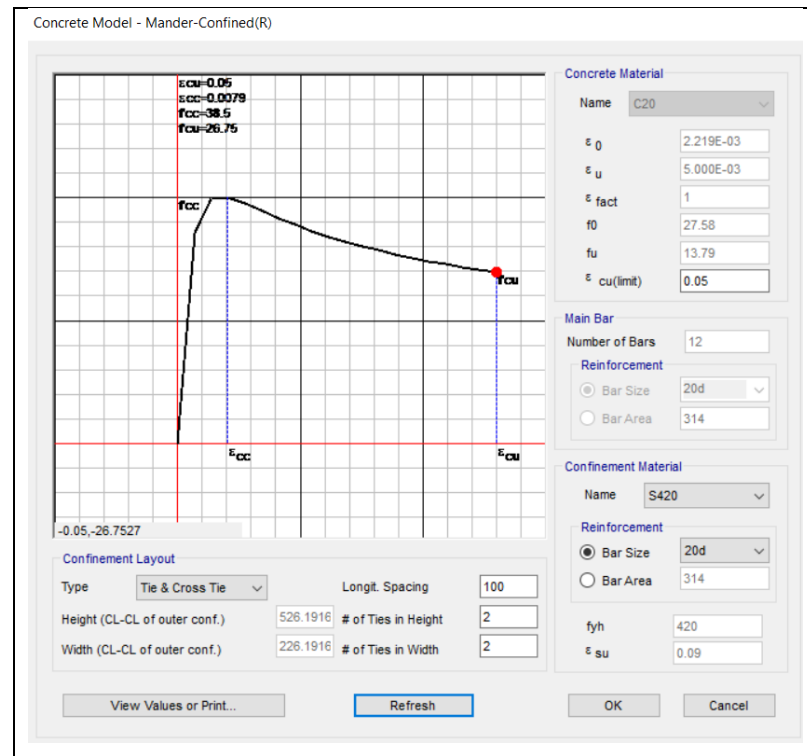


Figure 5.9: Consideration of the confined effect.

These processes were also performed for all column types and the necessary moment-rotation data for plastic hinge identification were obtained in ETABS environment. The selection of the hinge type for the columns in ETABS environment is shown in figure 5.10.

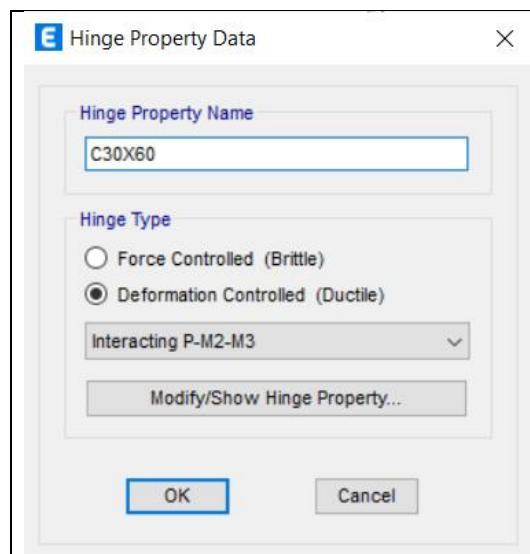


Figure 5.10: P-M2-M3 type plastic joint selection for the column.

Identification of the plastic hinge information of the column types in the ETABS environment is shown in figure 5.11.

Frame Hinge Property Data for C30X60 - Interacting P-M2-M3

Hinge Specification Type

- ☒ Moment - Rotation
- ☐ Moment - Curvature

Hinge Length:

☒ Relative Length

Scale Factor for Rotation (SF)

- ☐ SF is Yield Rotation per ASCE 41-13 Eqn. 9-2 (Steel Objects Only)
- ☒ User SF: rad

Load Carrying Capacity Beyond Point E

- ☒ Drops To Zero
- ☐ Is Extrapolated

Symmetry Condition

- ☐ Moment Rotation Dependence is Circular
- ☒ Moment Rotation Dependence is Doubly Symmetric about M2 and M3
- ☐ Moment Rotation Dependence has No Symmetry

Requirements for Specified Symmetry Condition

- Specify curves at angles of 0° and 90°.
- If desired, specify additional intermediate curves where: 0° < curve angle < 90°.

Axial Forces for Moment Rotation Curves

Number of Axial Forces:

Curve Angles for Moment Rotation Curves

Number of Angles:

Figure 5.11: Defining column joint properties.

The definition of the number of axial forces and the number values to be considered in order to define the plastic joint information of the column types is shown in Figure 5.12.

Axial Forces for C30X60 - Interacting P-M2-M3

This Number of Axial Force Values is Specified

Number of Axial Forces:

Axial Force Data

	Force kN
1	-1640.7
2	2978.5

Figure 5.12: Axial forces based on the definition of column joint properties.

The angles and number of column cross-section interaction surfaces to be considered for defining the plastic hinge information of the column types are defined as shown in Figure 5.13.

	Angle deg
1	0
2	45
3	90

Figure 5.13: The number and angles of interaction surfaces based on the definition of column joint properties.

Data entry of moment-rotation values for column section interaction surface and axial force values in ETABS environment is given in Figure 5.14.

Point	Moment/Yield Mom	Rotation/SF
A	0	0
B	1	0
C	1.1	0.015
D	0.2	0.015
E	0.2	0.025

Figure 5.14: Column joint definition for different axial load and interaction surfaces.

As mentioned, section 5.2.1 braces must be modelled as columns with a potential plastic hinge located in the middle of the brace to account for both primary axial effects and secondary flexural effects. These processes were also performed for all column types and the necessary moment-rotation data for plastic hinge identification were obtained in ETABS environment. The selection of the hinge type for the brace in ETABS environment is shown in figure 5.15.

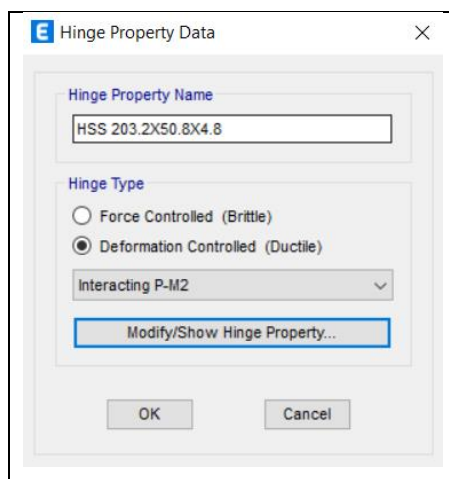


Figure 5.15: P–M2 type plastic joint selection for the column.

Data entry of moment-rotation values for column section interaction surface and axial force values in ETABS environment is given in Figure 5.16.

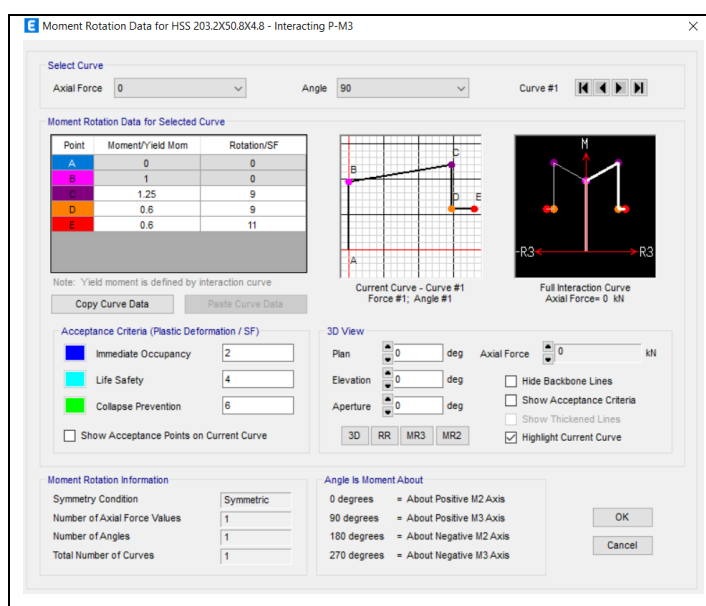


Figure 5.16: Brace joint definition for different axial load and interaction surfaces.

The assignment of the stacked plastic hinges at the ends of the braces with defined plastic hinge information to middle is shown in Figure 5.17.

Hinge Property	Location Type	Relative Distance	Distance from End m
HSS 203 2X50.8X4.8	Relative to clear length	0.5	
HSS 203 2X50.8X4.8	Relative to clear length	0.5	

Figure 5.17: Assignment of plastic hinges middle.

5.3. Brace Design for CBF

Steel Concentrically Braced Frames are excellent for seismic frame systems since they are strong, stiff, and ductile. The performance of the brace improves the level of the seismic response of Concentrically Braced Frames. To gain optimum performance from a CBF, the brace must fail before any other component of the frame. This is important because although the frame may sustain significant damage during an earthquake, it is expected to remain stable, and the building must be capable of resisting gravity loads and of withstanding aftershocks without collapse. Braces are the frame's weakest link of all other frame elements (columns, beams, and connections) are stronger than braces. The bracing was tested to ensure it meets members with good energy dissipation capability and fracture life in Table 5.2.

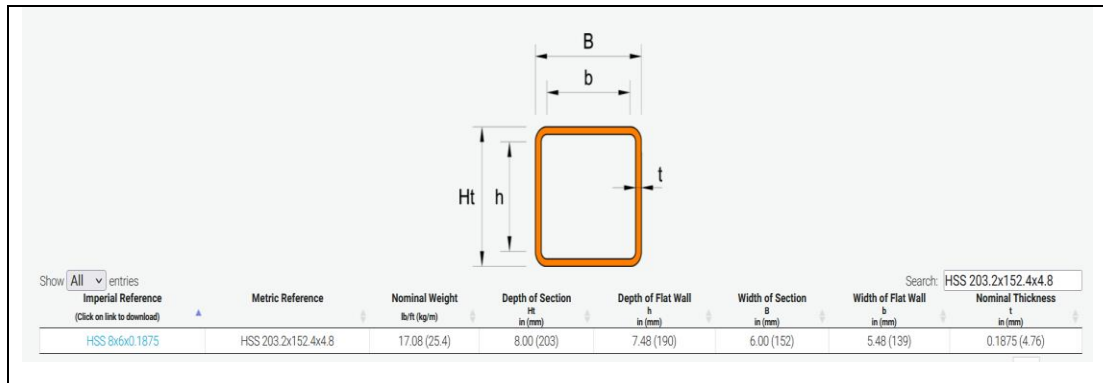


Figure 5.18: Brace Section.

Table 5.2: Brace Design for CBF

Brace Design for CBF															
Story	Brace Section	A _g (mm ²)	t (mm)	L _{h/c} (m)	L _{c/v} (KL/r)	KL/r ≤ 200	D/t (b/t)	Q _e (kN)	F _b Q _e (kN)	F _b D _L (kN)	F _b L _L (kN)	P _r (kN)	P _r (N/mm ²)	F _{cr} (N/mm ²)	P _n (kN)
Roof	HSS 203.2X30.0X4.8	2110	66.4	4.8104	72.44586	OK	8.5	245.61	173.67591	19.25	5.5	206.4329	376.0993174	263.1827162	555.3155
3rd	HSS 203.2X30.0X4.8	2110	66.4	4.8104	72.44586	OK	8.5	433.39	306.45537	19.25	5.5	339.2134	376.0993174	263.1827162	555.3155
2nd	HSS 203.2X30.0X4.8	2110	66.4	4.8104	72.44586	OK	8.5	558.58	394.97501	19.25	5.5	427.733	376.0993174	263.1827162	555.3155
1st	HSS 203.2X30.0X4.8	2110	66.4	4.8104	72.44586	OK	8.5	621.17	493.52592	19.25	5.5	526.2839	376.0993174	263.1827162	555.3155

5.4. Performance Analysis According to TBDY-2018

5.4.1. Existing building

The building has a three-dimensional model. The building has four floors with a rigid diaphragm, and columns and beams are described as rod elements.

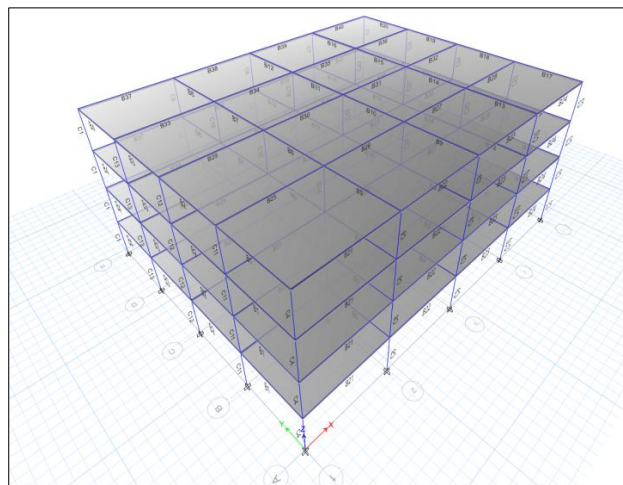


Figure 5.19: 3D Perspective View.

The effective section stiffness factors are applied to the bar, and shell members under bending effect. These values are taken from Table 3.4. According to Equation 4.5 and Equation 4.6, effective section stiffnesses are calculated.

After defining the effective section stiffness factors for the elements in the established model, the boundary conditions in the regulation were validated in order to execute the pushover analysis. The construction has been observed to meet the boundary conditions. The ratio of the effective mass of the first (dominant) vibration mode, calculated by assuming linear elastic behavior in both earthquake directions, to the mass participation of the whole building corresponds to the value of 0.70 (Table 5.3).

Table 5.3: Mass Participation Rate.

Modal Mass Participation Rate			
Mod	Period	UX	UY
1	1.3	0.8434	
2	1.1		0.7981

The torsional irregularity coefficient calculated by assuming linear elastic behavior in both earthquake directions satisfies the condition $\eta_{bi} < 1.4$. (Table 5.4, Table 5.5)

Table 5.4: X- Direction Torsional Irregularity.

Y - Direction					
Story	Load Case/Combo	Maximum Drift	Average Drift	Ratio	KONTROL
	DIRECTION	m	m	$\eta_{bi} > 1,4$	
Floor4	EQX	0.0038	0.0036	1.0554	OK
Floor3	EQX	0.0059	0.0056	1.0536	OK
Floor2	EQX	0.0069	0.0066	1.0520	OK
Floor1	EQX	0.0045	0.0043	1.0499	OK

Table 5.5: Y- Direction Torsional Irregularity.

X- Direction					
Story	Load Case/Combo	Maximum Drift	Average Drift	Ratio	KONTROL
	DIRECTION	m	m	nbi>1,4	
Floor4	EQY	0.0034	0.0030	1.1456	OK
Floor3	EQY	0.0050	0.0044	1.1479	OK
Floor2	EQY	0.0057	0.0049	1.1504	OK
Floor1	EQY	0.0035	0.0030	1.1537	OK

After it is comprehended that the building meets the requirements of the pushover analysis, the building is pushed up to a particular value, and a pushover curve is obtained as the base shear force-peak displacement ($V_{tx}-u_{Nx}$) relationship for both earthquake directions. (Figure 5.20, Figure 5.21)

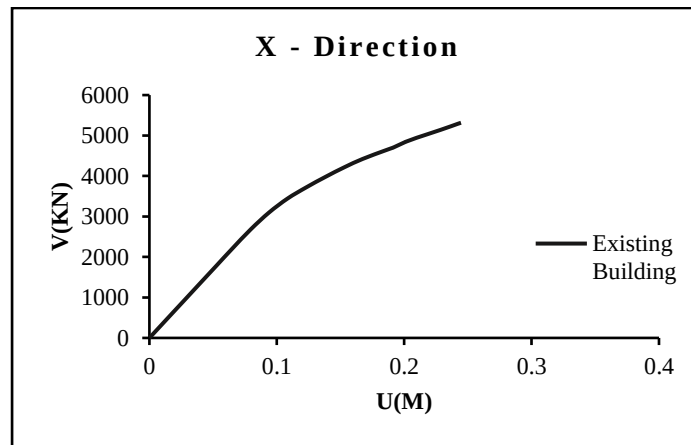


Figure 5.20: X - Direction Base Shear Force-peak Displacement Graph.

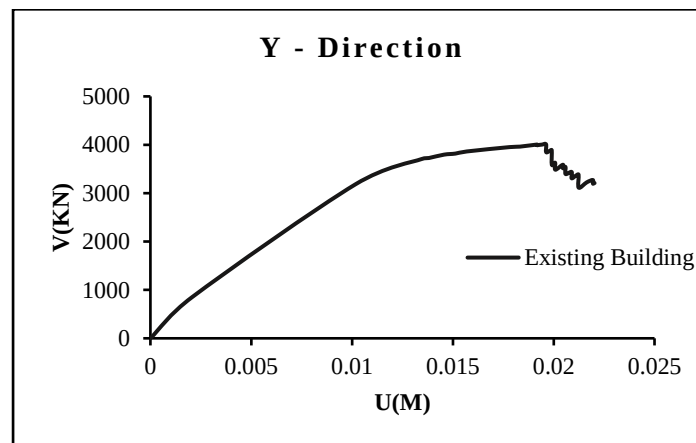


Figure 5.21: Y - Direction Base Shear Force-peak Displacement Graph.

After pushing the building up to a certain point, the pushover curve was obtained. In order to find the pushover demanded by the earthquake by placing this curve inside the linear earthquake spectrum, which is the spectral displacement-spectral acceleration (S_{de} - S_{ae}) graph, the base shear force-peak displacement (V_{tx} - u_{Nx}) is converted, and its coordinates are modal displacement-modal acceleration (d_1 , a_1) modal capacity diagram with has been created.

Table 5.6: Values from the Modal Analysis.

Direction	X	Y
Γ_{N1}	23	23.5
Φ_{N1}	3.50E-02	3.10E-02
$\sum M(t)$	5542	5542
Modal mass	0.84	0.79
M_1	2512	2506

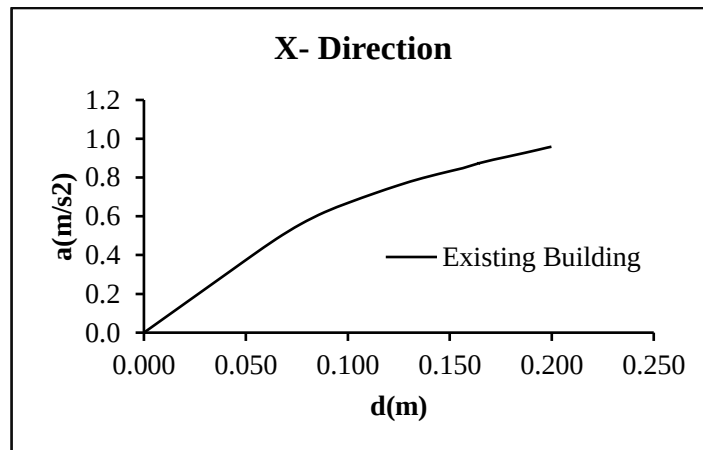


Figure 5.22: X - Direction Modal Capacitance Diagram.

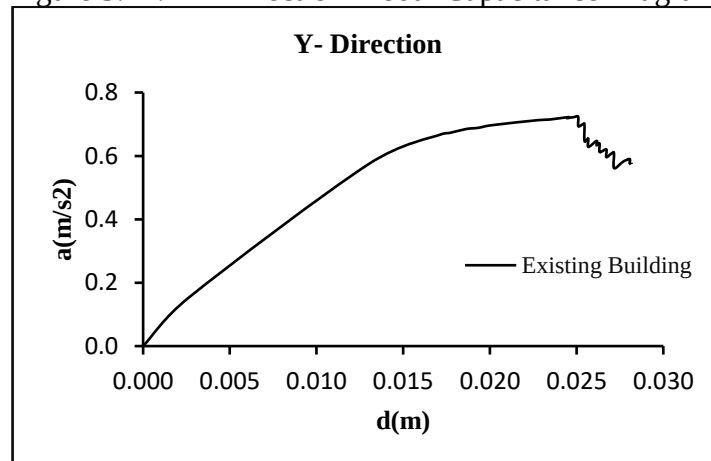


Figure 5.23: Y - Direction Modal Capacitance Diagram.

S_{DS} (short-period design spectral acceleration coefficient) and S_{D1} (map spectral acceleration coefficient for 1.0 second period) were found depending on the ground group and earthquake ground level on the "Turkey Earthquake Hazard Map", which was put into practice with TBDY-2018 [18]. By using these values, the horizontal elastic design spectrum is formed. (Figure 5.24)

Rapor Başlığı:	Location		
Deprem Yer Hareketi Düzeyi	DD-2	50 yılda aşılma olasılığı %10 (tekrarlanma periyodu 475 yıl) olan deprem yer hareketi düzeyi	
Yerel Zemin Sınıfı	ZC	Çok sıkı kum, çakıl ve sert kil tabakaları veya ayrışmış, çok çatlaklı zayıf kayalar	
Enlem:	40.789965°		
Boylam	29.492587°		
Çıktılar			
$S_S = 1.361$	$S_1 = 0.373$	$PGA=0.562$	$PGV=42.657$

Figure 5.24: TDTH output.

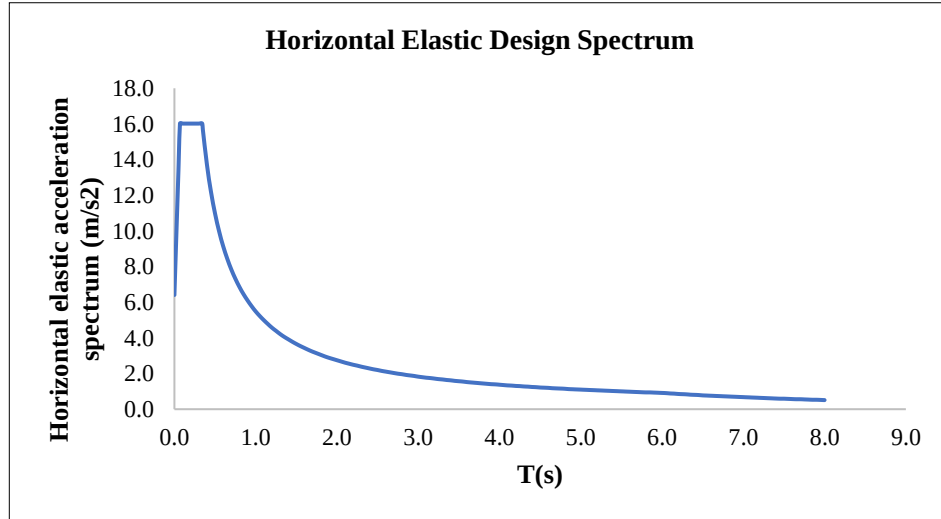


Figure 5.25: Elastic Acceleration Spectrum Graph.

The generated horizontal elastic design spectrum and the modal capacity diagram are superimposed, and the linear elastic spectral displacement value of the building is found. These elastic displacement demands were found using the Sae-Sde (Sae-Sde curve) values corresponding to the period of the building. It is 12 cm for the X direction and 7.5 cm for the Y direction. (Figure 5.26, Figure 5.27)

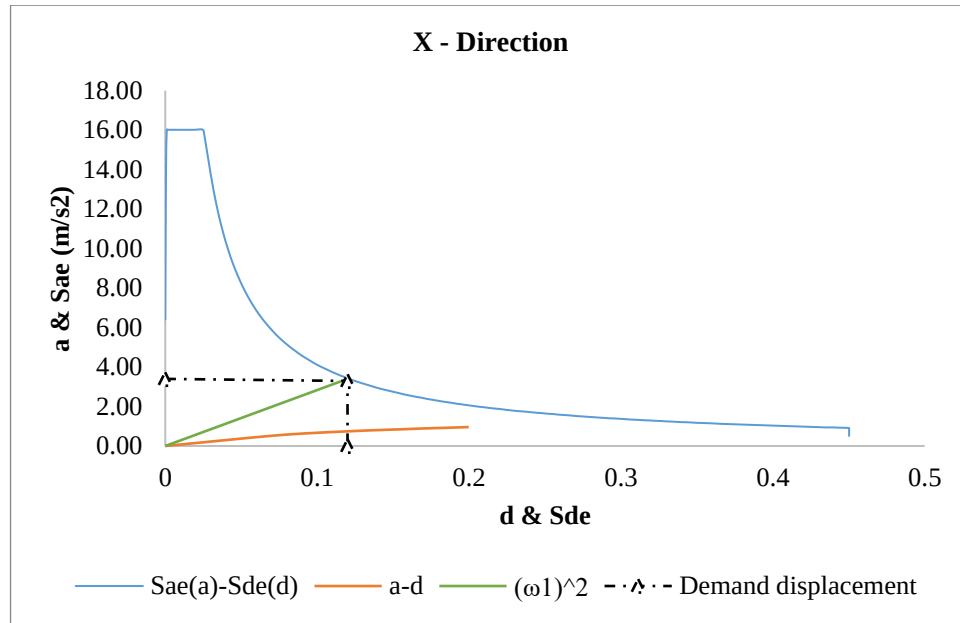


Figure 5.26: X - Direction Modal Displacement Request.

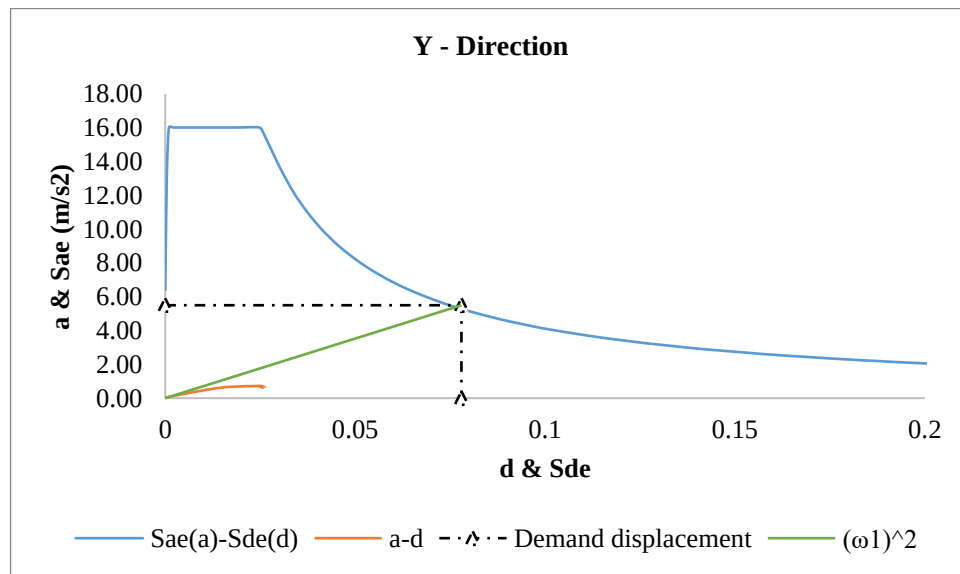


Figure 5.27: Y - Direction Modal Displacement Request.

The linear elastic spectral displacement value of the building was converted into nonlinear spectral displacement value with the help of the spectral displacement ratio CR. The period of the building is 1.3 s and 1.10 s in the X and Y directions, respectively. Since these values are greater than the TB value, which is the corner period found depending on the ground where the building is located, the CR value is taken as 1 in accordance with the equal displacement rule [18]

The nonlinear spectral displacement value of the building was calculated. The demand displacement value of the building is calculated by multiplying this value by the amplitude of the constant mode shape and the modal contribution factor, which did not change at all during the pushover calculation. These values were calculated as 20 cm for the X direction and 2.6 cm for the Y direction. (Table 5.7)

Table 5.7: X-Y direction nonlinear demand displacement values.

X Direction			Y Direction		
$T_X > T_B$			$T_Y > T_B$		
a	0.78	m/s ²	a	1.2	m/s ²
d	0.12	m	d	0.12	m
w	22.51	--	w	19.46	--
w	4.51	rad/s	w	6.21	rad/s
T	1.3	s	T	1.1	s
Sae	3.3	m/s ²	Sae	4.5	m/s ²
Sde	0.12	m	Sde	0.075	m
CR	1	--	CR	1	--
Sdi	0.12	m	Sdi	0.075	m
di	0.20	m	di	0.026	m

At this stage, the elements in the existing state of the building have been examined. Plastic hinges are used to detect damage in the elements. The distributed plastic hinge model is employed in columns. For beams, the stacked plastic hinge model is employed. Plastic hinges are used in columns and beams just outside the connection locations.

The boundary plastic rotation values of the sections were calculated according to each performance level. Pre-failure curvature (ϕ_u), the section from concrete or steel materials is the maximum strain curvature one can achieve. The yield curvature (ϕ_y) is the yield curvature of the reinforcement in the section. The plastic hinge length (L_p) is equal to half the height of the section in the running direction. The shear span (L_s) is taken as half of the net span in the column beams. The mean longitudinal bar diameter (d_b) is the mean longitudinal bar diameter in the section [18]. With the help of these values, the “Before Collapse” plastic rotation limit value was calculated.

Multiplying the plastic rotation limit value before collapse by 0.75 gave the “Controlled Damage” plastic rotation limit value. The plastic rotation limit value section of “Limited Damage”, which is the first damage limit, is taken into account as 0 (zero).

$$\theta_p^{(G\ddot{O})} = \frac{2}{3} \times \left[(\phi_u - \phi_y) L_p \left(1 - 0.5 \frac{L_p}{L_s} \right) + 4.5 \phi_u d_b \right]$$

$$\theta_p^{(KH)} = 0.75 \times \theta_p^{(G\ddot{O})} \quad (5.1)$$

$$\theta_p^{(SH)} = 0$$

Pre-failure curvature in the X and Y direction of the columns are calculated, when the demand displacement value of the earthquake is applied to the building, the plastic rotation values formed in the elements are taken. These values were compared with the limit values and the damage status of the building was checked.

Since the spacing of stirrups in columns and beams is much higher than the range specified in the regulations, the uncoated concrete model is used as the material behavior. Plastic rotation limit values are calculated for each element according to material capacity. Unit deformation “Limited Damage” limit value is 0.0035, “Controlled Damage Limit” limit value is 0.0026, “Prevention of Migration” limit value is 0.0025, which is the same for each element.

While evaluating the existing buildings, it is aimed to ensure that people leave the building alive after the earthquake. In the regulation, this requirement is referred to as "Controlled Damage Performance Level". According to the Controlled Damage performance level, the contribution of the columns to the shear force carried by the columns at each floor should be less than 20% [18].

Table 5.8: Damage Values in the X- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G\ddot{O})}$	$\epsilon_s^{(G\ddot{O})}$	$\theta_p^{(G\ddot{O})}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.00278	0.0025	0.008	0	4.49E-03	3.47E-03	GH	IH
4stry	C5	C-60/35	0.004	0.032	0.005	0.003	0.024	0.00375	0.0025	0.008	0	2.54E-04	4.64E-04	BH	GH
4stry	C6	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.00278	0.0025	0.008	0	8.03E-04	3.62E-03	GO	GO
4stry	C7	C-60/35	0.004	0.032	0.0058	0.003	0.024	0.00435	0.0025	0.008	0	3.00E-04	3.57E-04	IH	BH
4stry	C2	C-60/35	0.004	0.032	0.0044	0.003	0.024	0.0033	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.004	0.003	0.024	0.003	0.0025	0.008	0	1.15E-03	1.61E-03	BH	IH
4stry	C16	C-60/30	0.004	0.032	0.004	0.003	0.024	0.003	0.0025	0.008	0	9.86E-03	1.37E-03	GH	IH
4stry	C19	C-60/30	0.004	0.032	0.0044	0.003	0.024	0.0033	0.0025	0.008	0	4.13E-04	3.14E-03	GO	IH
4stry	C22	C-60/30	0.004	0.032	0.005	0.003	0.024	0.00375	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0046	0.003	0.024	0.00345	0.0025	0.008	0	3.21E-04	5.72E-03	IH	GH

Table 5.9: Damage Values in the Y- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0036	0.003	0.024	0.002692	0.0025	0.008	0	5.49E-03	4.47E-3	GH	IH
4stry	C5	C-60/35	0.004	0.032	0.0049	0.003	0.024	0.003638	0.0025	0.008	0	4.97E-03	3.47E-02	GO	IH
4stry	C6	C-60/35	0.004	0.032	0.0036	0.003	0.024	0.002692	0.0025	0.008	0	8.03E-04	3.62E-03	BH	GO
4stry	C7	C-60/35	0.004	0.032	0.0056	0.003	0.024	0.00422	0.0025	0.008	0	3.00E-04	3.57E-04	IH	BH
4stry	C2	C-60/35	0.004	0.032	0.0043	0.003	0.024	0.003201	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.0039	0.003	0.024	0.00291	0.0025	0.008	0	1.15E-03	1.61E-03	GO	IH
4stry	C16	C-60/30	0.004	0.032	0.0039	0.003	0.024	0.00291	0.0025	0.008	0	9.86E-04	1.37E-03	BH	BH
4stry	C19	C-60/30	0.004	0.032	0.0043	0.003	0.024	0.003201	0.0025	0.008	0	4.13E-04	3.14E-04	GO	IH
4stry	C22	C-60/30	0.004	0.032	0.0049	0.003	0.024	0.003638	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0045	0.003	0.024	0.003347	0.0025	0.008	0	3.21E-04	3.72E-04	BH	BH

The shear force corresponding to brittle fracture in columns and beams was compared with the capacities of these load-bearing elements and it was checked whether the elements were “ductile” or “brittle”. The shear force is determined according to the calculation in TS500 [9]. (Table 5.10, Table 5.11, Table 5.12)

Table 5.10: X - Direction Shear Control on Columns.

X - Direction																	
Column Information			Concrete Contribution					Stirrup Contribution						Check		Fracture Type	
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{tm} (MPa)	γ	N _d (N)	A _c (mm²)	V _{cr} (mm²)	V _c (kN)	A _{sw} (mm²)	s (mm)	f _{ywd} (MPa)	V _w (kN)	V _t (kN)		V _d (kN)
4stry	C1	600	350	28	2.2	0.1	160207	210000	142	114	79	150	365	67	181	32	DUCTILE
4stry	C5	600	350	28	2.2	0.1	304197	210000	142	114	79	150	365	67	181	14	DUCTILE
4stry	C6	600	350	28	2.2	0.1	301900	210000	142	114	79	150	365	67	181	21	DUCTILE
4stry	C7	600	350	28	2.2	0.1	304197	210000	142	114	79	150	365	67	181	30	DUCTILE
4stry	C2	600	350	28	2.2	0.1	160207	210000	142	114	79	150	365	67	181	21	DUCTILE
4stry	C13	600	350	28	2.2	0.1	293	210000	142	114	79	150	365	67	181	53	DUCTILE
4stry	C16	600	300	28	2.2	0.1	54609	180000	122	98	79	150	365	57	155	51	DUCTILE
4stry	C19	600	300	28	2.2	0.1	542	180000	122	98	79	150	365	57	155	38	DUCTILE
4stry	C22	600	300	28	2.2	0.1	546	180000	122	98	79	150	365	57	155	32	DUCTILE
4stry	C25	600	300	28	2.2	0.1	293	180000	122	98	79	150	365	57	155	14	DUCTILE
4stry	C12	600	350	28	2.2	0.1	284	210000	142	114	79	150	365	67	181	21	DUCTILE
4stry	C15	600	300	28	2.2	0.1	227	180000	122	98	79	150	365	57	155	30	DUCTILE

Table 5.11: Y - Direction Shear Control on Columns

Y - Direction																	
Column Information				Concrete Contribution					Stirrup Contribution				Check		Fracture Type		
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{tm} (MPa)	γ	N _d (N)	A _c (mm²)	V _{cr} (kN)	V _c (kN)	A _{sw} (mm²)	s (mm)	f _{ywd} (MPa)	V _w (kN)		V _t (kN)	V _d (kN)
4stry	C1	600	350	28	2.2	0.1	162301	210000	142	114	79	150	365	66	182	53	DUCTILE
4stry	C5	600	350	28	2.2	0.1	302157	210000	142	114	79	150	365	66	182	51	DUCTILE
4stry	C6	600	350	28	2.2	0.1	302358	210000	142	114	79	150	365	66	182	38	DUCTILE
4stry	C7	600	350	28	2.2	0.1	304078	210000	142	114	79	150	365	66	182	32	DUCTILE
4stry	C2	600	350	28	2.2	0.1	246207	210000	142	114	79	150	365	66	182	14	DUCTILE
4stry	C13	600	350	28	2.2	0.1	271	210000	142	114	79	150	365	66	182	21	DUCTILE
4stry	C16	600	300	28	2.2	0.1	53278	180000	122	98	79	150	365	56	156	30	DUCTILE
4stry	C19	600	300	28	2.2	0.1	456	180000	122	98	79	150	365	56	156	38	DUCTILE
4stry	C22	600	300	28	2.2	0.1	456	180000	122	98	79	150	365	56	156	32	DUCTILE
4stry	C25	600	300	28	2.2	0.1	237	180000	122	98	79	150	365	56	156	14	DUCTILE
4stry	C12	600	350	28	2.2	0.1	248	210000	142	114	79	150	365	66	182	21	DUCTILE
4stry	C15	600	300	28	2.2	0.1	273	180000	122	98	79	150	365	56	156	30	DUCTILE

Table 5.12: Shear Control in Beams.

X-Y Direction															
Beam Information				Concrete Contribution				Stirrup Contribution				Check			Fracture Type
Story	Beam	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{cm} (MPa)	A _c (mm ²)	V _{cr} (kN)	V _c (kN)	A _{sw} (mm ²)	s (mm)	f _{yd} (MPa)	V _w (kN)	V _r (kN)	V _e (kN)	
4stry	B1	450	300	28	2.2	135000	102	82	79	150	365	21	103	89	DUCTILE
4stry	B2	450	300	28	2.2	135000	102	82	79	150	365	21	103	120	DUCTILE
4stry	B3	450	300	28	2.2	135000	114	91	79	150	365	21	112	142	DUCTILE
4stry	B4	450	300	28	2.2	135000	114	91	79	150	365	21	112	174	DUCTILE
4stry	B5	450	300	28	2.2	135000	38	30	79	150	365	35	65	42	DUCTILE
4stry	B6	500	300	28	2.2	150000	38	30	79	150	365	35	65	42	DUCTILE
4stry	B7	500	300	28	2.2	150000	79	64	79	150	365	21	84	31	DUCTILE
4stry	B8	500	300	28	2.2	150000	79	64	79	150	365	21	84	36	DUCTILE
4stry	B9	500	300	28	2.2	150000	38	30	79	150	365	35	65	37	DUCTILE
4stry	B10	500	300	28	2.2	150000	38	30	79	150	365	35	65	24	DUCTILE
4stry	B11	450	300	28	2.2	135000	40	32	79	150	365	21	53	12	DUCTILE

5.4.2. Diagonal bracing building

The building has a three-dimensional model. The building has four floors with a rigid diaphragm, and columns and beams are described as rod elements.

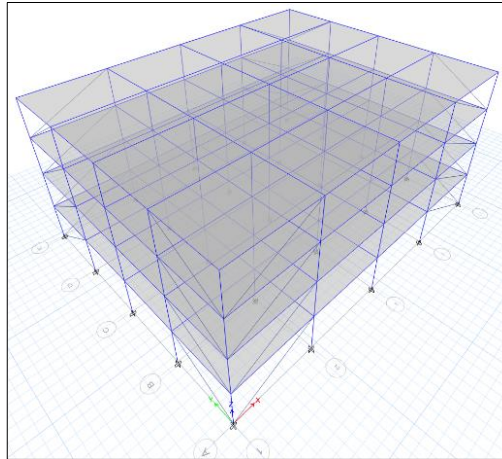


Figure 5.28: 3D Perspective View.

Table 5.13: Mass Participation Rate.

Modal Mass Participation Rate			
Mod	Period	UX	UY
1	0.73	0.8613	
2	0.69		0.8328

The torsional irregularity coefficient calculated by assuming linear elastic behavior in both earthquake directions satisfies the condition $\eta_{bi} < 1.4$. (Table 5.14, Table 5.15).

Table 5.14: X - Direction Torsional Irregularity.

Y - Direction					
Story	Load Case/Combo DIRECTION	Maximum Drift m	Average Drift m	Ratio nbi>1,4	KONTROL
Floor4	EQX	0.003749	0.0032220	1.163563004	OK
Floor3	EQX	0.005819	0.0054429	1.069099193	OK
Floor2	EQX	0.006895	0.0065740	1.048828719	OK
Floor1	EQX	0.004516	0.0043445	1.039475199	OK

Table 5.15: - Direction Torsional Irregularity.

X- Direction					
Story	Load Case/Combo DIRECTION	Maximum Drift m	Average Drift m	Ratio nbi>1,4	KONTROL
Floor4	EQY	0.003359	0.0029715	1.130405519	OK
Floor3	EQY	0.004998	0.0043935	1.137589621	OK
Floor2	EQY	0.005671	0.0049690	1.141275911	OK
Floor1	EQY	0.003488	0.0030625	1.138938776	OK

After it is comprehended that the building meets the requirements of the pushover analysis, the building is pushed up to a particular value, and a pushover curve is obtained as the base shear force-peak displacement (V_{tx} - u_{Nx}) relationship for both earthquake directions. (Figure 5.29, Figure 5.30)

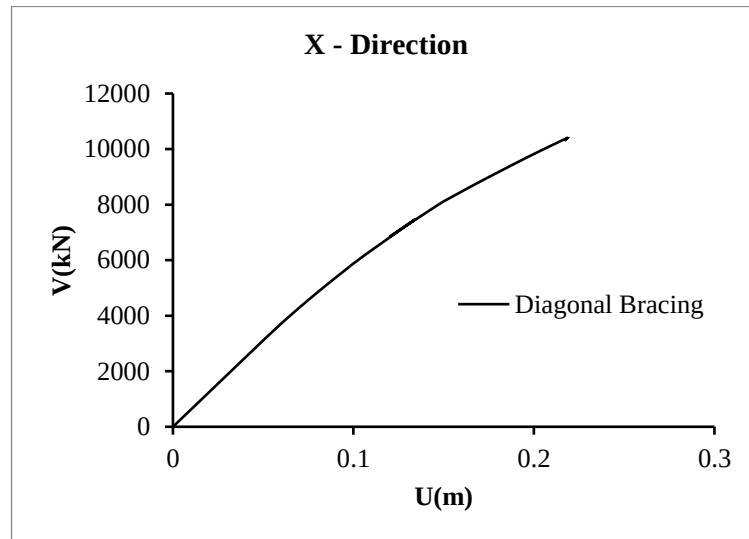


Figure 5.29: X - Direction base Shear Force-peak Displacement Graph.

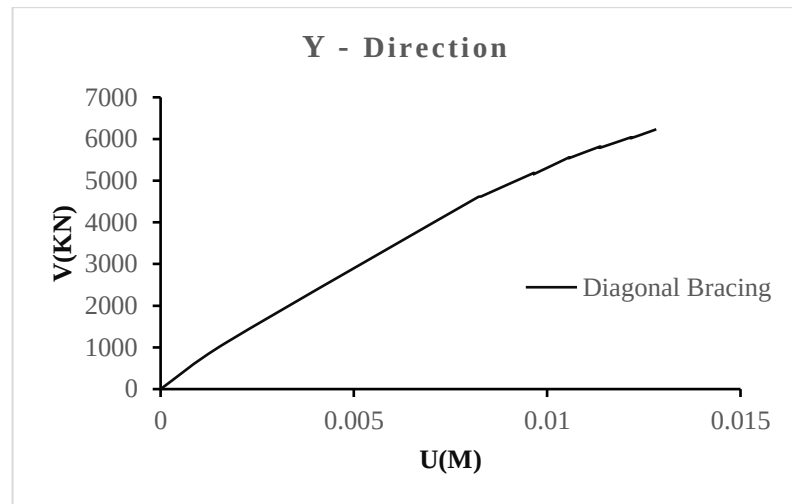


Figure 5.30: Y - Direction base Shear Force-peak Displacement Graph.

Table 5.16: Values from the Modal Analysis.

Direction	X	Y
Γ_{N1}	21	24
Φ_{N1}	3.50E-02	3.10E-02
$\sum M(t)$	5542	5542
Modal mass	0.86	0.83
M_1	2512	2506

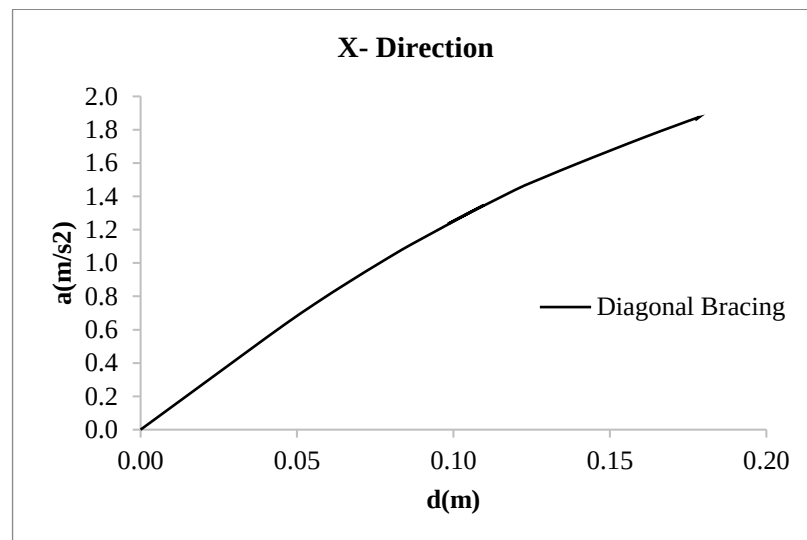


Figure 5.31: X-direction Modal Capacitance Diagram.

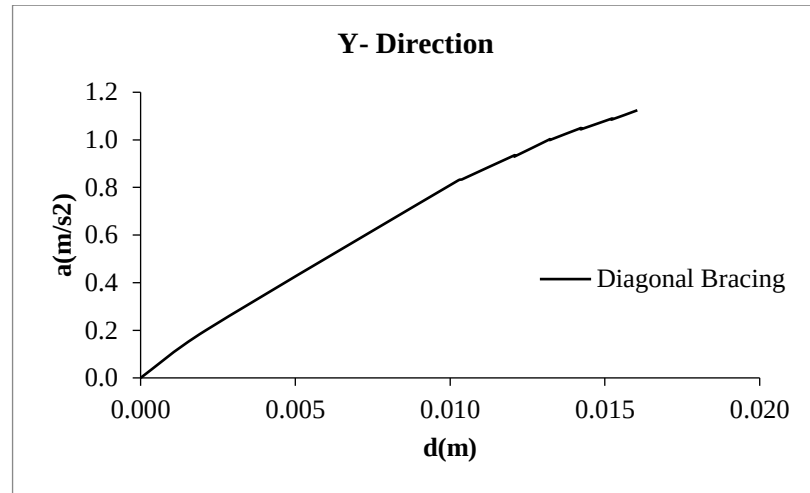


Figure 5.32: Y-direction Modal Capacitance Diagram.

The generated horizontal elastic design spectrum and the modal capacity diagram are superimposed, and the linear elastic spectral displacement value of the building is found. These elastic displacement demands were found using the Sae-Sde (Sae-Sde curve) values corresponding to the period of the building. It is 11 cm for the X direction and 7.0 cm for the Y direction. (Figure 5.33, Figure 5.34)

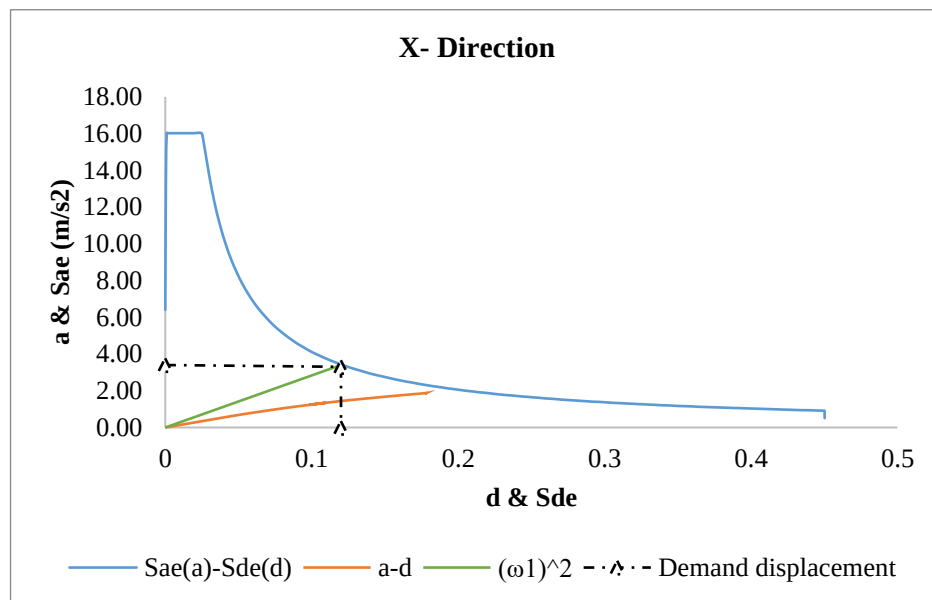


Figure 5.33: X - Direction Modal Displacement Request.

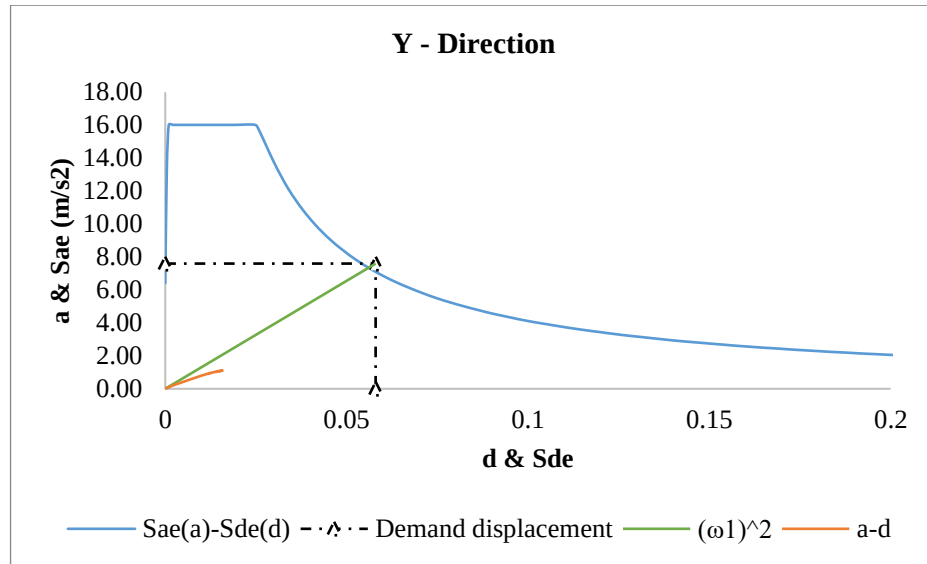


Figure 5.34: Y - Direction Modal Displacement Request.

The linear elastic spectral displacement value of the building was converted into nonlinear spectral displacement value with the help of the spectral displacement ratio CR. The period of the building is 0.73 s and 0.69 s in the X and Y directions, respectively. Since these values are greater than the TB value, which is the corner period found depending on the ground where the building is located, the CR value is taken as 1 in accordance with the equal displacement rule [18].

The nonlinear spectral displacement value of the building was calculated. The demand displacement value of the building is calculated by multiplying this value by the amplitude of the constant mode shape and the modal contribution factor, which did not change at all during the pushover calculation. These values were calculated as 20 cm for the X direction and 2.5 cm for the Y direction. (Table 5.17)

Table 5.17: X-Y Direction Nonlinear Demand Displacement Values.

X Direction			Y Direction		
$T_x > T_B$			$T_y > T_B$		
a	0.2	m/s ²	a	1.2	m/s ²
d	0.18	m	d	0.12	m
w	22.51	--	w	19.46	--
w	2.51	rad/s	w	3.21	rad/s
T	0.73	s	T	0.69	s
Sae	3.3	m/s ²	Sae	4.5	m/s ²
Sde	0.12	m	Sde	0.075	m
CR	1	--	CR	1	--
Sdi	0.12	m	Sdi	0.07	m
di	0.20	m	di	0.025	m

Table 5.18: Damage Values in the X- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0035	0.003	0.024	0.002596	0.0025	0.008	0	1.49E-04	4.47E-04	BH	IH
4stry	C5	C-60/35	0.004	0.032	0.0047	0.003	0.024	0.003508	0.0025	0.008	0	2.54E-04	4.64E-04	BH	BH
4stry	C6	C-60/35	0.004	0.032	0.0035	0.003	0.024	0.002596	0.0025	0.008	0	8.03E-04	3.62E-03	BH	BH
4stry	C7	C-60/35	0.004	0.032	0.0054	0.003	0.024	0.004069	0.0025	0.008	0	3.00E-04	3.57E-04	IH	BH
4stry	C2	C-60/35	0.004	0.032	0.0041	0.003	0.024	0.003087	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.002806	0.0025	0.008	0	1.15E-03	0	BH	SH
4stry	C16	C-60/30	0.004	0.032	0.0037	0.003	0.024	0.002806	0.0025	0.008	0	9.86E-04	1.37E-03	IH	BH
4stry	C19	C-60/30	0.004	0.032	0.0041	0.003	0.024	0.003087	0.0025	0.008	0	0	3.14E-04	SH	IH
4stry	C22	C-60/30	0.004	0.032	0.0047	0.003	0.024	0.003508	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0043	0.003	0.024	0.003227	0.0025	0.008	0	0	3.72E-04	SH	BH

Table 5.19: Damage Values in the Y- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0034	0.003	0.024	0.002581	0.0025	0.008	0	1.49E-04	4.47E-04	BH	IH
4stry	C5	C-60/35	0.004	0.032	0.0047	0.003	0.024	0.003488	0.0025	0.008	0	2.54E-04	4.64E-04	BH	BH
4stry	C6	C-60/35	0.004	0.032	0.0034	0.003	0.024	0.002581	0.0025	0.008	0	8.03E-04	0	BH	SH
4stry	C7	C-60/35	0.004	0.032	0.0054	0.003	0.024	0.004046	0.0025	0.008	0	3.00E-04	3.57E-04	IH	BH
4stry	C2	C-60/35	0.004	0.032	0.0041	0.003	0.024	0.003069	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.00279	0.0025	0.008	0	1.15E-03	1.61E-03	BH	BH
4stry	C16	C-60/30	0.004	0.032	0.0037	0.003	0.024	0.00279	0.0025	0.008	0	9.86E-04	1.37E-03	BH	BH
4stry	C19	C-60/30	0.004	0.032	0.0041	0.003	0.024	0.003069	0.0025	0.008	0	4.13E-04	3.14E-04	BH	IH
4stry	C22	C-60/30	0.004	0.032	0.0047	0.003	0.024	0.003488	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0043	0.003	0.024	0.003209	0.0025	0.008	0	3.21E-04	3.72E-04	BH	BH

The shear force corresponding to brittle fracture in columns was compared with the capacities of these load-bearing elements and it was checked whether the elements were “ductile” or “brittle”. The shear force is determined according to the calculation in TS500 [9]. (Table 5.20, Table 5.21)

Table 5.20: X - Direction Shear Control on Columns.

X - Direction																	
Column Information				Concrete Contribution							Stirrup Contribution				Check		Fracture Type
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{cm} (MPa)	γ	Nd (N)	A _c (mm ²)	V _{cr} (kN)	V _c (kN)	A _{sw} (mm ²)	s (mm)	f _{ywd} (MPa)	V _w (kN)	V _r (kN)	V _d (kN)	
4stry	C1	600	350	28	2.2	0.07	147871.061	210000	142	110	79	150	365	67	164	48.919	DUCTILE
4stry	C5	600	350	28	2.2	0.07	280773.831	210000	142	110	79	150	365	67	164	47.073	DUCTILE
4stry	C6	600	350	28	2.2	0.07	278653.7	210000	142	110	79	150	365	67	164	35.074	DUCTILE
4stry	C7	600	350	28	2.2	0.07	280773.831	210000	142	110	79	150	365	67	164	29.536	DUCTILE
4stry	C2	600	350	28	2.2	0.07	147871.061	210000	142	110	79	150	365	67	164	12.922	DUCTILE
4stry	C13	600	350	28	2.2	0.07	270.439	210000	142	110	79	150	365	67	164	19.383	DUCTILE
4stry	C16	600	300	28	2.2	0.07	50404.107	180000	122	94	79	150	365	57	138	27.69	DUCTILE
4stry	C19	600	300	28	2.2	0.07	500.266	180000	122	94	79	150	365	57	138	35.074	DUCTILE
4stry	C22	600	300	28	2.2	0.07	503.958	180000	122	94	79	150	365	57	138	29.536	DUCTILE
4stry	C25	600	300	28	2.2	0.07	270.439	180000	122	94	79	150	365	57	138	12.922	DUCTILE
4stry	C12	600	350	28	2.2	0.07	262.132	210000	142	110	79	150	365	67	164	19.383	DUCTILE
4stry	C15	600	300	28	2.2	0.07	209.521	180000	122	94	79	150	365	57	138	27.69	DUCTILE

Table 5.21: Y - Direction Shear Control on Columns.

Y - Direction																	
Column Information				Concrete Contribution							Stirrup Contribution				Check		Fracture Type
Story	Column	bW (mm)	d (mm)	fcm (MPa)	fctm (MPa)	γ	Nd (N)	AC (mm2)	Vcr (kN)	Vc (kN)	ASW (mm2)	s (mm)	fywd (MPa)	VW (kN)	Vr (kN)	Vd (kN)	
4stry	C1	600	350	28	2.2	0.07	145788.37	210000	142	110	79	150	365	67	164	48.23	DUCTILE
4stry	C5	600	350	28	2.2	0.07	276819.27	210000	142	110	79	150	365	67	164	46.41	DUCTILE
4stry	C6	600	350	28	2.2	0.07	274729	210000	142	110	79	150	365	67	164	34.58	DUCTILE
4stry	C7	600	350	28	2.2	0.07	276819.27	210000	142	110	79	150	365	67	164	29.12	DUCTILE
4stry	C2	600	350	28	2.2	0.07	145788.37	210000	142	110	79	150	365	67	164	12.74	DUCTILE
4stry	C13	600	350	28	2.2	0.07	266.63	210000	142	110	79	150	365	67	164	19.11	DUCTILE
4stry	C16	600	300	28	2.2	0.07	49694.19	180000	122	94	79	150	365	57	138	27.3	DUCTILE
4stry	C19	600	300	28	2.2	0.07	493.22	180000	122	94	79	150	365	57	138	34.58	DUCTILE
4stry	C22	600	300	28	2.2	0.07	496.86	180000	122	94	79	150	365	57	138	29.12	DUCTILE
4stry	C25	600	300	28	2.2	0.07	266.63	180000	122	94	79	150	365	57	138	12.74	DUCTILE
4stry	C12	600	350	28	2.2	0.07	258.44	210000	142	110	79	150	365	67	164	19.11	DUCTILE
4stry	C15	600	300	28	2.2	0.07	206.57	180000	122	94	79	150	365	57	138	27.3	DUCTILE

5.4.3. Inverted bracing building

The building has a three-dimensional model. The building has four floors with a rigid diaphragm, and columns and beams are described as rod elements.

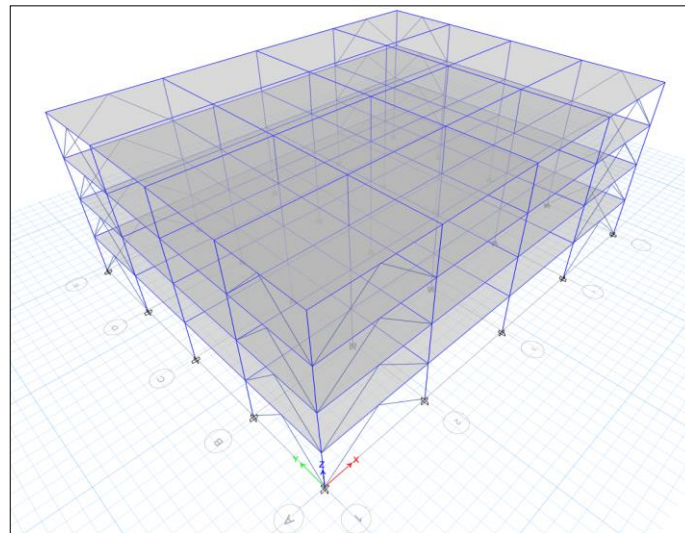


Figure 5.35: 3D Perspective View.

Table 5.22: Mass Participation Rate.

Modal Mass Participation Rate			
Mod	Period	UX	UY
1	0.31	0.8434	
2	0.28		0.7978

The torsional irregularity coefficient calculated by assuming linear elastic behavior in both earthquake directions satisfies the condition $\eta_{bi} < 1.4$. (Table 5.23, Table 5.24)

Table 5.23: X - Direction Torsional Irregularity.

Y - Direction					
Story	Load Case/Combo	Maximum Drift	Average Drift	Ratio	KONTROL
	DIRECTION	m	m	$\eta_{bi} > 1,4$	
Floor4	EQX	0.003853	0.0034582	1.114163438	OK
Floor3	EQX	0.005829	0.0055234	1.05532824	OK
Floor2	EQX	0.006894	0.0065740	1.048676605	OK
Floor1	EQX	0.004342	0.0043215	1.004743723	OK

Table 5.24: Y - Direction Torsional Irregularity.

X- Direction					
Story	Load Case/Combo	Maximum Drift	Average Drift	Ratio	KONTROL
	DIRECTION	m	m	$\eta_{bi} > 1,4$	
Floor4	EQY	0.003269	0.0029222	1.118677709	OK
Floor3	EQY	0.004908	0.00485501	1.010914499	OK
Floor2	EQY	0.005581	0.0049995	1.116311631	OK
Floor1	EQY	0.003398	0.003077	1.104322392	OK

After it is comprehended that the building meets the requirements of the pushover analysis, the building is pushed up to a particular value, and a pushover curve is obtained as the base shear force-peak displacement ($V_{tx}-u_{Nx}$) relationship for both earthquake directions. (Figure 5.36, Figure 5.37)

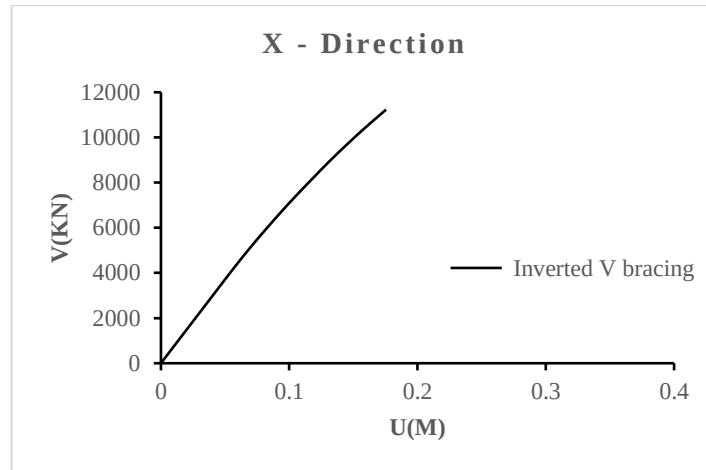


Figure 5.36: X - Direction base Shear Force-peak Displacement Graph.

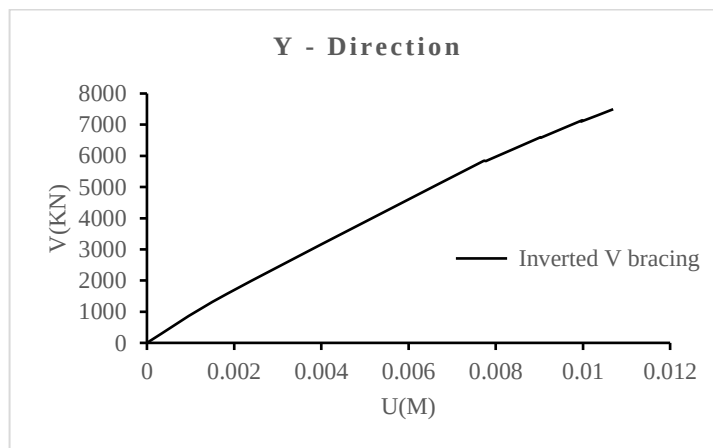


Figure 5.37: Y - Direction base Shear Force-peak Displacement Graph.

Table 5.25: Values from the Modal Analysis.

Direction	X	Y
Γ_{N1}	20.7	22.4
Φ_{N1}	3.50E-02	3.10E-02
$\sum M(t)$	5542	5542
Modal mass	0.81	0.80
M_1	2512	2506

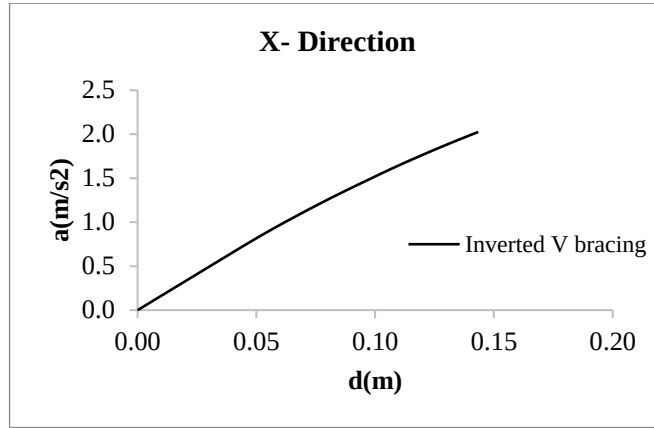


Figure 5.38: X – Direction Modal Capacitance Diagram.

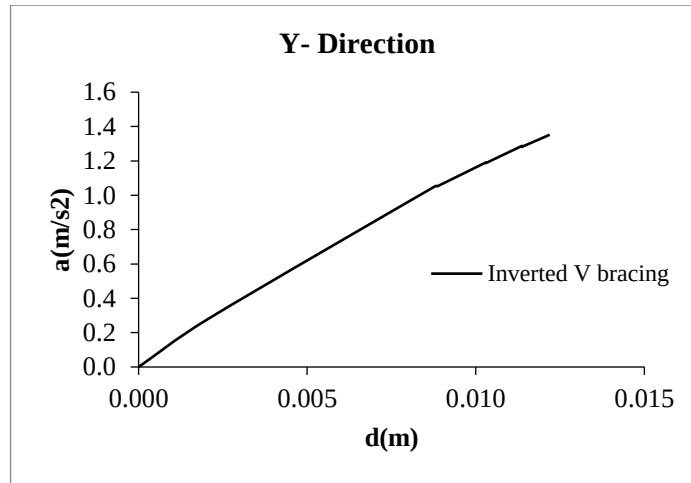


Figure 5.39: Y – Direction Modal Capacitance Diagram.

The generated horizontal elastic design spectrum and the modal capacity diagram are superimposed, and the linear elastic spectral displacement value of the building is found. These elastic displacement demands were found using the Sae-Sde (Sae-Sde curve) values corresponding to the period of the building. It is 12 cm for the X direction and 7.5 cm for the Y direction. (Figure 5.40, Figure 5.41)

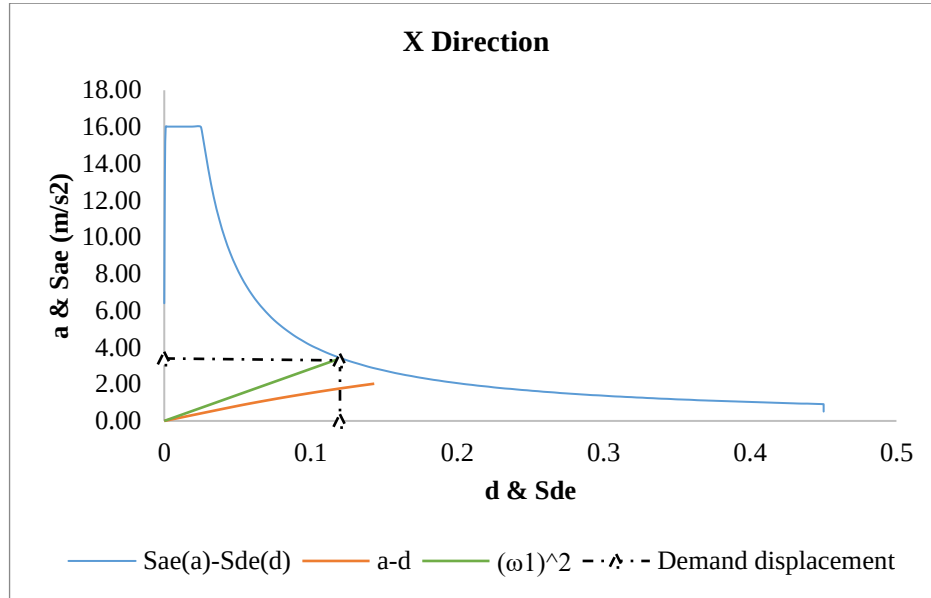


Figure 5.40: X - Direction Modal Displacement Request.

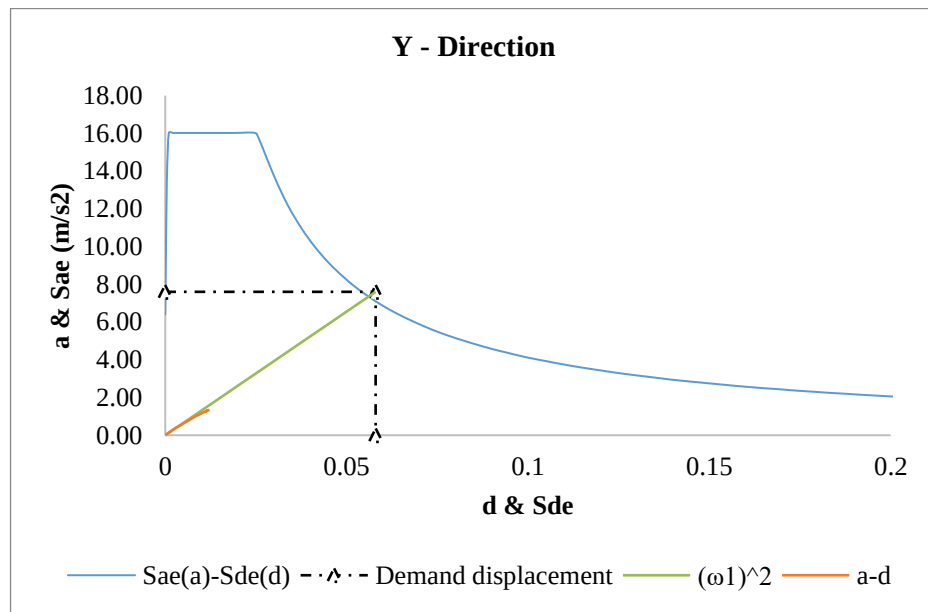


Figure 5.41: Y - Direction Modal Displacement Request.

The linear elastic spectral displacement value of the building was converted into nonlinear spectral displacement value with the help of the spectral displacement ratio CR. The period of the building is 0.31 s and 2.8 s in the X and Y directions, respectively. Since these values are greater than the TB value, which is the corner period found depending on the ground where the building is located, the CR value is taken as 1 in accordance with the equal displacement rule [18].

The nonlinear spectral displacement value of the building was calculated. The demand displacement value of the building is calculated by multiplying this value by the amplitude of the constant mode shape and the modal contribution factor, which did not change at all during the pushover calculation. These values were calculated as 20 cm for the X direction and 2.3 cm for the Y direction. (Table 5.26)

Table 5.26: X – Y Direction Nonlinear Demand Displacement Values.

X Direction			Y Direction		
$T_X > T_B$			$T_Y > T_B$		
a	0.2	m/s2	a	1.2	m/s2
d	0.18	m	d	0.12	m
w	21.22	--	w	22.32	--
w	2.31	rad/s	w	3.31	rad/s
T	0.31	s	T	0.28	s
Sae	3.3	m/s2	Sae	4.5	m/s2
Sde	0.12	m	Sde	0.075	m
CR	1	--	CR	1	--
Sdi	0.12	m	Sdi	0.07	m
di	0.20	m	di	0.023	m

Table 5.27: Damage Values in the X- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.002739	0.0025	0.008	0	1.49E-04	0	BH	SH
4stry	C5	C-60/35	0.004	0.032	0.0049	0.003	0.024	0.003701	0.0025	0.008	0	0	4.64E-04	SH	BH
4stry	C6	C-60/35	0.004	0.032	0.0037	0.003	0.024	0.002739	0.0025	0.008	0	8.03E-04	0	BH	SH
4stry	C7	C-60/35	0.004	0.032	0.0057	0.003	0.024	0.004293	0.0025	0.008	0	0	3.57E-04	SH	BH
4stry	C2	C-60/35	0.004	0.032	0.0043	0.003	0.024	0.003257	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.0039	0.003	0.024	0.002961	0.0025	0.008	0	1.15E-03	0	BH	SH
4stry	C16	C-60/30	0.004	0.032	0.0039	0.003	0.024	0.002961	0.0025	0.008	0	9.86E-04	1.37E-03	BH	BH
4stry	C19	C-60/30	0.004	0.032	0.0043	0.003	0.024	0.003257	0.0025	0.008	0	0	3.14E-04	SH	BH
4stry	C22	C-60/30	0.004	0.032	0.0049	0.003	0.024	0.003701	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0045	0.003	0.024	0.003405	0.0025	0.008	0	3.21E-04	3.72E-04	BH	BH

Table 5.28: Damage Values in the Y- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0036	0.003	0.024	0.00272	0.0025	0.008	0	1.49E-04	0	BH	SH
4stry	C5	C-60/35	0.004	0.032	0.0049	0.003	0.024	0.003675	0.0025	0.008	0	2.54E-04	4.64E-04	SH	BH
4stry	C6	C-60/35	0.004	0.032	0.0036	0.003	0.024	0.00272	0.0025	0.008	0	8.03E-04	3.62E-03	BH	BH
4stry	C7	C-60/35	0.004	0.032	0.0057	0.003	0.024	0.004263	0.0025	0.008	0	0	3.57E-04	SH	BH
4stry	C2	C-60/35	0.004	0.032	0.0043	0.003	0.024	0.003234	0.0025	0.008	0	7.32E-04	1.95E-03	BH	BH
4stry	C13	C-60/35	0.004	0.032	0.0039	0.003	0.024	0.00294	0.0025	0.008	0	1.15E-03	0	BH	SH
4stry	C22	C-60/30	0.004	0.032	0.0049	0.003	0.024	0.003675	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0045	0.003	0.024	0.003381	0.0025	0.008	0	3.21E-04	3.72E-04	BH	BH

The shear force corresponding to brittle fracture in columns was compared with the capacities of these load-bearing elements and it was checked whether the elements were “ductile” or “brittle”. The shear force is determined according to the calculation in TS500 [9]. (Table 5.29, Table 5.30)

Table 5.29: X - Direction Shear Control on Columns.

X - Direction																	
Column Information				Concrete Contribution							Stirrup Contribution				Check		Fracture
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{cm} (MPa)	γ	Nd (N)	A _c (mm ²)	V _w (kN)	V _c	A _{sw} (mm ²)	s (mm)	f _{wd} (MPa)	V _w (kN)	V _c	V _d (kN)	Type
4stry	C1	600	350	28	2.2	0.1	139380	210000	142	110	79	150	365	67	164	46	DUCTILE
4stry	C5	600	350	28	2.2	0.1	264651	210000	142	110	79	150	365	67	164	44	DUCTILE
4stry	C6	600	350	28	2.2	0.1	262653	210000	142	110	79	150	365	67	164	33	DUCTILE
4stry	C7	600	350	28	2.2	0.1	264651	210000	142	110	79	150	365	67	164	28	DUCTILE
4stry	C2	600	350	28	2.2	0.1	139380	210000	142	110	79	150	365	67	164	12	DUCTILE
4stry	C13	600	350	28	2.2	0.1	255	210000	142	110	79	150	365	67	164	18	DUCTILE
4stry	C16	600	300	28	2.2	0.1	47510	180000	122	94	79	150	365	57	138	26	DUCTILE
4stry	C19	600	300	28	2.2	0.1	472	180000	122	94	79	150	365	57	138	33	DUCTILE
4stry	C22	600	300	28	2.2	0.1	475	180000	122	94	79	150	365	57	138	28	DUCTILE
4stry	C25	600	300	28	2.2	0.1	255	180000	122	94	79	150	365	57	138	12	DUCTILE
4stry	C12	600	350	28	2.2	0.1	247	210000	142	110	79	150	365	67	164	18	DUCTILE
4stry	C15	600	300	28	2.2	0.1	197	180000	122	94	79	150	365	57	138	26	DUCTILE

Table 5.30: Y - Direction Shear Control on Columns.

Y – Direction																	
Column Information				Concrete Contribution						Stirrup Contribution				Check		Fracture	
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{cm} (MPa)	γ	Nd (N)	A _c (mm ²)	V _w (kN)	V _c	A _{sw} (mm ²)	s (mm)	f _{wd} (MPa)	V _w (kN)	V _c	V _d (kN)	Type
4stry	C1	600	350	28	2.2	0.1	139380	210000	142	110	79	150	365	67	164	46	DUCTILE
4stry	C5	600	350	28	2.2	0.1	264651	210000	142	110	79	150	365	67	164	44	DUCTILE
4stry	C6	600	350	28	2.2	0.1	262653	210000	142	110	79	150	365	67	164	33	DUCTILE
4stry	C7	600	350	28	2.2	0.1	264651	210000	142	110	79	150	365	67	164	28	DUCTILE
4stry	C2	600	350	28	2.2	0.1	139380	210000	142	110	79	150	365	67	164	12	DUCTILE
4stry	C13	600	350	28	2.2	0.1	255	210000	142	110	79	150	365	67	164	18	DUCTILE
4stry	C16	600	300	28	2.2	0.1	47510	180000	122	94	79	150	365	57	138	26	DUCTILE
4stry	C19	600	300	28	2.2	0.1	472	180000	122	94	79	150	365	57	138	33	DUCTILE
4stry	C22	600	300	28	2.2	0.1	475	180000	122	94	79	150	365	57	138	28	DUCTILE
4stry	C25	600	300	28	2.2	0.1	255	180000	122	94	79	150	365	57	138	12	DUCTILE
4stry	C12	600	350	28	2.2	0.1	247	210000	142	110	79	150	365	67	164	18	DUCTILE
4stry	C15	600	300	28	2.2	0.1	197	180000	122	94	79	150	365	57	138	26	DUCTILE

5.4.4. X bracing building

The building has a three-dimensional model. The building has four floors with a rigid diaphragm, and columns and beams are described as rod elements.

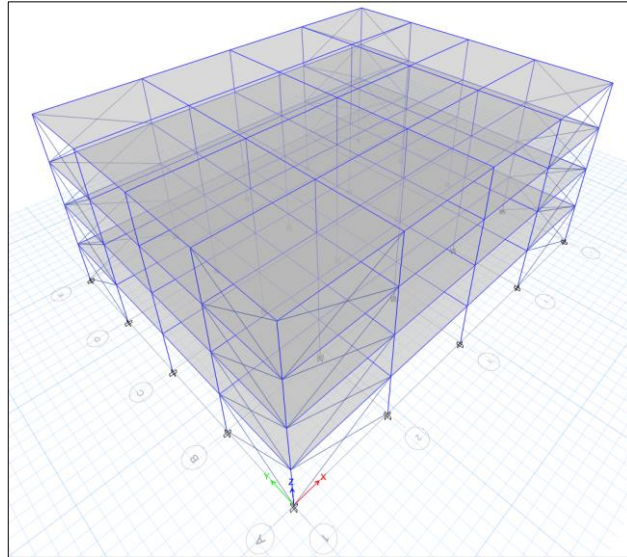


Figure 5.42: 3D Perspective View.

Table 5.31: Mass Participation Rate.

Modal Mass Participation Rate			
Mod	Period	UX	UY
1	0.59	0.8549	
2	0.56		0.8412

The torsional irregularity coefficient calculated by assuming linear elastic behavior in both earthquake directions satisfies the condition $\eta_{bi} < 1.4$. (Table 4.32, Table 4.33)

Table 5.32: X - Direction Torsional Irregularity.

Y - Direction					
Story	Load Case/Combo	Maximum Drift	Average Drift	Ratio	KONTROL
	DIRECTION	m	m	$\eta_{bi} > 1,4$	
Floor4	EQX	0.003775	0.0035770	1.055353648	OK
Floor3	EQX	0.005851	0.0055535	1.053569821	OK
Floor2	EQX	0.006916	0.0065740	1.052023121	OK
Floor1	EQX	0.004537	0.0043215	1.049866944	OK

Table 5.33: Y - Direction Torsional Irregularity.

Story	X- Direction				KONTROL
	Load Case/Combo	Maximum Drift	Average Drift	Ratio	
	DIRECTION	m	m	nbi>1,4	
Floor4	EQY	0.003248	0.0028449	1.141692151	OK
Floor3	EQY	0.004887	0.0043834	1.114887986	OK
Floor2	EQY	0.00556	0.0049480	1.123686338	OK
Floor1	EQY	0.003377	0.0030314	1.11400673	OK

After it is comprehended that the building meets the requirements of the pushover analysis, the building is pushed up to a particular value, and a pushover curve is obtained as the base shear force-peak displacement ($V_{tx}-u_{Nx}$) relationship for both earthquake directions. (Figure 5.43, Figure 5.44)

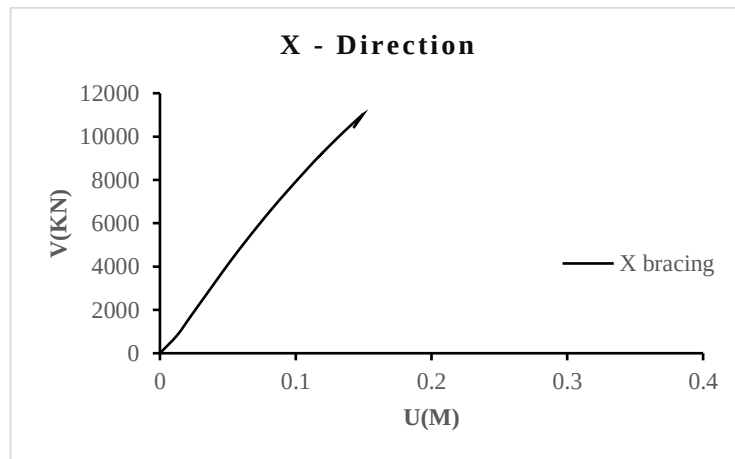


Figure 5.43: X - Direction base Shear Force-peak Displacement Graph.

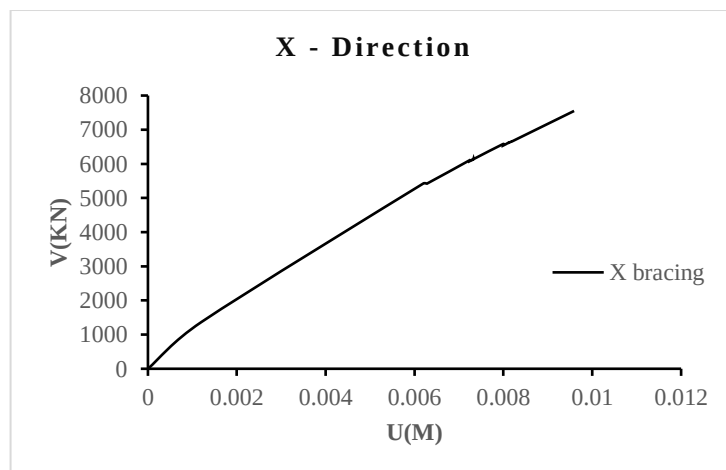


Figure 5.44: Y - Direction base Shear Force-peak Displacement Graph.

Table 5.34: Values from the Modal Analysis

Direction	X	Y
Γ_{N1}	21.32	23.05
Φ_{N1}	3.50E-02	3.10E-02
$\sum M(t)$	5542	5542
Modal mass	0.85	0.84
M_1	2512	2506

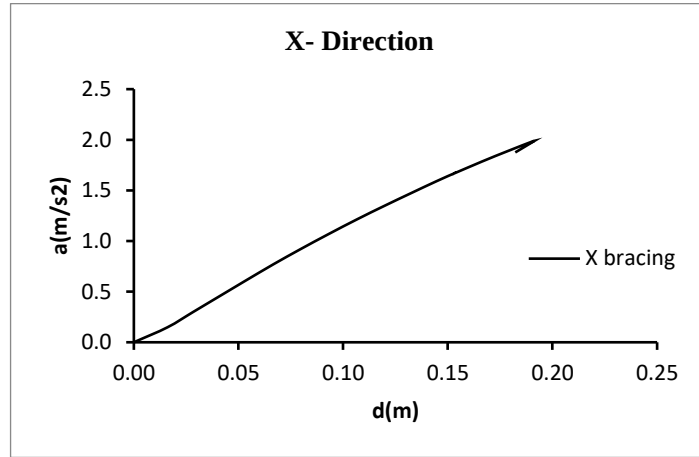


Figure 5.45: X – Direction Modal Capacitance Diagram.

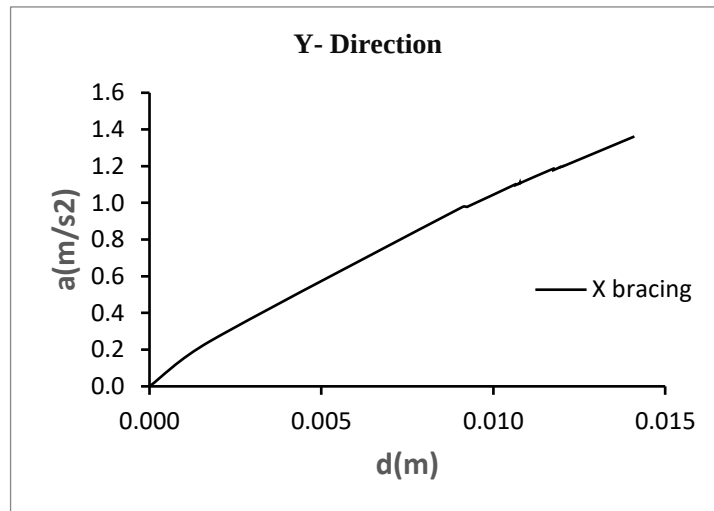


Figure 5.46: Y – Direction Modal Capacitance Diagram.

The generated horizontal elastic design spectrum and the modal capacity diagram are superimposed, and the linear elastic spectral displacement value of the building is found. These elastic displacement demands were found using the Sae-Sde

(Sae-Sde curve) values corresponding to the period of the building. It is 12 cm for the X direction and 7.5 cm for the Y direction. (Figure 5.47, Figure 5.48)

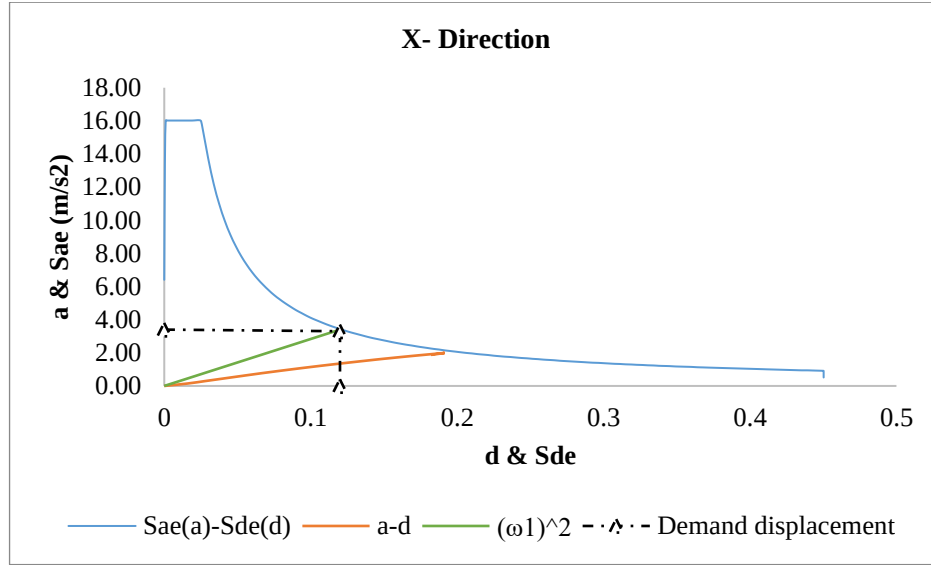


Figure 5.47: X – Direction Modal Displacement Request.

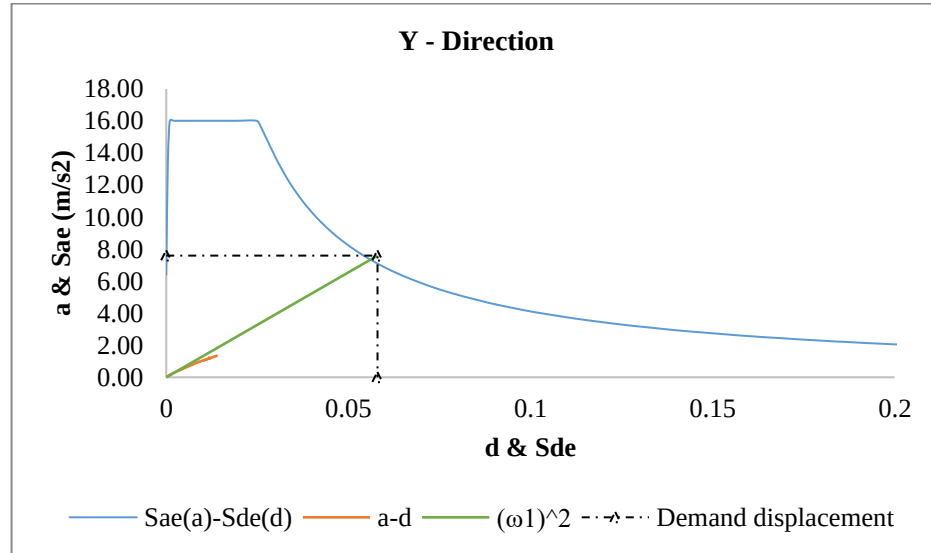


Figure 5.48: Y – Direction Modal Displacement Request.

The linear elastic spectral displacement value of the building was converted into nonlinear spectral displacement value with the help of the spectral displacement ratio CR. The period of the building is 1.20 s and 1.10 s in the X and Y directions, respectively. Since these values are greater than the TB value, which is the corner period found depending on the ground where the building is located, the CR value is taken as 1 in accordance with the equal displacement rule [18].

The nonlinear spectral displacement value of the building was calculated. The demand displacement value of the building is calculated by multiplying this value by the amplitude of the constant mode shape and the modal contribution factor, which did not change at all during the pushover calculation. These values were calculated as 20 cm for the X direction and 2.3 cm for the Y direction. (Table 4.35)

Table 5.35: X- Y Direction Nonlinear Demand Displacement Values.

X Direction			Y Direction		
$T_X > T_B$			$T_Y > T_B$		
a	0.2	m/s2	a	1.2	m/s2
d	0.18	m	d	0.12	m
w	21.22	--	w	22.32	--
w	2.31	rad/s	w	3.31	rad/s
T	0.59	s	T	0.56	s
Sae	3.3	m/s2	Sae	4.5	m/s2
Sde	0.12	m	Sde	0.075	m
CR	1	--	CR	1	--
Sdi	0.12	m	Sdi	0.07	m
di	0.20	m	di	0.025	m

Table 5.36: Damage Values in the X- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0046	0.003	0.024	0.003413	0.0025	0.008	0	0	0	SH	SH
4stry	C5	C-60/35	0.004	0.032	0.0062	0.003	0.024	0.004613	0.0025	0.008	0	0	4.64E-04	SH	BH
4stry	C6	C-60/35	0.004	0.032	0.0046	0.003	0.024	0.003413	0.0025	0.008	0	8.03E-04	0	BH	SH
4stry	C7	C-60/35	0.004	0.032	0.0071	0.003	0.024	0.005351	0.0025	0.008	0	0	3.57E-04	SH	BH
4stry	C2	C-60/35	0.004	0.032	0.0054	0.003	0.024	0.004059	0.0025	0.008	0	7.32E-04	1.95E-03	SH	BH
4stry	C13	C-60/35	0.004	0.032	0.0049	0.003	0.024	0.00369	0.0025	0.008	0	1.15E-03	1.61E-03	BH	SH
4stry	C16	C-60/30	0.004	0.032	0.0049	0.003	0.024	0.00369	0.0025	0.008	0	0	1.37E-03	SH	BH
4stry	C19	C-60/30	0.004	0.032	0.0054	0.003	0.024	0.004059	0.0025	0.008	0	4.13E-04	3.14E-04	BH	SH
4stry	C22	C-60/30	0.004	0.032	0.0062	0.003	0.024	0.004613	0.0025	0.008	0	1.70E-04	0	BH	SH
4stry	C25	C-60/35	0.004	0.032	0.0057	0.003	0.024	0.004244	0.0025	0.008	0	0	3.72E-04	SH	BH

Table 5.37: Damage Values in the Y- Direction on the Columns.

Story	Column	Section	$\epsilon_c^{(G0)}$	$\epsilon_s^{(G0)}$	$\theta_p^{(G0)}$	$\epsilon_c^{(KH)}$	$\epsilon_s^{(KH)}$	$\theta_p^{(KH)}$	$\epsilon_c^{(SH)}$	$\epsilon_s^{(SH)}$	$\theta_p^{(SH)}$	θ_{Lower} hinges	θ_{uppr} hinges	Damage status	Damage status
4stry	C1	C-60/35	0.004	0.032	0.0048	0.003	0.024	0.003608	0.0025	0.008	0	0	4.47E-04	SH	BH
4stry	C5	C-60/35	0.004	0.032	0.0065	0.003	0.024	0.004875	0.0025	0.008	0	0	4.64E-04	SH	BH
4stry	C6	C-60/35	0.004	0.032	0.0048	0.003	0.024	0.003608	0.0025	0.008	0	8.03E-04	3.62E-03	BH	BH
4stry	C7	C-60/35	0.004	0.032	0.0075	0.003	0.024	0.005655	0.0025	0.008	0	3.00E-04	0	BH	SH
4stry	C2	C-60/35	0.004	0.032	0.0057	0.003	0.024	0.00429	0.0025	0.008	0	0	SH	SH	BH
4stry	C13	C-60/35	0.004	0.032	0.0052	0.003	0.024	0.0039	0.0025	0.008	0	0	1.61E-03	SH	BH
4stry	C22	C-60/30	0.004	0.032	0.0065	0.003	0.024	0.004875	0.0025	0.008	0	1.70E-04	2.01E-04	BH	BH
4stry	C25	C-60/35	0.004	0.032	0.0060	0.003	0.024	0.004485	0.0025	0.008	0	0	0	SH	SH

The shear force corresponding to brittle fracture in columns was compared with the capacities of these load-bearing elements and it was checked whether the elements were “ductile” or “brittle”. The shear force is determined according to the calculation in TS500 [9]. (Table 5.38, Table 5.39)

Table 5.38: X - Direction Shear Control on Columns.

X – Direction																	
Column Information			Concrete Contribution								Stirrup Contribution				Check		Fracture Type
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{tm} (MPa)	γ	Nd (N)	A _c (mm ²)	V _{cr} (kN)	V _c (kN)	A _{sw} (mm ²)	s (mm)	f _{wd} (MPa)	V _{sw} (kN)	V _r (kN)	V _d (kN)	
4stry	C1	600	350	28	2.2	0.07	143706	210000	142	110	79	150	365	67	164	29	DUCTILE
4stry	C5	600	350	28	2.2	0.07	272865	210000	142	110	79	150	365	67	164	13	DUCTILE
4stry	C6	600	350	28	2.2	0.07	270804	210000	142	110	79	150	365	67	164	19	DUCTILE
4stry	C7	600	350	28	2.2	0.07	272865	210000	142	110	79	150	365	67	164	27	DUCTILE
4stry	C2	600	350	28	2.2	0.07	143706	210000	142	110	79	150	365	67	164	19	DUCTILE
4stry	C13	600	350	28	2.2	0.07	263	210000	142	110	79	150	365	67	164	48	DUCTILE
4stry	C16	600	300	28	2.2	0.07	48984	180000	122	94	79	150	365	57	138	46	DUCTILE
4stry	C19	600	300	28	2.2	0.07	486	180000	122	94	79	150	365	57	138	34	DUCTILE
4stry	C22	600	300	28	2.2	0.07	490	180000	122	94	79	150	365	57	138	29	DUCTILE
4stry	C25	600	300	28	2.2	0.07	263	180000	122	94	79	150	365	57	138	13	DUCTILE
4stry	C12	600	350	28	2.2	0.07	255	210000	142	110	79	150	365	67	164	19	DUCTILE
4stry	C15	600	300	28	2.2	0.07	204	180000	122	94	79	150	365	57	138	27	DUCTILE

Table 5.39: Y - Direction Shear Control on Columns.

Y– Direction																		
Column Information				Concrete Contribution								Stirrup Contribution				Check		Fracture Type
Story	Column	b _w (mm)	d (mm)	f _{cm} (MPa)	f _{tm} (MPa)	γ	Nd (N)	A _c (mm ²)	V _{cr} (kN)	V _c (kN)	A _{sw} (mm ²)	s (mm)	f _{wd} (MPa)	V _{sw} (kN)	V _r (kN)	V _d (kN)		
4stry	C1	600	350	28	2.2	0.07	149804	210000	142	110	79	150	365	67	164	30	DUCTILE	
4stry	C5	600	350	28	2.2	0.07	278891	210000	142	110	79	150	365	67	164	13	DUCTILE	
4stry	C6	600	350	28	2.2	0.07	279076	210000	142	110	79	150	365	67	164	19	DUCTILE	
4stry	C7	600	350	28	2.2	0.07	280664	210000	142	110	79	150	365	67	164	28	DUCTILE	
4stry	C2	600	350	28	2.2	0.07	227249	210000	142	110	79	150	365	67	164	19	DUCTILE	
4stry	C13	600	350	28	2.2	0.07	250	210000	142	110	79	150	365	67	164	49	DUCTILE	
4stry	C16	600	300	28	2.2	0.07	49176	180000	122	94	79	150	365	57	138	47	DUCTILE	
4stry	C19	600	300	28	2.2	0.07	421	180000	122	94	79	150	365	57	138	35	DUCTILE	
4stry	C22	600	300	28	2.2	0.07	421	180000	122	94	79	150	365	57	138	30	DUCTILE	
4stry	C25	600	300	28	2.2	0.07	219	180000	122	94	79	150	365	57	138	13	DUCTILE	
4stry	C12	600	350	28	2.2	0.07	229	210000	142	110	79	150	365	67	164	19	DUCTILE	
4stry	C15	600	300	28	2.2	0.07	252	180000	122	94	79	150	365	57	138	28	DUCTILE	

5.5. Comparison of existing and strengthened buildings

5.5.1. Fundamental vibration period

Fundamental vibration periods for the first two modes for the existing and strengthened buildings are compared. The fundamental periods of vibration decreased as seen in Figure 5.49, Figure 5.50.

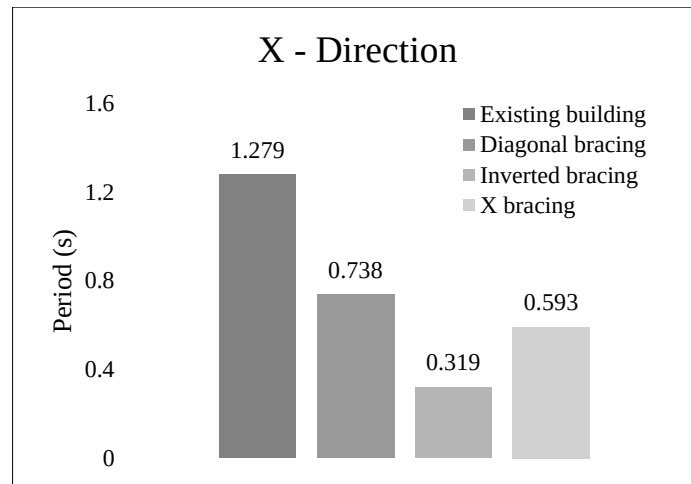


Figure 5.49: Fundamental Vibration Periods.

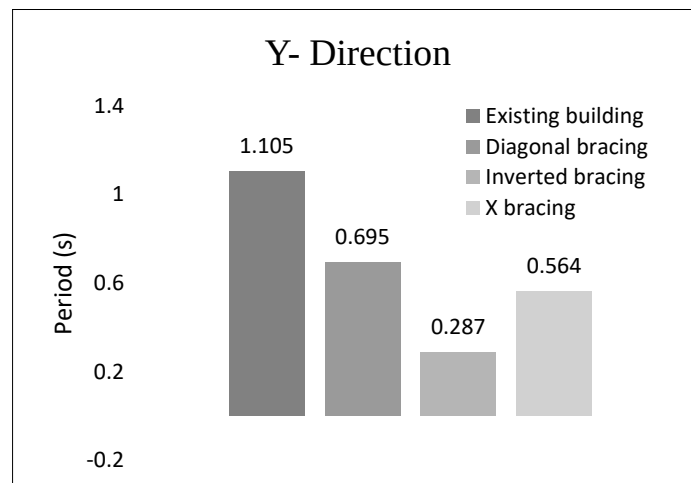


Figure 5.50: Fundamental Vibration Periods.

5.5.2. Capacity Curve of the Structures

The pushover analysis is applied to all models by using lateral triangular pattern. The outcome result has remained used to showed and compered through the capacity curve to inform and report the performance deformation criteria; also, the comparison of the lateral displacement capacity has been approved and presented in the other curves. This curve below defines the effect and the response performance of the different types of steel bracing systems when the lateral loads applied to the structure. This result proves that the inserting steel bracing in the structure is dramatically increase the strength of the structure. The base shear results for various types of bracing systems and unbraced frames are illustrated in Figure 5.51, Figure 5.52.

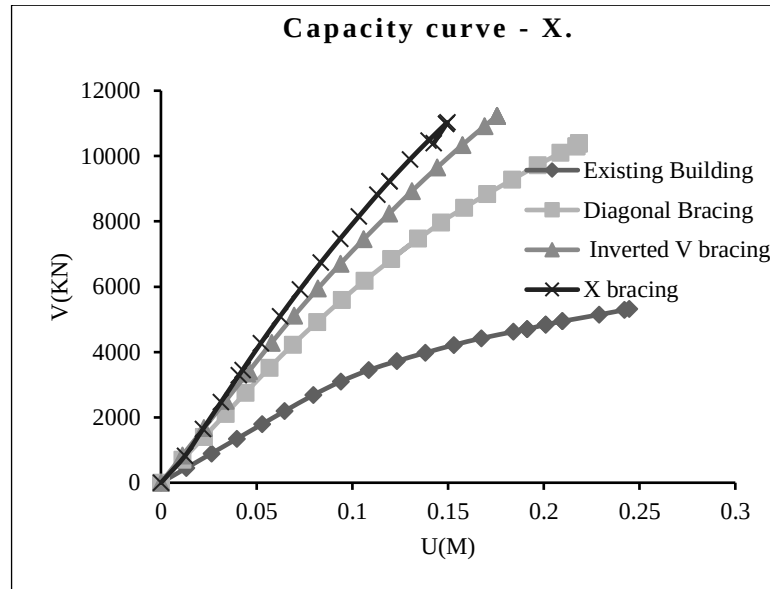


Figure 5.51: Capacity Curve in X Direction.

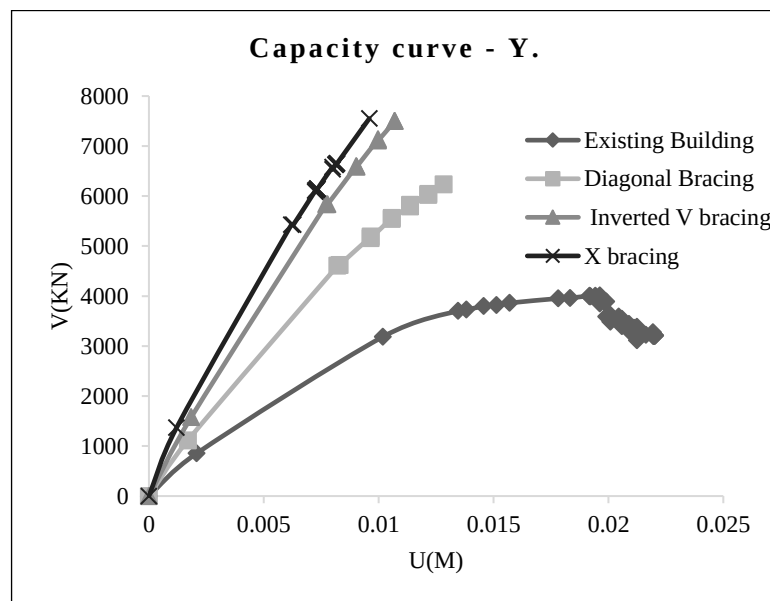


Figure 5.52: Capacity Curve in Y Direction.

It can be noticed that the base shear in the unbraced frames was less than that of the braced frames. The increment in base shear in braced frames has little influence on the columns because most of the shear forces were supported by the braces.

5.5.3. Story Drift

This study proves when the height of the structure increases, also the drift story increase after that then decreases. This study proves that the inserting steel bracing is in the structure, it dramatically reduces the inner story drift, therefore, the result of the different types of steel bracing systems that tested during this study had different effects, so x-bracing system becomes when of the most effective for the reduction of the story drift which illustrated in Figure 5.53-Figure 5.54.

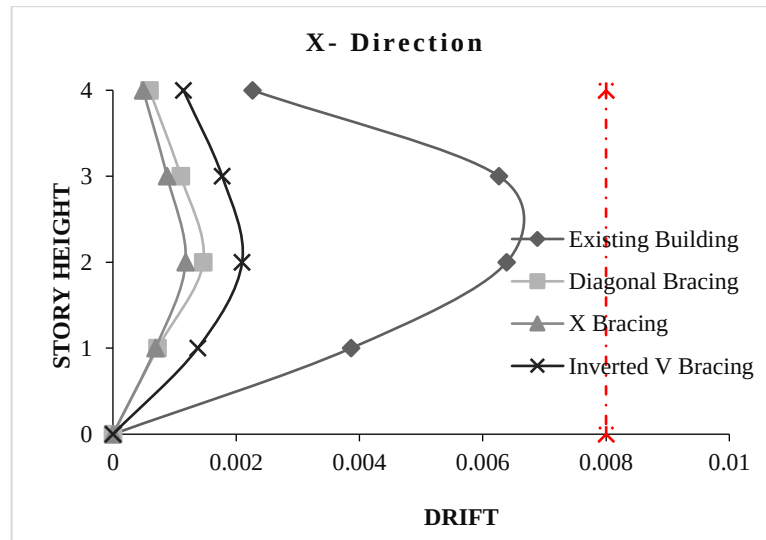


Figure 5.53: Story Drift in X Direction.

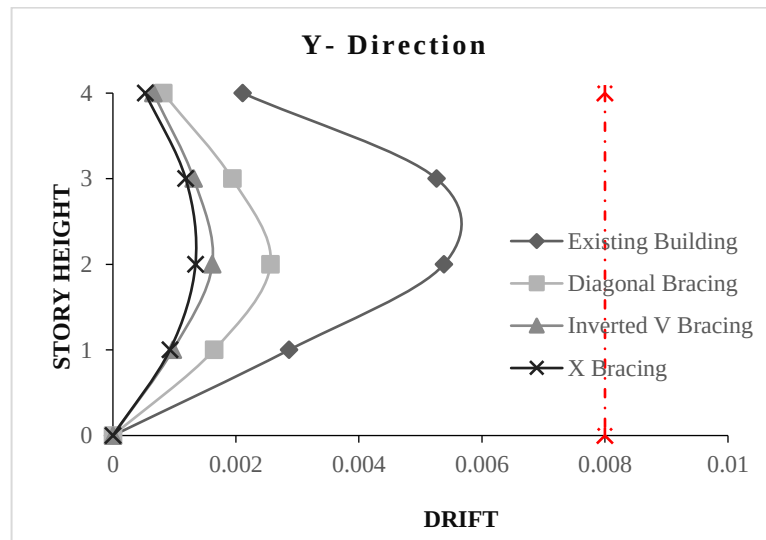


Figure 5.54: Story Drift in Y Direction.

5.5.4. Story displacement

This result is based on the comparison of the horizontal load-displacement in the x-z direction in the frames compared during this research. This investigation proves that the top story floors have greater displacement than the lower stories; also, this study proves that the uses of steel bracing in the building frame it dramatically decreases the story displacement.

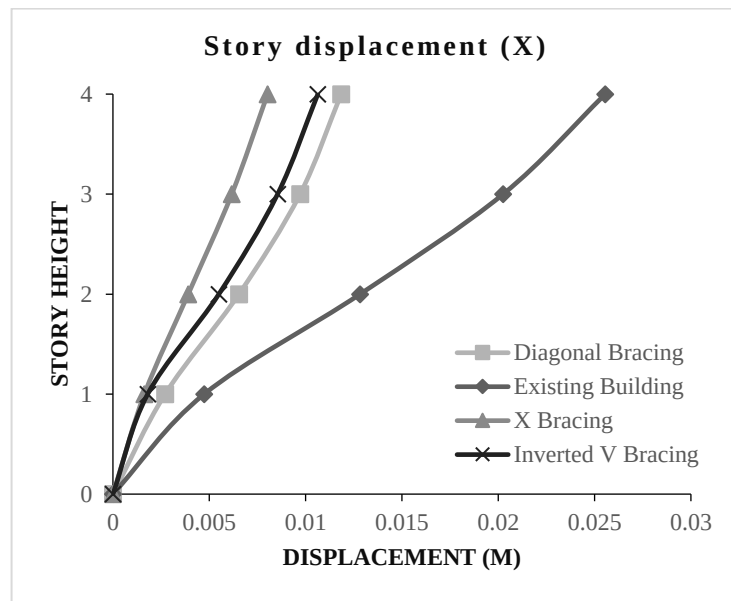


Figure 5.55: Story Displacement in X Direction.

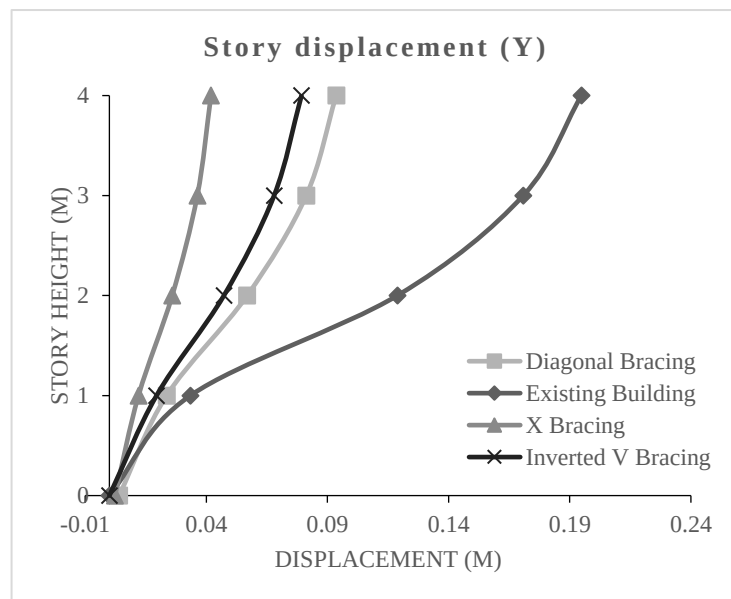


Figure 5.56: Story Displacement in Y Direction.

During this research, the different types of CBFs are compared, considering unbraced, ones to know the most resistible system during the seismic activities. So, x-bracing becomes the most resistible and the good reduction for the story displacement in the following points, first, steel bracing increases the strength of the structure, secondly increasing the rigidity and the strength of the joints decrease the lateral displacement of the structure so x- bracing members are all connecting directly to the joints compared to the other steel bracing system members used in this study.

6. CONCLUSION

This study investigated the performance analysis of an existing RC building strengthen with different types of steel braces as an effective method in the form of retrofitting. The bracing systems are considered as X-braced frames, diagonal braced frames, inverted V braced frames. The structure has also been retrofitted using bracing techniques to examine the efficacy of different strengthening methods. Throughout the model, the pushover mechanism is performed. The results of these investigations lead us to the following conclusions:

- Overall, the bracing configuration played an important role in the initial stiffness, inelastic deformation distributions, and the overstrength available in the dual structures.
- The outcomes revealed that the addition of bracing improved the seismic resistance of frames significantly. The results showed that the bracing elements were particularly effective in reducing drifts since they reduced inter-story drifts concerning unbraced frame.
- CBFs are utilized in the structural system for seismic retrofitting, the objective displacement is significantly reduced, and story drift are reduced as an effect.
- During this study, X bracing system was the most irresistible compared with the others, which increases the strength of the structure and also, it takes part in the energy dissipation of the frame.
- This investigation demonstrates that the story drift increases when the height of the structure increases a certain point then starts to decrease as the height gets closer.
- General, the bracing went through many stages of out-of-plane inelastic buckling and tension yielding. The braces were also observed to be permanently elongated. Buckling ultimately developed in the braces' plastic hinges, and fracture of the bracing elements occurred when they were subsequently pushed under tension.
- In the X bracing, the tension brace can provide out-of-plane assist to the compression brace, although the effectiveness of the support was diminished at the zero story drift point due to the braces' permanent extension.

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BIOGRAPHY

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