

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL

**RETROFIT OF REINFORCED CONCRETE FRAMES WITH
BUCKLING-RESTRAINED BRACES**



DERYA ERASLAN

**A THESIS OF MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL
ENGINEERING PROGRAM**

ADVISOR: ASST. PROF. DR. ÜLGEN MERT

Co-ADVISOR: ASSOC. PROF. DR. ONUR ŞEKER

JULY 2025

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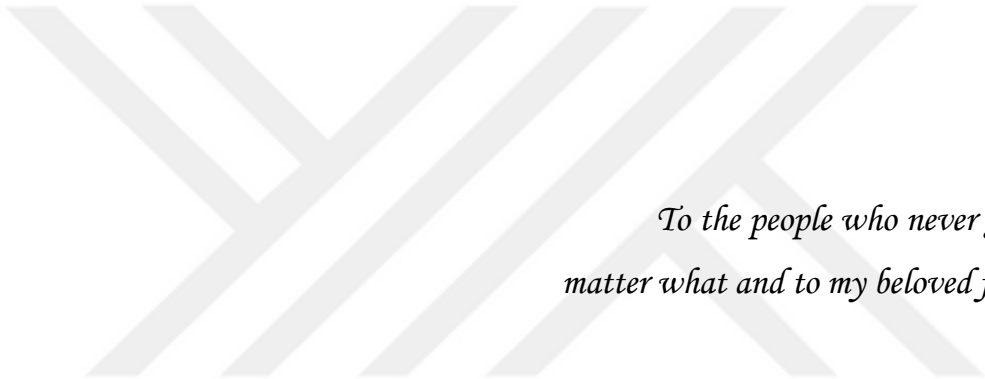
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*To the people who never give up, no
matter what and to my beloved family...*

ABSTRACT

The devastating earthquake of February 6th in our country served as a stark reminder of the vulnerabilities inherent in our built environment. The majority of structures in seismic regions like Türkiye consist of reinforced concrete (RC) systems, which are often inadequate to resist strong ground motions due to aging, poor detailing, or outdated design codes. In such a context, retrofitting has become a more practical and necessary approach than demolition and reconstruction.

Worldwide, many innovative retrofitting techniques are being implemented. BRBs (Buckling-Restrained Braces) are considered an innovative system for our country. What distinguishes this system from other retrofitting methods is its ability to be removed and reinstalled after an earthquake, allowing the building to maintain post-earthquake operability. Additionally, it significantly increases structural stiffness and enhances the energy dissipation capacity of the building. Moreover, since it does not require full coverage, it can be applied only in critical areas, offering a cost-effective solution.

This study aims to retrofit an existing seven-story reinforced concrete hospital building, located in a high seismic risk region, with a total height of 23.2 meters. The structure includes shear walls in the Y-direction that provide resistance against seismic loads. BRBs were applied to the outermost axes to ensure uninterrupted use of the building. In addition, the placement of the braces was not uniform but was optimized within the scope of this thesis

After BRB retrofitting, significant reductions in story drift ratios were observed in the first three stories. At the first story, drift was reduced from 2.4% to 1.3%; at the second story, from 2.6% to 1%; and at the third story, from 1.8% to 0.7%. Roof displacement also decreased from 0.29 m to 0.16 m, indicating a 55% reduction. No significant increase in axial loads was observed, as the placement of BRBs was optimized to minimize their impact on the columns.

According to PUSH-X results, all columns met the performance limits defined by TBDY-2018 under the DD-1 earthquake scenario. Initially, 32% of first-story columns were in collapse, 61% in advanced damage, and 7% in significant damage zones. After retrofitting, all first-story columns shifted to the limited damage zone, and all other structural elements remained within the TBDY-2018 and AISC (2016) standards. The analyses were carried out using ETABS (v21.2.1, CSI, USA).

Additionally, the effects of the Pazarcık and Elbistan earthquakes, which struck Türkiye on February 6, were discussed within the scope of the study.

Keywords: Buckling-Restrained Brace, Concrete Strengthening, Seismic Performance, Earthquake Engineering

ÖZET

6 Şubat'ta ülkemizde meydana gelen yıkıcı deprem, mevcut yapı stokumuzda var olan zayıflıkları açık bir şekilde gözler önüne sermiştir. Türkiye gibi deprem riski yüksek bölgelerdeki yapıların büyük çoğunluğu betonarme sistemlerden oluşmakta olup; bu yapılar, yaşlanma, yetersiz detaylandırma ve eski tasarım yönetmelikleri nedeniyle şiddetli yer hareketlerine karşı koymakta yetersiz kalmaktadır. Bu bağlamda, güçlendirme uygulamaları, yıkım ve yeniden inşaya kıyasla daha pratik ve gerekli bir çözüm haline gelmiştir.

Dünya genelinde birçok yenilikçi güçlendirme yöntemi uygulanmaktadır. BÖÇ'ler (Burkulması Önlenmiş Çaprazlar), ülkemiz için yenilikçi bir sistem olarak kabul edilmektedir. Bu sistemi diğer yöntemlerden ayıran en önemli özelliği, deprem sonrasında sökülüp yeniden takılabilmesi sayesinde yapının kesintisiz kullanımına izin vermesidir. Buna ek olarak, yapının rijitliğini önemli ölçüde artırmakta ve enerji sönümleme kapasitesini geliştirmektedir. Ayrıca, bu sistemin yekpare uygulanması gerekmediğinden yalnızca kritik bölgelere yerleştirilerek ekonomik bir çözüm sunmaktadır.

Bu çalışma kapsamında, yüksek sismik risk bölgesinde yer alan toplam 23.2 metre yüksekliğe sahip yedi katlı mevcut bir betonarme hastane binasının güçlendirilmesi amaçlanmıştır. Yapının Y yönünde deprem yüklerine karşı direnç sağlayan perdeler bulunmaktadır. BÖÇ'ler, yapının kesintisiz kullanımı sağlamak amacıyla dış akslara uygulanmıştır. Ayrıca, bu çapraz elemanların yerleşimi yekpare şekilde değil, bu tez kapsamında optimize edilerek belirlenmiştir.

BÖÇ ile yapılan güçlendirme sonrası, ilk üç katta kat ötelemelerinde önemli azalmalar gözlemlenmiştir. Birinci katta ötelenme %2.4'ten %1.3'e; ikinci katta %2.6'dan %1'e; üçüncü katta ise %1.8'den %0.7'ye düşmüştür. Çatı ötelenmesi ise 0.29 metreden 0.16 metreye düşerek %55 oranında azalmıştır. Kolonlara gelen eksenel yüklerde belirgin bir artış gözlenmemiştir; çünkü BÖÇ yerleşimleri, kolonlara olan etkilerini en aza indirecek şekilde optimize edilmiştir.

PUSH-X sonuçlarına göre, tüm kolonlar DD-1 deprem senaryosu altında TBDY-2018 tarafından tanımlanan performans sınırlarını sağlamıştır. Başlangıçta, birinci kat kolonlarının %32'si göçme, %61'i ileri hasar ve %7'si belirgin hasar bölgelerinde yer almaktaydı. Güçlendirme sonrasında, birinci kat kolonlarının tamamı sınırlı hasar bölgesine geçmiş, diğer tüm yapısal elemanlar ise TBDY-2018 ve AISC (2016) standartları kapsamında kalmıştır. Analizler ETABS (v21.2.1, CSI, ABD) yazılımı kullanılarak gerçekleştirilmiştir.

Ayrıca, 6 Şubat'ta Türkiye'de meydana gelen yıkıcı Pazarcık ve Elbistan depremlerinin etkileri de bu çalışma kapsamında tartışılmıştır.

Anahtar Kelimeler: Burkulması Önlenmiş Çapraz, Betonarme Güçlendirme, Sismik Performans, Deprem Mühendisliği

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LIST OF SYMBOLS AND ABBREVIATIONS

ε_s (CP)	:	Permissible unit strain limit for reinforcing steel under the collapse prevention performance level.
ε_s (LS)	:	Permissible unit strain limit for reinforcing steel under the life safety performance level.
ε_s (IO)	:	Permissible unit strain limit for reinforcing steel under the immediate occupancy performance level.
F_S	:	Site coefficient for short-period range
F_1	:	Site coefficient for 1-second period
g	:	Acceleration due to gravity ($g = 9.81 \text{ m/s}^2$)
$S_{AE}(T)$	:	Elastic spectral acceleration (m/s^2)
$S_{aeD}(T)$	:	Elastic vertical spectral acceleration
$S_{de}(T)$	:	Elastic spectral displacement
S_{D1}	:	Design spectral response acceleration at 1-second period
S_S	:	Design spectral response acceleration at short periods (S_s)
I	:	Building importance factor
n	:	Live load coefficient
$S(T)$	:	Spectrum coefficient
η_{bi}	:	Torsional irregularity coefficient
Φ_{xin}	:	Lateral mode shape component in x-direction
Φ_{yin}	:	Lateral mode shape component in y-direction
ε_s	:	Unit strain of rebar steel
ϕ_p	:	Plastic curvature
ϕ_t	:	Total curvature
ϕ_y	:	Yield curvature
θ_p	:	Plastic rotation
ρ_s	:	Ratio of the volume of longitudinal rebar
$(EI)_e$	:	Effective bending stiffness
$(EI)_0$	:	Bending stiffness
f_{cm}	:	Existing concrete strength.
f_{ctm}	:	Tensile strength of concrete
f_{yw}	:	Yield strength of steel
h	:	Height of beam
A_c	:	The gross area of the column or shear wall

ε_s (CP)	:	Permissible unit strain limit for reinforcing steel under the collapse prevention performance level.
ε_s (LS)	:	Permissible unit strain limit for reinforcing steel under the life safety performance level.
ε_s (IO)	:	Permissible unit strain limit for reinforcing steel under the immediate occupancy performance level.
ε_{su}	:	Rebar strain rate at maximum strength
ϕ_u	:	Curvature at collapse point
ω_{we}	:	Ratio of effective wrap reinforcement to total reinforcement
$\rho_{sh, min}$	:	The lower volumetric lateral rebar ratio in two horizontal directions
θ_p (CP)	:	Permissible plastic rotation limit for collapse prevention performance level (radians)
θ_p (LS)	:	Permissible plastic rotation limit for life safety performance level (radians)
θ_p (IO)	:	Permissible plastic rotation limit for immediate occupancy performance level (radians)
$A(T)$	:	Spectral acceleration coefficient
g	:	Acceleration of gravity
$S(T)$	:	Spectrum coefficient
T	:	Building vibration period (s)
T_1	:	Binanın birinci doğal titreşim periyodu
T_A, T_B	:	Spectrum characteristic periods
CP	:	Collapse prevention
IO	:	Immediate occupancy
TBDY-2018	:	Turkish seismic code 2018
BRB	:	Buckling-Restrained Braces
BKS	:	Building Use Class
DD-1	:	Design-level earthquake with 2475-year return period

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1. INTRODUCTION

The devastating earthquake of February 6th 2023 Pazarcık and Elbistan Earthquakes in Türkiye served as a stark reminder of the vulnerabilities inherent in our built environment. As a structural engineer specializing in earthquake engineering, our primary goal is to reduce the catastrophic damage caused by seismic forces. Given that the vast majority of the building stock in Türkiye consists of reinforced concrete structures, the susceptibility of these structures to earthquake damage necessitates innovative solutions [ASCE, 2017]. This essay explores the potential of Buckling-Restrained Braces [BRBs] as a method for strengthening existing structures and enhancing their resilience against earthquakes [Sabelli et al., 2003].

The prevalence of reinforced concrete buildings in Türkiye makes retrofitting these structures a more viable option compared to the impractical and often unrealistic scenario of demolishing and rebuilding them [FEMA, 2006]. While conventional steel braces can increase structural strength, they often inadvertently reduce the building's fundamental vibration period, potentially amplifying seismic loads—a critical drawback in earthquake-prone regions [Chopra, 2012].

Buckling-Restrained Braces (BRBs) offer a promising solution, presenting significant advantages over conventional bracing methods. Their capacity to undergo large cyclic strain reversals and effectively dissipate seismic energy without sacrificing strength distinguishes them significantly [Uang & Nakashima, 2004]. This results in several notable benefits:

1. **Wide applicability:** BRBs can be seamlessly integrated into steel, reinforced concrete, and composite structures, making them suitable for diverse building types [Tremblay et al., 2004].
2. **Enhanced stiffness and damping:** By integrating BRBs into structural systems, overall stiffness increases, while additional damping significantly absorbs earthquake-induced energy [Black et al., 2002].

3. **Reduced displacements:** Studies indicate that BRBs can reduce structural displacements by up to 50%, significantly decreasing the risk of structural damage [Iwata & Murai, 2006].
4. **Cost-effectiveness:** Strategic placement of BRBs optimizes costs while providing substantial seismic protection, avoiding unnecessary widespread application [Clark et al., 1999].
5. **Aesthetic appeal:** Externally mounted BRBs can even enhance the architectural aesthetics of buildings [Sabelli et al., 2003].
6. **Versatility:** Various types and models of BRBs are available, allowing solutions tailored to specific structural requirements [Takeuchi et al., 2006].

This study investigates the seismic performance of a reinforced concrete building with structural irregularities and evaluates various retrofitting techniques to improve its structural resilience. Numerical analyses were carried out using ETABS and Microsoft software, taking into account pushover and modal analyses.

Within the scope of this thesis, the seismic performance analysis of an existing hospital building located in a high seismicity region and on ZC soil class was conducted based on the DD1 earthquake ground motion. As a result of the analysis, it was determined that due to the damage observed, especially in the columns, the global lateral load-carrying capacity of the structure was insufficient, and a global strengthening, as required by the relevant seismic code, was necessary [TBDY. (2018).]. Buckling-Restrained Braces (BRBs) were chosen as the strengthening method due to their advantages. In this way, the structure can continue to operate without interruption through external strengthening.

Since the building is a hospital, BRBs were applied on the outer perimeter axes to ensure operational continuity during and after the retrofitting process. The number and placement of BRBs were determined based on optimal locations. With a focus on enhancing ductility, reducing displacement demands, and maximizing overall seismic resistance, the effects of BRB retrofitting techniques on building performance were thoroughly investigated.

1.1 Statement of Purpose

In this study, the aim is to retrofit a hospital building located in a high seismic risk region, which requires global strengthening according to the 2018 Turkish Seismic Code. An innovative retrofitting method using Buckling-Restrained Braces (BRBs), which can be considered new in Türkiye, was applied. The main goal of this approach is to ensure that the building reaches the operational performance level after an earthquake and to remain on the economic side. Unlike traditional methods, instead of strengthening all stories equally, BRBs were placed only on the outer axes in the direction where strengthening was needed. BRB placement was optimized on each floor.

The structural modeling was carried out using the ETABS software. Through pushover analysis, drifts and damage levels were identified. Reduction in story drift ratios contributed to minimizing structural damage. Additionally, it was aimed to optimize axial load distribution, and with the help of the BRB layout, potential issues due to increased axial forces in the lower columns after retrofitting were intended to be avoided.

2. BRB DEVELOPMENT

2.1. BRB History

BRBs are a ground-breaking milestone in earthquake engineering because they provide a solution to a major weakness of traditional braces: their tendency to buckle under compressive forces. This buckling often led to both local and global instabilities, causing the steel members to fail before reaching their expected performance. The origin of this innovation dates back to pioneering research conducted in Japan during the 1980s, with a foundational contribution by Atsushi Watanabe et al., presented at the Ninth World Conference on Earthquake Engineering in 1988.

Traditional steel braces were often unreliable due to early buckling during compression loading. This instability limited their ability to dissipate energy and caused asymmetric hysteresis loops. As a result, their effectiveness under seismic loads was severely constrained. The BRB concept aimed to overcome these limitations by isolating the axial force-resisting core from lateral instability. In Watanabe's design, the steel core was encased in concrete and then enclosed within a steel tube [Figure 2.1]. The concrete and additional coatings served to prevent both local and global buckling.

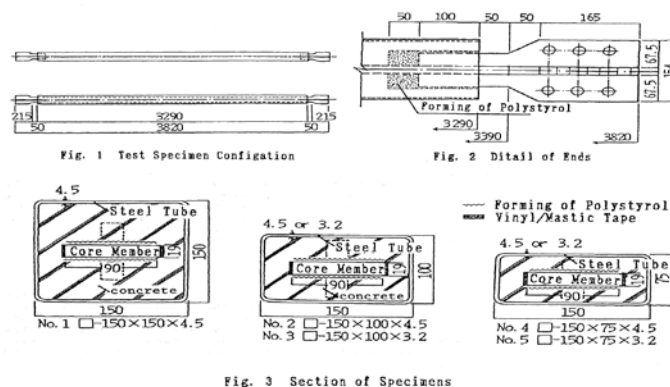


Figure 2.1: Section of specimens

A key innovation in Watanabe's work was the control of brace behaviour through the ratio of Euler buckling load (P_e) of the steel tube to the yield strength (P_y) of the steel core. In traditional braces, Euler's critical buckling load is calculated by the formula:

$$P_e = \frac{\pi^2 EI}{(KL)^2} \quad (1.1)$$

Where E is the modulus of elasticity, I is the moment of inertia, K is the effective length factor, and L is the unbraced length. In BRB design, we aim for $P_e > P_y$ so that the core can yield completely without buckling, allowing the brace to behave in a ductile manner and effectively dissipate energy.

When the brace is laterally restrained along its length, the Euler buckling formula takes the form:

$$P_e = \frac{\pi^2 EI}{(nKL)^2} \quad (1.2)$$

Watanabe examined five specimens, each with a different cross-section, and adjusted the P_e/P_y ratios from 0.55 to 3.82. The experiments involved the application of cyclic loading and the observation of stable hysteresis loops in specimens where P_e was higher than P_y . This finding indicated that the control of the slenderness of the outer steel tube was a crucial factor in ensuring the stability of brace performance under seismic loads. The findings indicated that in scenarios where P_e was less than P_y , buckling occurred in a premature manner, thereby diminishing the brace's load-bearing capacity.

The experimental findings demonstrated that when the Euler buckling load of the steel tube exceeded the yield strength of the core member, the buckling of the element was effectively prevented by the restraining effect of the tube, thereby allowing the core to yield and exhibit stable hysteretic behavior. Conversely, when the steel tube's buckling load fell below the core's yield strength, premature buckling occurred, leading to diminished overall yield capacity of the member due to inadequate flexural stiffness. Furthermore, it was observed that initial imperfections had a notable impact: while

small initial deflections allowed stable yielding behavior, larger deflections—particularly around 1/100—led to early buckling before the core yielded, accompanied by a decrease in strength and stiffness. **[Figure 2.2]**

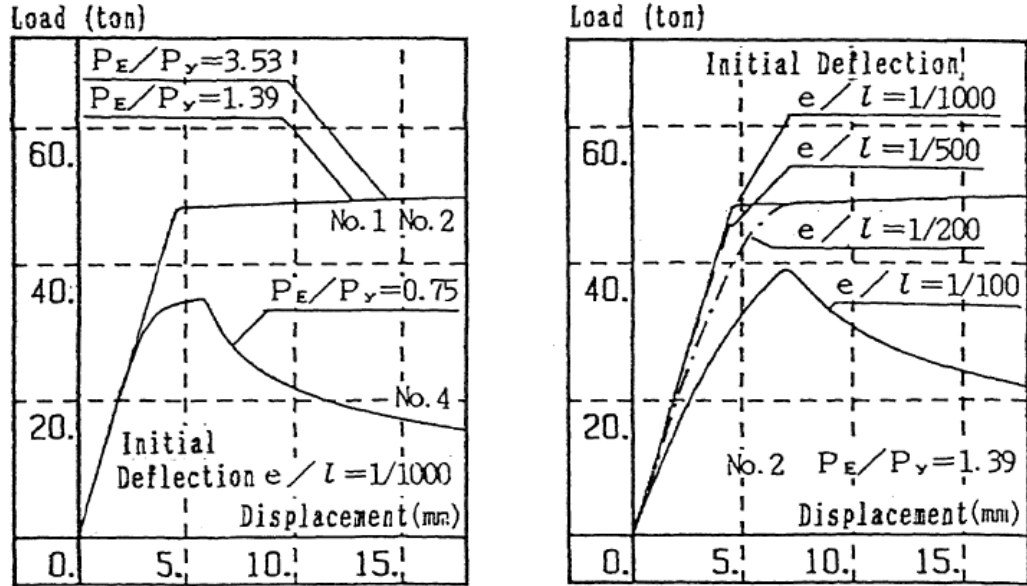


Figure 2.2: Result of the analysis [Watanabe et al., 1988].

According to the findings of the study, when the Euler buckling load of the steel tube exceeds the yield strength of the core element, buckling does not occur in the structural member. Furthermore, a stable hysteretic response is maintained even after the core yields. Based on these observations, it was recommended that the Euler buckling capacity of the diagonal members encased in steel tubes should be at least 1.5 times greater than the yield strength of the core element **[Watanabe et al., 1988]**.

This research marked the starting point of modern BRB systems, where initial stiffness and yield strength can be independently designed without concerns about buckling. The ability to ensure stable, symmetric energy dissipation under repeated seismic loading has made BRBs a widely adopted solution in both new construction and seismic retrofitting.

Subsequent to the pioneering experimental work of Watanabe and his associates in 1988, which demonstrated that Buckling-Restrained Braces (BRBs) could maintain stable behavior under cyclic loading **[Watanabe et al., 1988]**, endeavors to implement

these systems in actual buildings gained significant momentum. In 1989, engineers such as Wada and Saeki, along with their respective collaborators, initiated the utilization of BRBs in actual structures, with a particular focus on 10- to 15-story steel office buildings. This development signified the inception of the practical application of BRBs in earthquake-resistant design. **[Takeuchi et al., 2017].**

During the 1990s, BRB systems received significant attention in Japan. By the end of the decade, more than 160 buildings had adopted BRBs, reflecting structural engineers' growing trust in their performance [Takeuchi et al., 2017]. A significant conceptual development was initiated in July 1995 with the introduction of the concept of "Damage-Resistant Structures" by Wada and Iwata. This design approach aimed to reduce seismic damage by integrating elasto-plastic BRBs within an elastic main frame, thereby enhancing the building's energy dissipation capacity while preserving its overall integrity during earthquakes. **[Takeuchi et al., 2017].**

In order to facilitate the safe and standardised implementation of BRBs, the Architectural Institute of Japan (AIJ) published the first official design guidelines in 1996. These guidelines provided engineers with clear parameters for modelling and practical use.

The technology soon expanded beyond Japan. In 1998, BRBs were used internationally for the first time in the construction of the "Plant and Environmental Sciences" building at the University of California, Davis. To confirm the performance of the unbonded brace system used in this structure, three full-scale laboratory tests were conducted in 2000 at the Civil Engineering Structures Laboratory at the University of California, Berkeley. **[Clark, Aiken, Kasai, Ko, & Kimura, 1999]** **[Black, Makris, & Aiken, 2002].**

Developed in 1999 by Clark :

Following the preliminary studies, researchers in the United States initiated the development and testing of unbonded buckling-restrained braces (UBRBs). These braces employed a steel core encased within a concrete-filled steel tube, a configuration analogous to the Japanese design. However, a notable distinction emerged: the integration of a slip surface between the core and the tube. This modification aimed to mitigate bonding strength and facilitate independent axial movement **[Clark et al., 1999].**

In a series of large-scale experiments conducted to assess the performance of unbonded braces under various seismic conditions, Clark et al. tested three full-scale specimens—T-1, T-2, and T-3—each with identical lengths but different cross-sectional core areas. T-1 and T-2 featured rectangular core sections of 4.5 in² and 6.0 in² respectively, while T-3 had a cruciform core section of 8.0 in². These specimens were subjected to the SAC Basic Loading History, which was developed during Phase II of the FEMA/SAC Steel Project to evaluate steel moment frame connections. Additional test protocols included SAC near-field loading for T-1, low-cycle fatigue for T-2, and displacement records derived from the 1940 El Centro and 1994 Sylmar earthquakes for T-3. The goal was to simulate a variety of realistic seismic demands and to assess how these brace types perform under such stressors.

The force–displacement curves under the SAC Basic Loading History showed that all three specimens maintained their cross-sectional properties. The braces yielded at force levels close to those observed in coupon tests. Only the T-3 specimen showed slight fractures in the hysteresis behavior at higher force levels. The hysteresis loops are presented in **[Figure 2.3]. [Clark et al., 1999]**

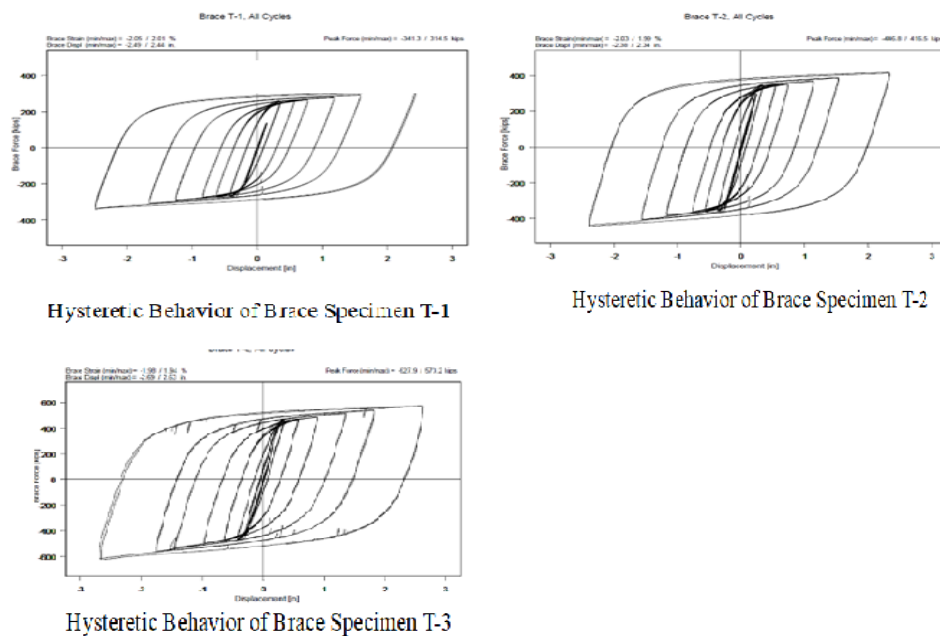


Figure 2.3: Hysteretic behavior of brace specimen T1,T2 and T3

The results indicate that the brace maintained stable hysteretic behavior even when it was subjected to a displacement offset of approximately 3.34 inches. Moreover, it continued to perform reliably under a maximum tension displacement of 4.84 inches, nearly twice the expected design displacement. This demonstrates that the T-1 specimen was able to resist large and asymmetric deformations without a significant loss of strength or stiffness. The test results for T-1 are presented in **[Figure 2.4]** confirming its stable performance under the SAC near-fault loading history.[Corte et al., n.d.] **[Clark et al., 1999]**.

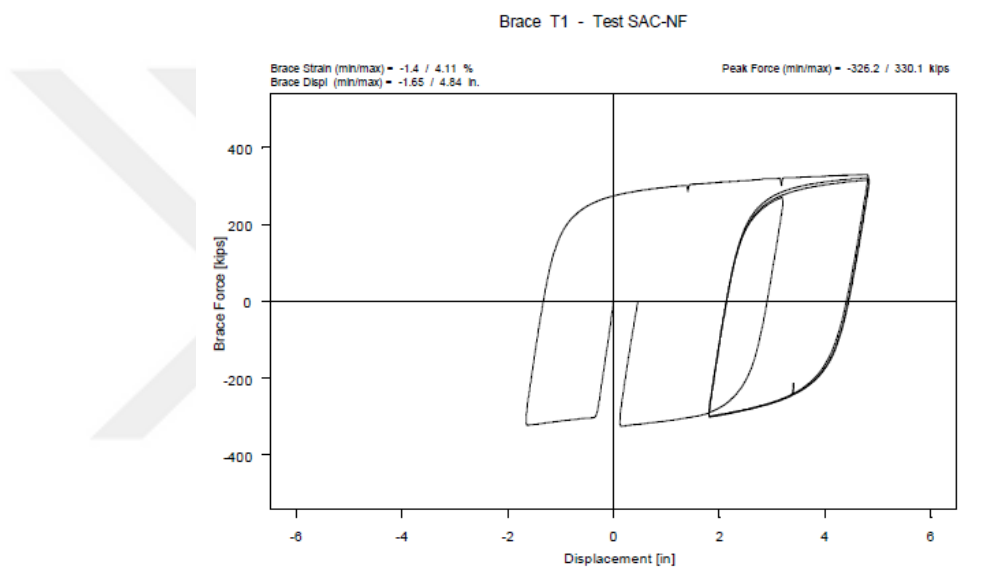


Figure 2.4: Hysteretic for specimen T-1 in SAC Near-field loading test

As illustrated in **[Figure 2.5]**, the force-displacement response of specimen T-2 under the low-cycle fatigue test is demonstrated. The brace demonstrated consistent stability throughout the loading process, exhibiting minimal loss in strength or stiffness. However, in the latter half of the 15th cycle, the core plate fractured internally within the outer tube. The specimen completed a total of 17.5 cycles at 2% strain before failure, including two earlier cycles from the Basic Loading History. **[Clark et al., 1999]**.

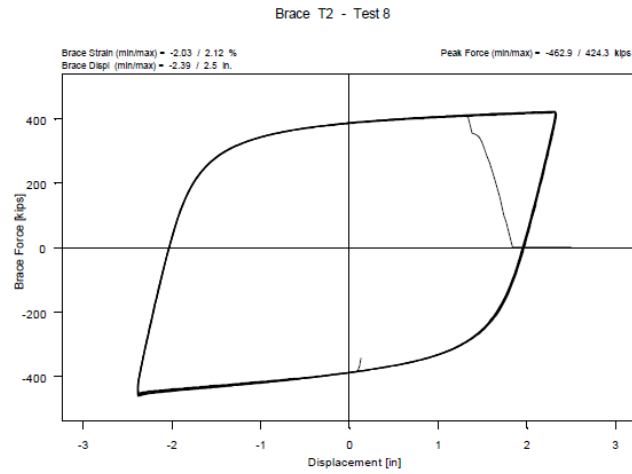


Figure 2.5: Hysteretic for specimen T-2 during Low-cycle fatigue test

As shown in [Figure 2.6], the T-3 specimen demonstrated consistent hysteretic behavior during the seismic displacement test based on the 1994 Sylmar earthquake record. The brace performed without any noticeable loss in strength or stiffness. Since the El Centro earthquake test produced a similar force–displacement response, its results were not included in this study [Clark et al., 1999] [Figure 2.6]

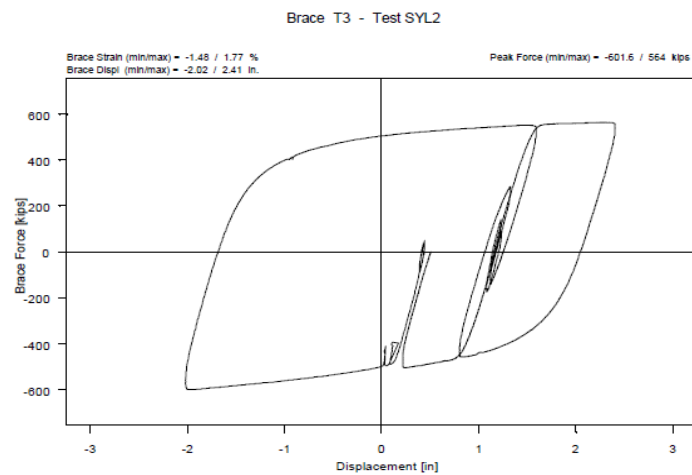


Figure 2.6: Hysteretic for specimen T-3 during Sylmar Earthquake

2.2 BRB Strengthening Applications in Reinforced Concrete Systems

In 1990, [Nagoa et al., 1990] presented a BRB design that incorporated a wide-headed section encased in reinforced concrete. This study was further advanced by Horie and colleagues in 1993, who proposed a concrete encased BRB with a cruciform steel core reinforced with steel fibers, aiming to improve lateral stiffness and advance the concept further.

In a study conducted by [Maheri & Sahebi, 1997], the use of steel bracing elements in reinforced concrete (RC) structures was investigated. According to the research findings, it was suggested that when steel braces are properly connected to RC frame systems, they could be considered as an alternative to conventional shear walls. The study indicated that, provided appropriate connection conditions are ensured, steel braces could positively contribute to the seismic performance of structures.

One of the most important features of BRBs—their ability to exhibit nearly identical deformations in both tension and compression zones—was emphasized in a study by [Kim & Choi, 2004]. According to this study, this characteristic significantly contributes to the improvement of key earthquake engineering parameters such as stiffness enhancement and energy dissipation. This, in turn, has played a role in increasing confidence in the use of BRBs.

By 2006, [Takeuchi et al., 2006] introduced the concept of integrated façade engineering as an innovative approach that combines architectural, structural, and environmental considerations. Within the scope of their study, they emphasized that for existing buildings, especially those requiring seismic retrofitting and posing safety risks in their current state—it is critical to preserve architectural aesthetics and maintain building functionality throughout the retrofitting process. This perspective led to the development of BRBs integrated into façade systems. As a result, this innovation paved the way for more energy-efficient, environmentally friendly, and architecturally appealing structures. This innovative façade system approach was implemented at the Tokyo Institute of Technology on a 6-story structure. [Figure 2.7]

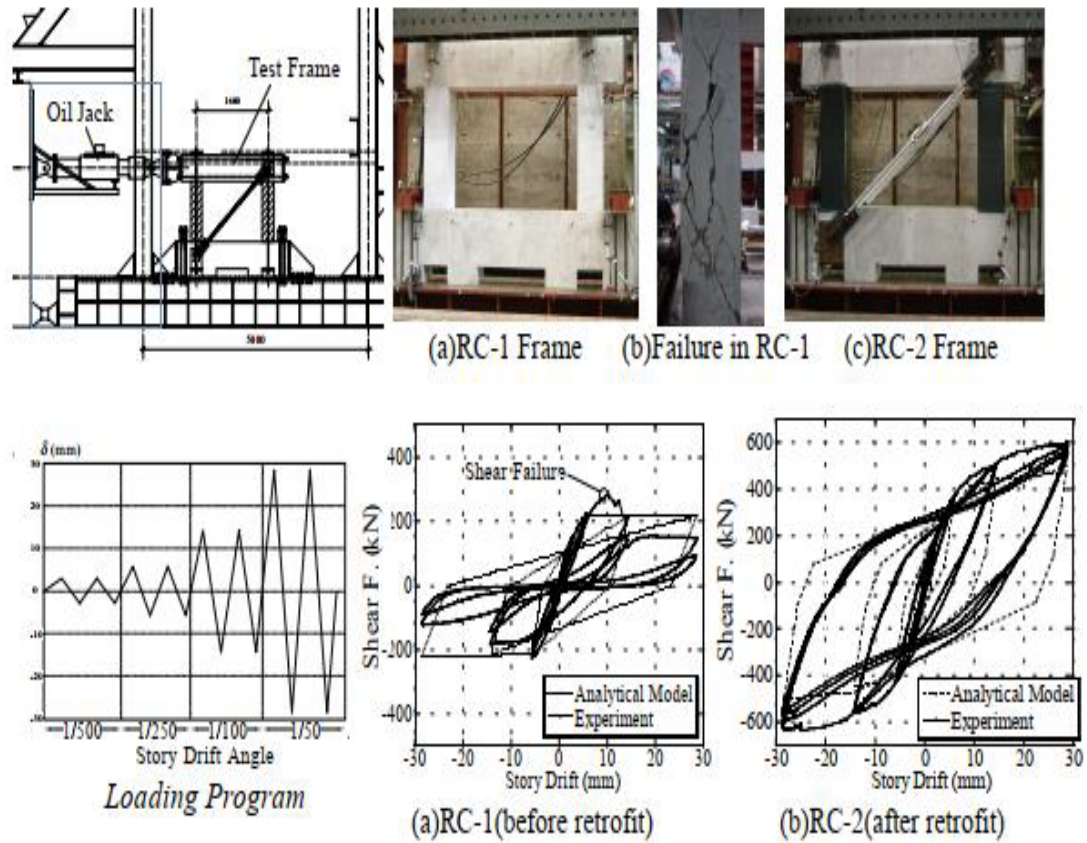


Figure 2.7: Experimental setup and results

A study conducted by **[Di Sarno & Manfredi, 2010]** analyzed the performance of a typical reinforced concrete structure. The nonlinear dynamic analyses revealed that, at the maximum damage state, BRBs were responsible for absorbing more than 60% of the total seismic energy input.

A 2014 study **[Khampanit, Leelataviwat, Kochanin & Warnitchai, 2014]** found that the interstory drifts of retrofitted frames were successfully controlled during DBE and MCE earthquakes through the use of Buckling-restrained Braces (BRBs). The findings also revealed that BRBs played a significant role in preventing soft-story irregularities in non-ductile structures. Furthermore, the retrofitted frames exhibited stable hysteretic behaviors, with minimal pinching and a response resembling a bilinear elastic-plastic curve. This controlled deformation allowed the structure to dissipate seismic energy more effectively, resulting in reduced overall damage. Test results showed that the energy dissipation capacity of frames strengthened with BRBs was up to ten times higher than that of bare reinforced concrete frames.

By 2015, Zhang, Wu, Mei, and Shing [2015] conducted a study in which a reinforced concrete frame with structural deficiencies was retrofitted using BRBs. The experimental and numerical results were in good agreement, and the proposed retrofit method was found to significantly prevent damage in beam and column regions while improving stiffness, lateral load capacity, and energy dissipation. The BRB-retrofitted frame demonstrated full and stable hysteretic loops, reflecting a consistent and reliable seismic response throughout the loading cycles. Moreover, the absence of any pinching up to 2.5% story drift ratio was clearly observed. [Zhang, Wu, Mei, & Shing, 2015]

One of the most common global strengthening techniques used today for reinforced concrete (RC) frame systems is the addition of shear walls or the integration of steel bracing elements. In a study conducted by [Mahrenholtz et al., 2014], it was shown that Buckling-Restrained Braces (BRBs) can generally be designed with smaller cross-sections compared to conventional steel braces, while also significantly improving the stiffness of the structure. The study emphasized that the fundamental advantage of BRBs lies in their ability to dissipate large amounts of seismic energy without experiencing strength degradation. It was also demonstrated that BRBs are capable of reliably absorbing energy under repeated cyclic loading.

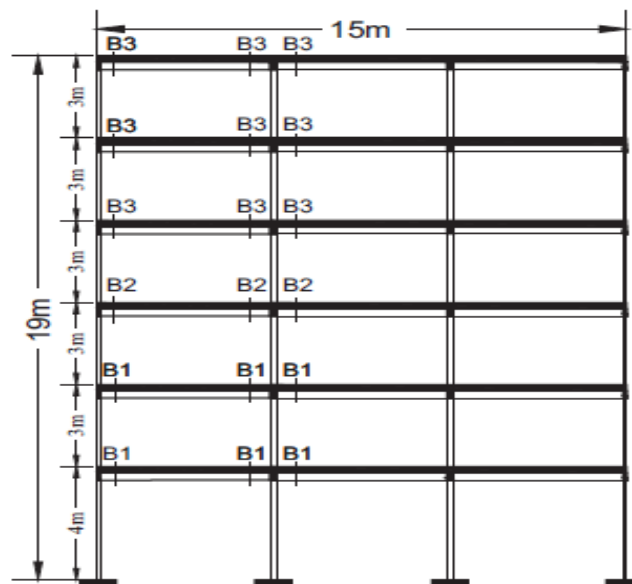


Figure 2.8: Figure typical frames elevation in the short direction

In the study conducted by **[Abou-Elfath et al., 2017]** , the seismic capacity of a six-story reinforced concrete (RC) building **[Figure 2.8]** was evaluated through the application of Buckling-Restrained Braces (BRBs). The seismic assessment was carried out using nonlinear static pushover analysis and time history analysis. Ten different ground motion records with varying levels of Peak Ground Acceleration (PGA) were used in the analyses. The key findings of the study are summarized as follows:

- Installing BRBs in only one bay of each exterior frame resulted in a significant improvement in the base shear capacity of the RC structure.
- The use of BRBs was found to be a highly effective technique for enhancing the PGA capacity of RC buildings.
- The study emphasized the importance of anchorage and welding zones, indicating that proper connection detailing can prevent premature failure of the braces.

[Guerrero et al., 2018] investigated the seismic performance of precast RC models with and without Buckling-Restrained Braces (BRBs) through shaking table tests. BRBs were found to effectively dissipate energy, reduce stiffness degradation, and improve compressive strength. Their use led to significantly lower inter-story drifts and lateral displacements, especially in Model 2 compared to a traditional moment-resisting frame. Even in retrofitted models with prior damage, BRBs contributed to better seismic behavior, showing strong potential to mitigate both structural and non-structural damage.

In a study conducted **[Erdil et al., 2024]** , RC specimens were constructed as reference frames and externally retrofitted. This external strengthening improved the rigidity and lateral strength of the deficient RC structure. Moreover, this system allowed the structure to be retrofitted without disturbing the occupants.

3. STRUCTURAL INFORMATION

This section provides detailed information regarding the building utilized in the numerical analyses conducted within this thesis. This section describes the retrofitting of an existing seven-story hospital building. The structure is located in a highly seismic region of Türkiye and represents the typical deficiencies found in existing reinforced concrete structures. This hospital building is located in a region of Türkiye with a very high seismic risk. All analytical procedures were performed using the ETABS 21.1.0 software.

In the evaluation of the existing conditions of the structure, detailed investigations were conducted, and the configuration details of critical structural elements—such as shear walls, columns, and beams—were examined, as they play a vital role in determining the current performance of the building. This inclusion aims to evaluate how traditional and innovative retrofit methods affect structural response and overall performance during seismic events.

3.1 Building Configuration and Structural Components

The building consists of 6 bays in the X direction and 5 bays in the Y direction, with each bay measuring 5 meters." The total residential area of the building is 600 m².

In the structural design of the existing building, two types of reinforcing steel were used: S420 for the longitudinal rebars and S220 for the stirrups. The concrete strength was taken as C20/25. The detailing of the reinforcement was based on the original project drawings and verified according to current seismic code requirements.

The ground floor height is 4.0 meters, while the typical story height is 3.2 meters. The total building height is 23.2 meters. The structure does not have a basement; only the superstructure is considered in the analysis. **[Figure 3.1]**

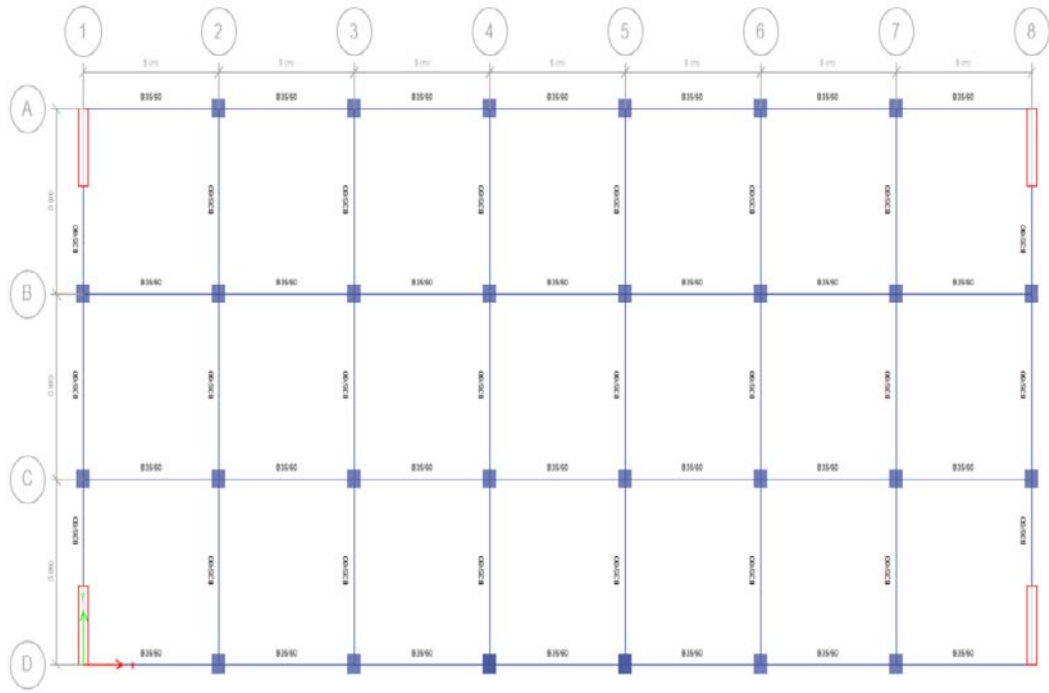


Figure 3.1: Structural layout plan of the building

Additionally, 35×210 cm reinforced concrete shear walls are placed along axes A and D, spanning from gridlines 1 to 8. These walls contribute significantly to the lateral load resistance of the building. **[Figure 3.2]**

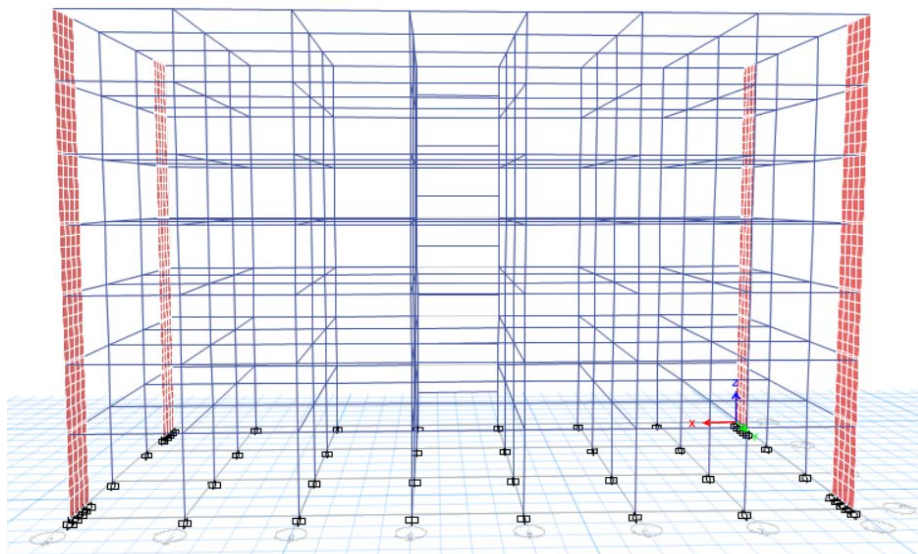


Figure 3.2: 3D view of the building

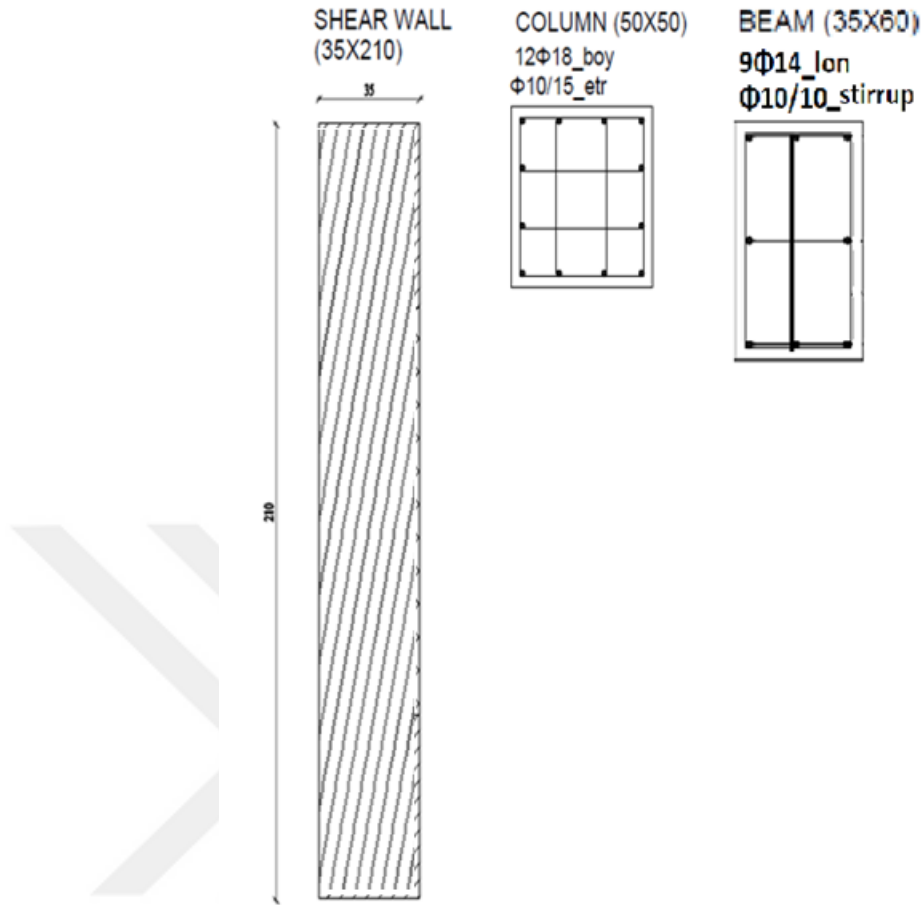


Figure 3.3: Element dimensions and reinforcement details

As shown in Figure 3.3, all columns have dimensions of 50×50 cm and all beams have dimensions of 35×60 cm. In addition, the shear walls are designed with dimensions of 35×210 cm, and detailed reinforcement configurations have been provided for the beams. **[Figure 3.3]**. The columns are reinforced with 12Φ18 mm longitudinal bars and Φ10/15 stirrups, placed in double stirrup arrangements. The beams are reinforced with Φ10/10 stirrups and have longitudinal reinforcement consisting of 9Φ14 bars. The hooks of the stirrups are bent at 135 degrees, ensuring proper confinement.

3.2 Material Property Definition

In the initial structural analysis of the building, material properties corresponding to concrete class C20 and reinforcing steel grade S420 were assigned. For transverse

reinforcement (stirrups and ties), steel grade S220 was used. The following material properties were adopted in the analysis: the modulus of elasticity for concrete (E_c) was taken as 28,000 MPa, the modulus of elasticity for reinforcing steel bars (E_s) was considered as 200,000 MPa, and for transverse reinforcement steel (stirrups), it was considered as 210,000 MPa.

Material safety factors were also applied to account for uncertainties in material properties and ensure structural safety. Specifically, a safety factor of 1.5 (γ_{mc}) was applied for concrete, 1.15 (γ_{ms}) for longitudinal reinforcement (S420), and 1.15 (γ_{ms}) for transverse reinforcement (S220), consistent with code requirements.

A concrete cover of 30 mm was provided for both longitudinal and transverse reinforcements in columns and beams, in accordance with durability and fire protection standards. It is assumed that all reinforcement was placed accurately according to the approved project drawings and relevant standards, ensuring the structural performance and serviceability criteria of the building.

3.3 Applied Loads and Load Definitions

In structural modeling, assigning correct loads for each floor is critically important to ensure accurate analysis results. Since the building is a hospital, the floor loads were determined carefully based on the intended use of different areas.

The loads for the internal walls were calculated separately for 15 cm and 10 cm thick brick walls. In addition to wall loads, the floor finishes were also considered in the model. The dead loads for different areas were calculated as follows:

- **Rooms:** Screed, ceiling plaster, and parquet layers were included.
- **Staircases:** Screed, ceiling plaster, and marble finishes were considered.

Each material's unit weight and thickness were taken into account to calculate the total dead load (in kN/m²) for each area. These loads were applied uniformly over the slabs in the model. [Figure 3.4]

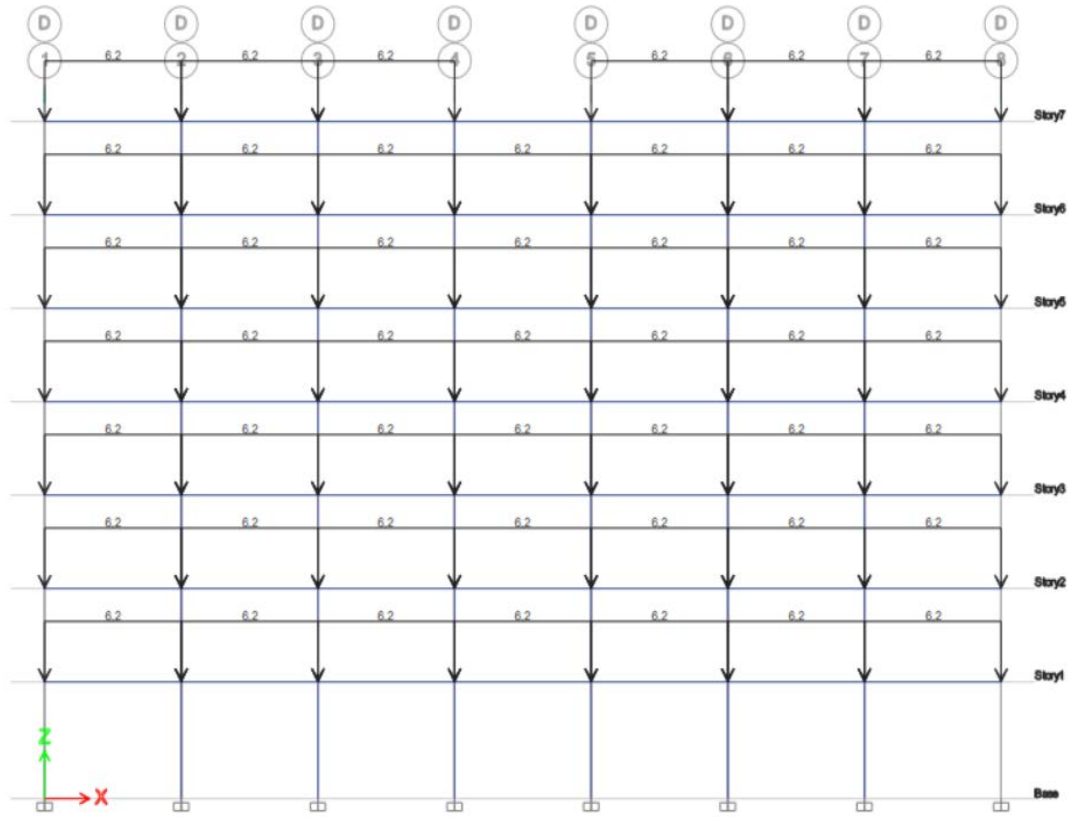


Figure 3.4: Wall elements loads along axis D

The following load cases were defined and assigned:

- **Dead Load (Dead)** It includes the self-weight of the structural elements and the additional permanent loads such as screed, ceiling plaster, floor finishes, and other fixed elements.
- **Live Load (Live)** :Live loads were assigned according to the usage functions of hospital areas, such as patient rooms, corridors, staircases, and balconies, following the according to TDDY 2018 earthquake design code.
- **Partition Wall Load (G_{wall})**: The loads from internal partition walls, made of 15 cm and 10 cm thick bricks, were calculated separately and defined under the load case G_{duvar} .
- **Additional Permanent Load**: Other minor permanent loads, such as door weights and small fixed elements, were considered under Additional Permanent Load.

- **Snow Load (S):** Snow load was applied only on the roof slab (top story) according to local snow load regulations.

Each load case was categorized under the appropriate type (Dead, Live, or Snow) and multiplied with the correct factor (1.0 for actual loads, 0.0 for reserved cases) in ETABS. By considering both permanent and live loads accurately, the model reflects the actual behavior and weight distribution of the hospital building.

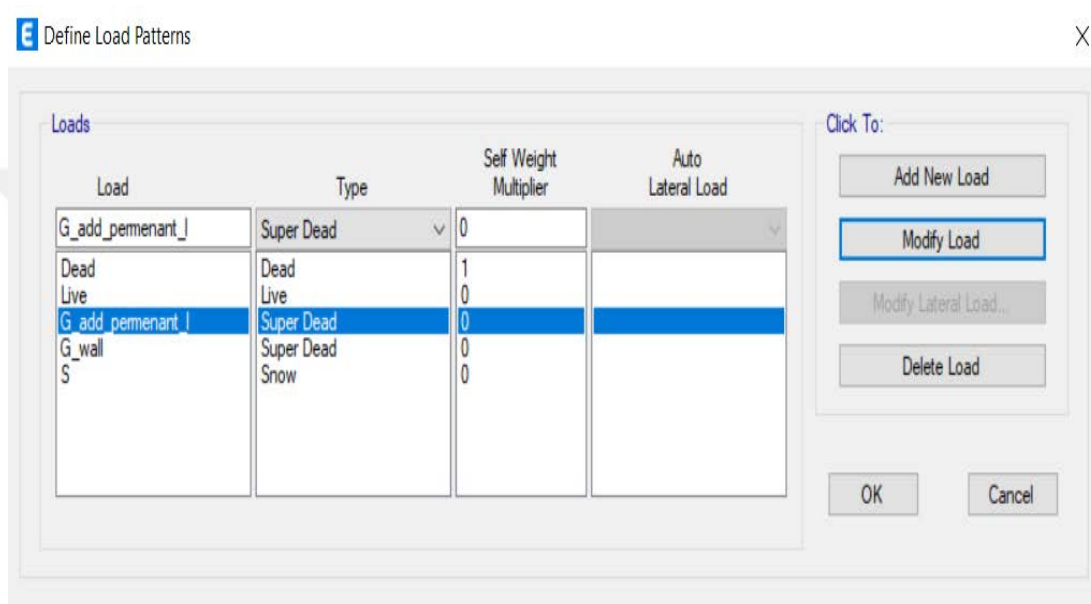


Figure 3.5: Definition of load pattern

3.4 Floor Loads and Assignments

In this project, floor loads were defined separately for the typical stories (Stories 1 to 6) and the roof story (Story 7) to reflect the real usage conditions of the building.

For the typical stories (Stories 1 to 6), the following uniform loads were applied:

- **Additional Dead Load (G_kap):** 3.8 kN/m²(This value includes the weight of floor finishes, screed layers, and ceiling plaster.)
- **Live Load (Live):** 3.5 kN/m²
(Assigned based on hospital occupancy requirements.)

For the roof story (Story 7), the load definitions were slightly different due to environmental conditions:

- Additional Dead Load (G_{kap}): 3.8 kN/m^2
- Live Load (Live): 1.5 kN/m^2
- Snow Load: 0.8 kN/m^2 (Snow load was only considered on the roof according to the local climatic data.) **[Figure 3.6]**

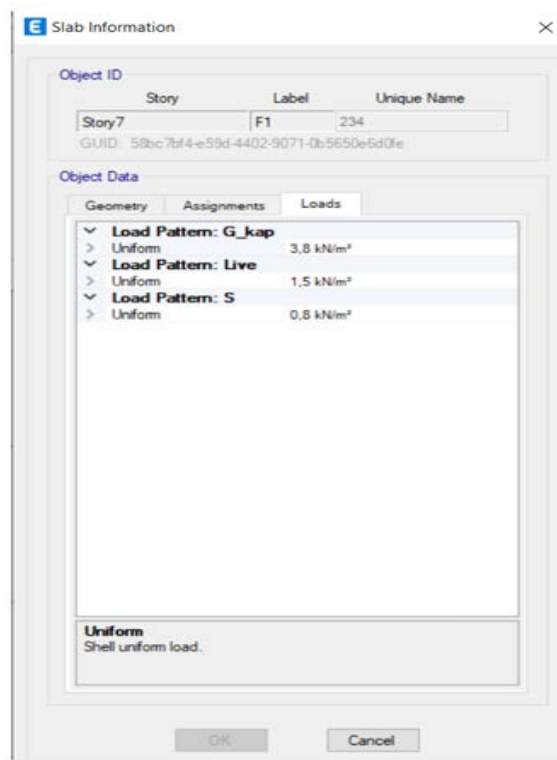


Figure 3.6: Floor loads

In addition to these loads:

- Floor Finish Load (Coating Load): 1.85 kN/m^2 was considered.
- Self-Weight of the RC Slab: 3.75 kN/m^2 , based on a slab thickness of 20 cm and assuming a concrete unit weight of 25 kN/m^3 .

All loads were assigned as uniform shell loads on the slab elements within the considered model.

Dead loads consist of both structural self-weight and non-structural permanent weights,

while live loads represent temporary service loads depending on the usage of each area.

[Figure 3.7]

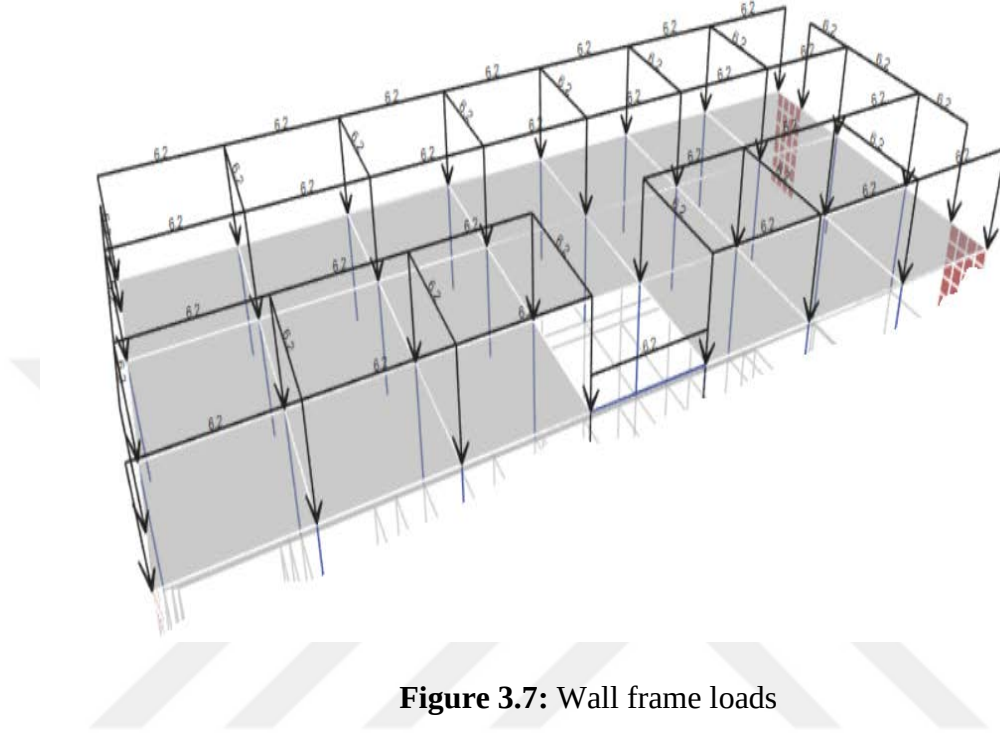


Figure 3.7: Wall frame loads

3.5 Earthquake Ground Motion Characteristics

In earthquake engineering, several ground motion parameters are used to describe seismic effects. **S_s** refers to the spectral acceleration for short-period structures, usually affecting low-rise and stiff buildings. **S₁** represents the spectral acceleration for long-period structures, such as tall or flexible buildings. **PGA** (Peak Ground Acceleration) shows the highest ground acceleration during an earthquake, while **PGV** (Peak Ground Velocity) indicates the maximum speed of ground movement. These values help engineers understand how ground motion will affect different types of structures.

The ground motion parameters corresponding to DD-1 level and ZC soil class were obtained from the official website provided by AFAD (<https://tdth.afad.gov.tr/TDTH/main.xhtml>), based on the coordinates of the building. The map showing the relevant location, as provided by AFAD, is presented in [Figure 3.8].



Figure 3.8: Structure location on AFAD map

Each parameter and its corresponding meaning, obtained from the AFAD website, are presented in [Table 3.1]. The methodology used to derive these values will be explained in the following sections.

Table 3.1 Seismic Parameters Provided by AFAD

Parameter	Value	Description
Seismic Ground Motion Level	DD-1	2% probability of exceedance in 50 years (return period of 2475 years)
Local Soil Class	ZC	Very dense sand, gravel, and stiff clay layers or weathered, highly fractured weak rock
S_s	1.862	Short-period spectral acceleration coefficient
S₍₁₎	0.536	Spectral acceleration coefficient at 1.0-second period
S_(DS)	2.234	Design spectral acceleration coefficient at short period
S_(D1)	0.785	Design spectral acceleration coefficient at 1.0-second period
PGA	0.739	Peak Ground Acceleration [g]
PGV	47.093	Peak Ground Velocity [cm/s]

Soil impact factors represented [Table 3.2] [Table 3.3]

Table 3.2 Local soil impact coefficients for short period region [TBDY,2018]

Local Soil Class	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \geq 1.50$
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	1
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF						

Table 3.3 Local soil impact coefficients for 1.0 second period [TBDY,2018]

Local Soil Class	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2
ZF						

3.6 Seismic Properties of the Structure

In the Turkish Building Earthquake Code (TBDY, 2018), local soil conditions are given significant weight in seismic design. Soils are categorized into six distinct types, ranging from ZA to ZF, based on their respective levels of stiffness. The construction of the building was undertaken using ZC soil, which is classified as medium-stiff. It is generally accepted that this type of soil typically results in moderate ground motion amplification. In order to reflect the soil effect in the design spectrum, two coefficients are utilised, depending on the spectral values. With the coefficients defined in the code and the spectral parameters obtained for our region, the design spectrum was calculated as shown below. These coefficients help us understand how much the earthquake forces will be affected by the local ground conditions. This ensures that the structural performance is evaluated more realistically.

According to TBDY 2018 equation 4.1, S_{DS} and S_{D1} values are calculated below.

$$S_{DS} = S_S \times F_S = 1,862 \times 1,200 = 2,234 \quad (3.1)$$

$$S_{D1} = S_1 \times F_1 = 0,536 \times 1,464 = 0,785 \quad (3.2)$$

[Figure 3.9] displays the Horizontal Elastic Design Spectrum, calculated in accordance with TBDY 2018 guidelines specified in Articles 4.3 to 4.6.

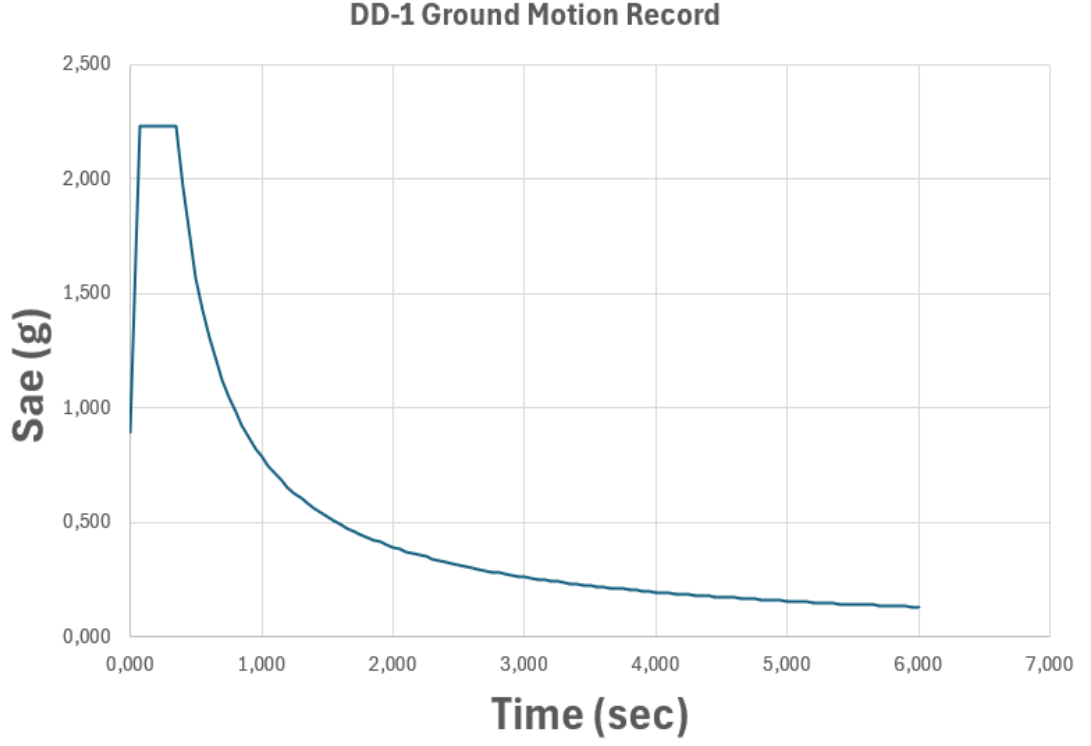


Figure 3.9: DD1 design spectrum

$$S_{AE}(T) = \left(0,4 + 0,6 \times \frac{T}{T_A}\right) \times S_{DS} \quad (0 \leq T \leq T_a) \quad (3.3)$$

$$S_{ae}(T) = S_{DS} \quad (T_a \leq T \leq T_B) \quad (3.4)$$

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

$$S_{ae}(T) = S_{D1} \times \frac{T_L}{T^2} \quad (3.6)$$

3.7 Building Importance Coefficient and Live Load Participation Coefficient

It is important to note that, in contrast to static loads, live loads may fluctuate over time, thereby introducing variability in their effect on the structure. This variation plays a pivotal role in dynamic analyses, given its influence on the interaction between mode shapes and natural frequencies. Consequently, the contribution of live loads to mass participation is regarded as one of the pivotal parameters during dynamic calculations. In Table 4.1 shows that, the live load mass participation coefficients are presented. As demonstrated in the following table, the coefficient (n) for hospital buildings is specified as 0.3. [Table 3.4]

Table 3.4: Building usage type

Building Usage Type	n
Warehouse, storage, etc.	0.80
School, dormitory, sports facility, cinema, theater, concert hall, place of worship, restaurant, store, etc.	0.60
<i>Residential, office, hotel, hospital, parking garage, etc.</i>	<i>0.30</i>

The building that has been retrofitted is a hospital. While evaluating its performance level before retrofitting, the earthquake code [TBDY 2018] was followed, which assigns different important factors based on the building's function. As shown in the table below, the importance factor (I) for hospital buildings is given as 1.5. The related section is marked clearly in [Table 3.5].

Table 3.5 Building use classification

Building Use Class	Purpose of Use of the Building	Building Importance Coefficient (I)
BKS 1	Buildings required for immediate use after an earthquake; buildings housing long-term intensive occupancy, valuable goods, or hazardous materials. (a) Buildings required for immediate occupancy after earthquakes (Hospitals, infirmaries, health centers, fire stations and associated facilities, PTT and other communication buildings, transportation terminals, energy production and distribution facilities, provincial and district governorate buildings, municipality administration facilities, emergency management and disaster coordination centers). (b) Educational buildings such as schools, dormitories, military barracks, prisons, etc. (c) Museums and historical buildings. (d) Facilities storing hazardous materials (toxic, explosive, flammable substances).	1,50
BKS 2	Buildings with short-term but intensive occupancy, such as shopping malls, sports facilities, cinemas, theaters, concert halls, and places of worship.	1,20
BKS 3	Other buildings not classified under BKS 1 and BKS 2, including residences, workplaces, hotels, industrial-type structures, etc.	1,00

4. IDENTIFICATION OF THE ANALYSIS APPROACH

In the earthquake code implemented in Türkiye since 2018, specific methods are required for the assessment of existing buildings. Therefore, in this study, nonlinear methods will be used to evaluate the existing structure. These nonlinear analysis methods are explained in detail in Chapter 5 of the Turkish Seismic Code (TBDY). Within the scope of this study, the pushover analysis method has been selected and will be applied.

As part of the pushover analysis, the building will be pushed until it reaches the target displacement, allowing us to determine the current performance level of the structure. If the results indicate a need for retrofiting, an appropriate strengthening method will be proposed for this hospital building.

The table below presents the analysis method and performance objective to be used for the existing structure. Since the design is based on the DD-1 earthquake, which corresponds to a return period of 2475 years, the relevant section has been selected and marked. [Table 4.1]

Table 4.1: Existing structure target performance analysis approach [TBDY 2018].

Earthquake Ground Motion Level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a, 2a	
	Standard Performance Objective	Design Approach	Enhanced Performance Objective	Design Approach
DD-3	—	—	SH	ŞGDT
DD-2	KH	ŞGDT	—	—
DD-1	—	—	KH	ŞGDT

4.1 Method Selection for Nonlinear Static Evaluation

In order to apply Pushover Analysis, certain conditions defined in TBDY 2018 must be met. First, the building must have regular torsional behavior, and second, the effective mass participation of the dominant mode in the direction of the earthquake must be at least 70% of the total building mass. These rules help ensure that the analysis reflects the actual behavior of the structure.

In this method, the structure is pushed step by step until a target displacement is reached. This process allows engineers to evaluate the performance level of the building. If the result shows that the structure does not meet the required performance, retrofitting strategies can be developed.

Since the mass participation ratios in both directions are greater than 70% and there is no torsional irregularity, the pushover analysis can be confidently applied to this structure. The analysis has been defined separately for both directions, making it suitable for evaluating the performance of the existing building effectively.

4.2 Determination of Plastic Hinge Model

A variety of approaches to hinge modelling are employed internationally in the field of nonlinear structural analysis to represent inelastic behavior. These methods are generally classified as either concentrated plasticity, where deformations are assumed to occur at specific points (e.g., plastic hinge, nonlinear spring), or distributed plasticity, where inelastic behavior is spread along the element (e.g., or finite-length hinge zone, fiber section or finite element models).

In this study, the fiber hinge model was used with the help of the ETABS program. The modeling was carried out separately for shear walls, columns, and beams.

4.3 Nonlinear Static Analysis

In this section, the suitability of the building for pushover analysis was evaluated. According to the requirements of the seismic code, mass participation ratios must exceed 70% in both directions, and no torsional irregularity should be present. Since these conditions are satisfied in this structure, the pushover analysis is considered

applicable. The analyses were defined separately in both directions to ensure a reliable evaluation of the building's seismic performance.

4.3.1 Mass Participation Ratios in Both Directions

As demonstrated in the accompanying [Table 4.2], the mass participation ratios in the X and Y directions are 0.866 and 0.747, respectively. The values obtained correspond to 86.6% and 74.7%, both of which are greater than the minimum requirement of 70% stipulated in the seismic code [TBDY 2018] for the application of pushover analysis.

Therefore, it can be concluded that the structure meets the necessary conditions for pushover analysis in both principal directions, and the modal responses are sufficient to represent the dynamic behavior of the system.

Table 4.2: Dynamic properties of structure

Dynamic Properties of Structure					
Mod	Periyot (s)	UX	UY	Toplam UX	Toplam UY
1	1,0460	0,8657	0,0000	0,866	0,000
2	0,7790	0,0000	0,7467	0,866	0,747

4.3.2 Torsional irregularity Control (β -Check)

The building has a symmetric configuration, and torsional irregularity checks were carried out in both directions. The maximum ratio was found to be 1.05 in the X direction, and nearly 1 in the Y direction due to the building's symmetry. Thus, the structure satisfies the torsional irregularity criterion of 1.2 defined in the seismic code.

4.3.3 Overview of Pushover Method

Summary of Conditions for Single-Mode Pushover Analysis Applicability:

- Modal mass participation ratio of the first mode $\geq 70\%$.
- Torsional irregularity coefficient ($B \leq 1.2$).

Following a thorough examination of the requisite criteria, it has been verified that the structure demonstrates regular behavior and is deemed suitable for nonlinear static (pushover) analysis.

4.4 Material Nonlinear Properties

In this section, the limit values of the reinforcement and concrete materials to be used in the studied structure will be calculated for use in nonlinear analysis methods.

4.4.1 Determination of the Rebar Model

In this section, the formulas provided in Section 5A.7 of the Turkish Seismic Code (TBDY) will be used to evaluate the displacement characteristics of structural elements through nonlinear analysis methods. These formulas are essential for defining material behavior. The properties of reinforcement steel elements are defined in accordance with TBDY 2018, as illustrated in the graph provided. **[Figure 4.1]**

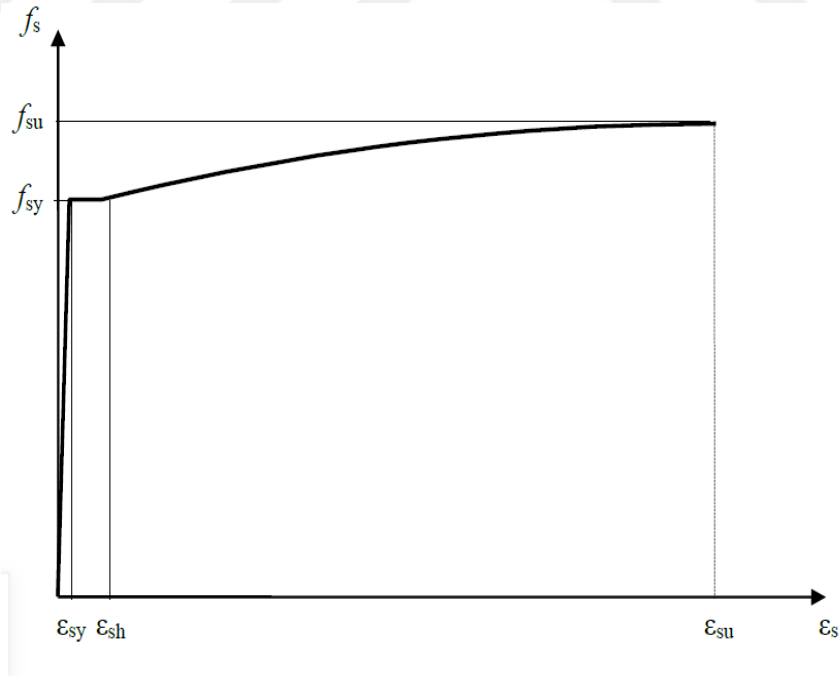


Figure 4.1: Properties of reinforcement steel according to [TBDY 2018]

$$f_s = E_s \varepsilon_s \quad (\varepsilon_s \leq \varepsilon_{sy}) \quad (4.1)$$

$$f_s = f_{sy} \quad (\varepsilon_{sy} \leq \varepsilon_s \leq \varepsilon_{sh}) \quad (4.2)$$

$$f_s = f_{su} - (f_{su} - f_{sy}) \frac{(\varepsilon_{su} - \varepsilon_s)^2}{(\varepsilon_{su} - \varepsilon_{sh})^2} \quad (\varepsilon_{sh} < \varepsilon_s \leq \varepsilon_{su}) \quad (4.3)$$

The information regarding reinforcement steels has been presented based on the values defined in the seismic code. The characteristic properties and corresponding parameters, as specified in the regulation, are given in the table for clarity and reference. **[Table 4.3]**

Table 4.3: Information on reinforcement steels [TBDY-2018]

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

The table below was created using the relevant formulas provided in the seismic code for the S420 reinforcement steel. **[Table 4.4]**

Table 4.4: Stress-strain relationship S420

Limit State	Strain	Stress	Limit State	Strain	Stress
es,u	-0,08	-525		0,000525	105
	-0,072	-523,704		0,00105	210
	-0,036	-485,787		0,001575	315
es,CP	-0,032	-478,333		0,0021	420
es,LS	-0,024	-461,481	es,IO	0,008	420
	-0,02	-452,083		0,012	431,3426
	-0,016	-442,037	es,LS	0,024	461,4815
	-0,012	-431,343	es,CP	0,032	478,3333
es,IO	-0,008	-420		0,042	495,7523
	-0,0021	-420		0,052	509,1204
	-0,00158	-315		0,062	518,4375
	-0,00105	-210		0,072	523,7037
	0,000525	105	es,u	0,08	525
	0	0			

Based on the values provided in the table, the stress-strain graph of the reinforcement steel has been generated, with the limit values clearly indicated. **[Figure 4.2]**

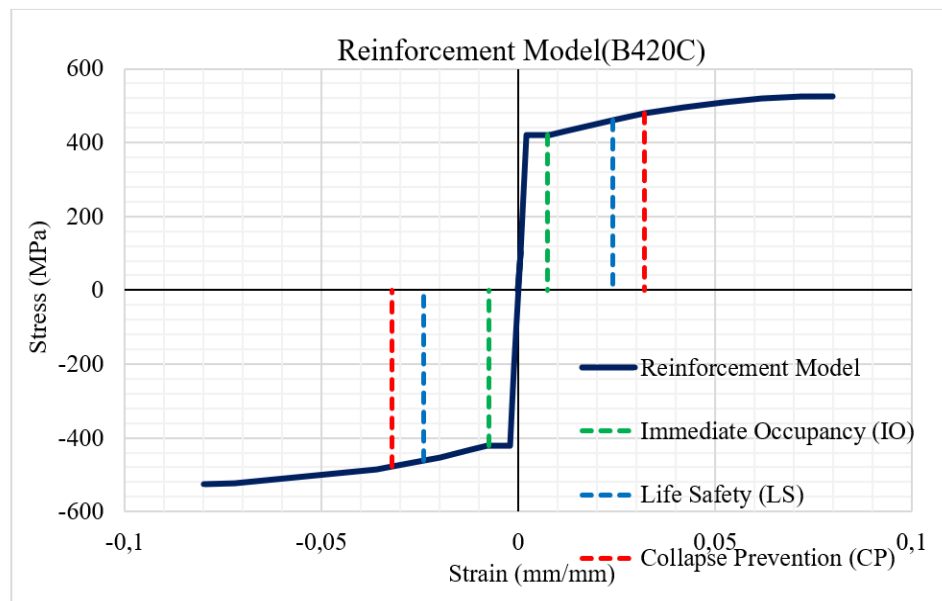


Figure 4.2: Reinforcement model for S420B

The table below was created using the relevant formulas provided in the seismic code for S220 grade reinforcement steel, which was used for stirrups. The values were obtained as presented in the [Table 4.5].

Table 4.5: Table stress-strain relationship S220

Limit State	Strain	Stress	Limit State	Strain	Stress
ϵ_s, u	-0,12	-264		0,000275	55
	-0,108	-263,467		0,00055	110
	-0,078	-257,467		0,000825	165
ϵ_s, CP	-0,048	-244,802		0,0011	220
ϵ_s, LS	-0,036	-237,869	ϵ_s, IO	0,011	220
	-0,023	-229,155		0,015	223,1701
	-0,019	-226,222	ϵ_s, LS	0,036	237,8689
	-0,015	-223,17	ϵ_s, CP	0,048	244,8016
ϵ_s, IO	-0,011	-220		0,058	249,7642
	-0,0011	-220		0,068	253,986
	-0,00083	-165		0,078	257,4672
	-0,00055	-110		0,088	260,2077
	0,000275	55	ϵ_s, u	0,12	264

The stress-strain graph of the reinforcement steel has been generated based on the values provided in the table, with the limit values clearly indicated. [Figure 4.3]

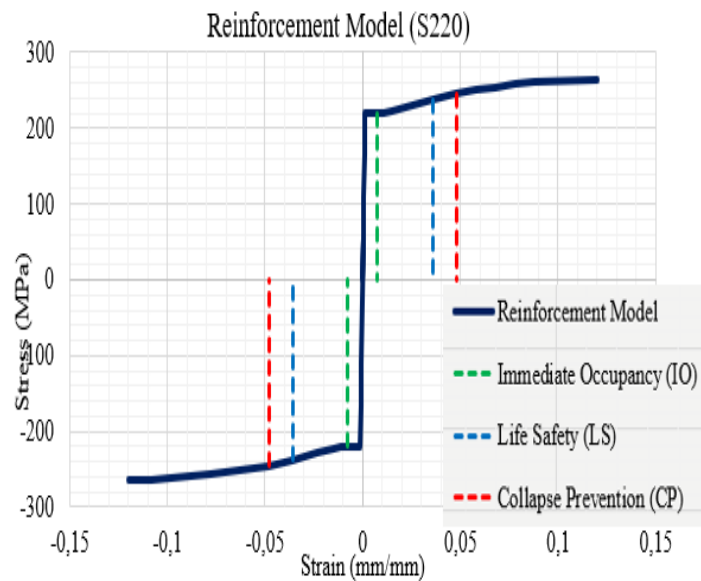


Figure 4.3 Reinforcement model for S220

4.5 Mander Model Definition

According to TBDY 2018, the Mander model helps to define the behavior of confined concrete more accurately. It considers the effect of stirrups or ties, which improve the strength and ductility of concrete. This model is especially useful in critical zones like plastic hinges and gives a better stress–strain curve under compression. That’s why it is suitable for nonlinear analysis. The confined and unconfined concrete models defined in the code are illustrated in **[Figure 4.4]**

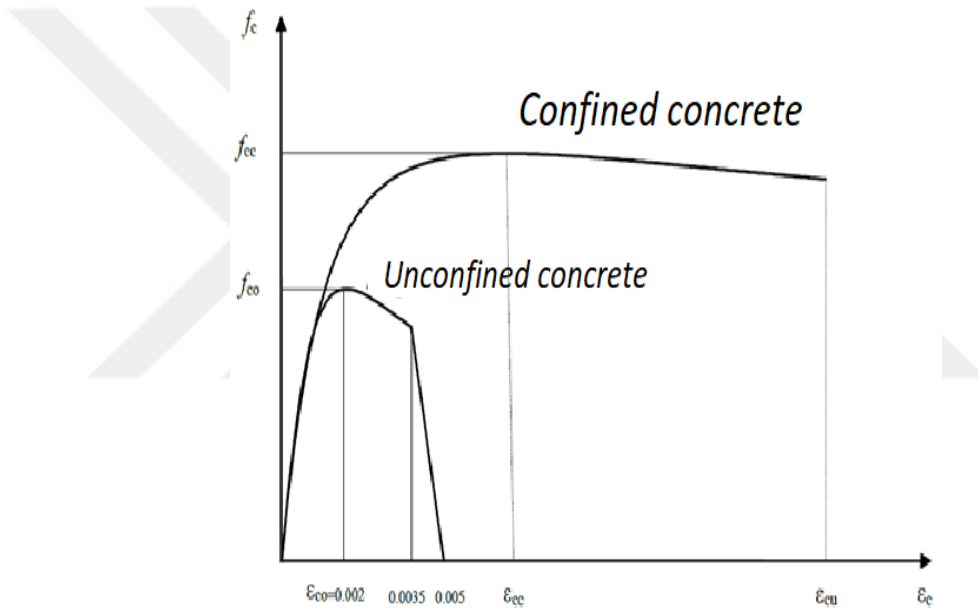


Figure 4.4 Confined and unconfined concrete – Mander Model [TBDY 2018].

In confined concrete, the concrete compressive stress f_c is expressed as a function of the compressive ε_c using the relation given in Equations: **[TBDY-2018]**

$$f_c = \frac{f_{cc} \times r}{r - 1 + x^r} \quad \text{TBDY-2018 5A.1 (4.4)}$$

The relationship between the confined concrete strength and the unconfined concrete strength is expressed in the equation below.

$$f_{cc} = \lambda_c f_{co} \quad ; \quad \lambda = 2,254 \sqrt{1 + 7,94 \frac{f_e}{f_{co}}} - 2 \frac{f_e}{f_{co}} - 1,254 \quad (4.5)$$

f_{cc} = Compressive strength of confined concrete

f_{co} = Compressive strength of unconfined concrete

f_e = Effective lateral confining pressure

f_e = Effective lateral confining pressure

k_e = Confinement effectiveness coefficient

f_{yw} = Yield strength of transverse reinforcement

ρ_y = Volumetric ratio of transverse reinforcement in the respective directions

The effective lateral confining pressure, f_e , in rectangular sections can be considered as the average of the values calculated in two orthogonal directions, as defined in Equation (4.6).

$$f_{ex} = k_e \rho_x f_{yw} \quad ; \quad f_{ey} = k_e \rho_y f_{yw} \quad (4.6)$$

In these expressions, f_{yw} denotes the yield strength of the transverse reinforcement, ρ_x and ρ_y represent the volumetric ratios of transverse reinforcement in the respective directions, and k_e refers to the confinement effectiveness coefficient as defined in Equation (4.7).

$$k_e = \left(1 - \frac{\sum a_i^2}{6b_o h_o}\right) \times \left(1 - \frac{s}{2b_o}\right) \times \left(1 - \frac{s}{2h_o}\right) \left(1 - \frac{A_s}{b_o h_o}\right)^{-1} \quad (4.7)$$

a_i = Distance between the axes of vertical reinforcement around the perimeter

A_s = Area of longitudinal reinforcement

b_o = Core dimension measured between the centerlines of transverse reinforcement

h_o = Core dimension measured between the centerlines of transverse reinforcement

s= Spacing transverse reinforcement

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \quad ; \varepsilon_{cc} = \varepsilon_{co} [1 + 5(\lambda_c - 1)] \quad ; \varepsilon_{co} = 0,02 \quad (4.8)$$

$$r = \frac{E_C}{E_C - E_{SEC}} \quad ; E_C \cong 500\sqrt{f_{co}} \text{ [Mpa]} \quad ; E_{SEC} = \frac{f_{cc}}{\varepsilon_{cc}} \quad (4.9)$$

Mander Model for Column

Detailed information regarding the column element defined in the structural model using the Mander approach is presented in **[Table 4.6]**.

Table 4.6: Column configuration details

Abbreviation List		Column Information		
n_x	Number of stirrups Xdirection	n_x	4	-
n_y	Number of stirrups ydirection	n_y	4	-
h	Section dimension Ydirection	b	50	cm
b	Section dimension Xdirection	h	50	cm
s	Stirrups Distance	d	3	cm
d	Clear Concrete Cover	s	10/15	cm
φ	Diameter of Stirrups	φ	10	0,785
CXX	Concrete Grade	C	20	-
SXX	Rebar Grade	S	420	-
The number and radius of the reinforcement bars.		12	φ	18

The values, which were obtained through the application of the relevant code-defined formulas, are comprehensively presented in **[Table 4.7]** for clarity and reference.

Table 4.7: Mander Model parameters for column

Deformation Limits		
a_{iy}	14,7	cm
a_{ix}	14,66667	cm
b_o	44	cm
h_o	44	cm
α_{se}	0,535	-
ρ_{sx}	0,0042	
ρ_{sy}	0,0042	
ω_{we}	0,0434	
ϵ_c^{CP}	0,0118	
ϵ_c^{LS}	0,0089	
ϵ_c^{IO}	0,0025	
A_s	30,5	cm ²
ρ'	0,015765	-
k_e	0,544	-
f_e/f_{co}	0,048	-
λ_c	1,298	-
f_{cc}	24,7	Mpa
ϵ_{cc}	0,0050	mm/mm
E_{sec}	4965,473	MPa
E	22360,68	MPa
r	1,285451	-

As shown in **[Figure 4.5]**, the graph illustrates the stress–strain behavior of confined concrete, which is widely used in seismic applications due to its improved ductility. The curve represents the behavior of the material under increasing strain, showing an initial rise in stress followed by a gradual reduction after the peak point. Performance levels are marked with dashed lines, indicating the strain limits defined for different damage states. These thresholds are used to assess whether structural elements meet the expected performance criteria under seismic loading, in line with performance-based design principles.

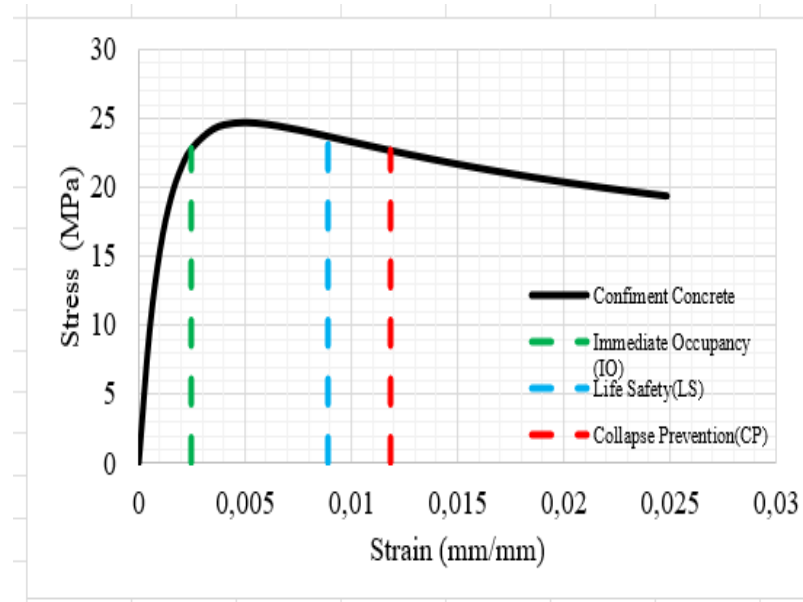


Figure 4.5 Column stress-strain curve (Mander Model)

Mander Model for Beams:

In the numerical analysis of the structural system, detailed information regarding the beam element defined using the Mander model is presented in [Table 4.8].

Table 4.8: Beam configuration details

Abbreviation List		BEAM-SECTION(35X60)		
n_x	Number of stirrups X direction	n_x	3	-
n_y	Number of stirrups y direction	n_y	3	-
h	Section dimension Y direction	b	35	cm
b	Section dimension X direction	h	60	cm
s	Stirrups Distance	d	3	cm
d	Clear Concrete Cover	s	10	cm
ϕ	Diameter of Stirrups	ϕ	10	0,785
CXX	Concrete Grade	C	20	-
SXXX	Rebar Grade	S	420	-
The number and radius of the reinforcement bars.		6	ϕ	14

The required parameters for the beam element, modeled using the Mander approach, were calculated based on the code-specified formulas and are presented in [Table 4.9].

Table 4.9: Mander model parameters for beam

a_{ty}	27,0	cm
a_{tx}	14,5	cm
b_o	29	cm
h_o	54	cm
α_{se}	0,451	-
ρ_{sx}	0,0067	
ρ_{sy}	0,0039	
ω_{we}	0,0343	
ϵ_c^{CP}	0,0109	
ϵ_c^{LS}	0,0082	
ϵ_c^{IO}	0,0025	
A_s	9,2	cm ²
ρ'	0,005895	-
k_e	0,453	-
f_e/f_{co}	0,051	-
λ_c	1,314	-
f_{cc}	25,0	Mpa
ϵ_{cc}	0,0051	mm/mm
E_{sec}	4868,903	MPa
E	22360,68	MPa
r	1,278354	-

The graph was generated and plotted in Excel based on the values obtained from the table using the necessary parameters for the Mander model [Figure 4.6]

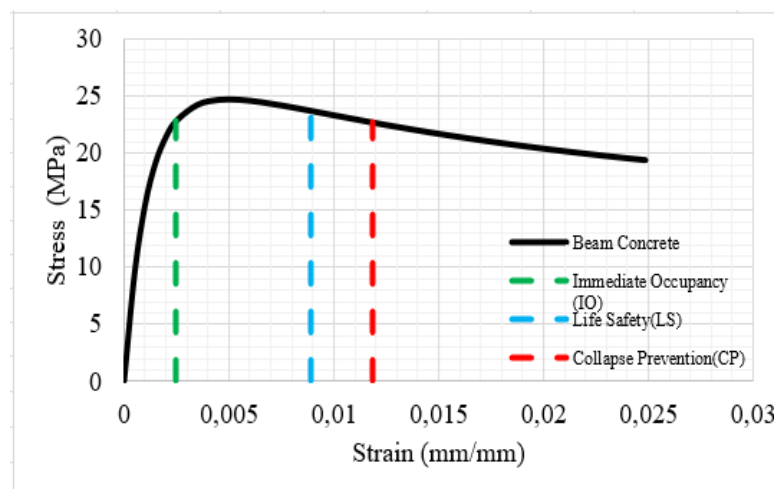


Figure 4.6: Beam stress-strain curve (Mander Model)

4.6 Definition of Plastic Hinges

Masses are defined in structural models in accordance with Section 4.5.9 of the Turkish Seismic Code (TBDY, 2018), which outlines the procedures for assigning seismic masses prior to modal analysis. Following this, modal analysis is performed to determine the dynamic characteristics of the structure. In reinforced concrete buildings, the effective section stiffness of structural members is determined based on the expected axial load levels and section properties. According to **[TBDY, 2018]** Section 5.4.4., when modeling plastic behavior using lumped plasticity approaches, the effective stiffness of reinforced concrete sections should be defined in accordance with Equations 4.10 and 4.11, which consider the influence of cracking and inelastic behavior under seismic loads. Fiber hinge assignment was performed in the structural model.

$$(EI)_C = \frac{M_y L_s}{\theta_y} \quad (4.10)$$

$$\theta_y = \frac{\varphi_y L_s}{3} + 0,0015\eta \left(1 + 1.5 \frac{h}{L_s} \right) + \frac{\varphi_y d_b f_{ye}}{8\sqrt{f_{ce}}} \quad (4.11)$$

EI = Bending stiffness

M_y = Effective yield moment at member ends (plastic hinges)

L_s = Shear span

θ_y = Average yield rotation

φ_y = Effective yield curvature at plastic hinge section

d_b = Average diameter of bars anchored to support (node or foundation)

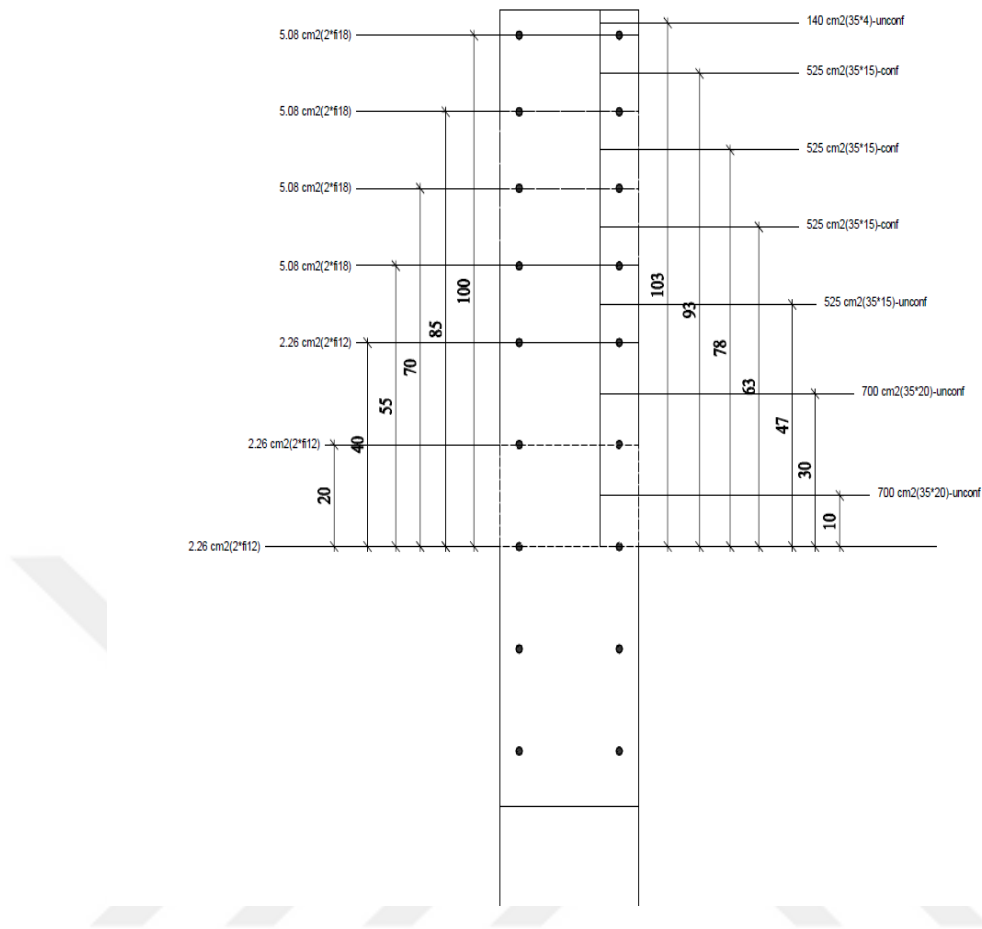


Figure 4.7: Fiber hinge shear wall definition

The header region of the shear wall element was divided into 27 segments. The first 13 of these correspond to the regions containing reinforcement. The distances from the center of each reinforcement bar were specified. **[Figure 4.7].**

The table below provides a detailed calculation breakdown for each of the segmented elements. The first row indicates the element number. The second row shows the reinforcement areas for each region up to the 13th point; after that, the area of the selected concrete is expressed in cm². The third column gives the distance to the reference point, along with coordinates, and the material types used are listed in the final column **[Figure 4.10].**

Table 4.10: Fiber hinge for shear wall

1	5,1	100	S420 /// SSC1
2	5,1	85	S420 /// SSC1
3	5,1	70	S420 /// SSC1
4	5,1	55	S420 /// SSC1
5	2,3	40	S420 /// SSC1
6	2,3	20	S420 /// SSC1
7	2,3	0	S420 /// SSC1
8	2,3	-20	S420 /// SSC1
9	2,3	-40	S420 /// SSC1
10	5,1	-55	S420 /// SSC1
11	5,1	-70	S420 /// SSC1
12	5,1	-85	S420 /// SSC1
13	5,1	-100	S420 /// SSC1
14	140	103	C20-conf /// SSC1
15	525	93	C20-conf /// SSC1
16	525	78	C20-conf /// SSC1
17	525	63	C20-conf /// SSC1
18	525	47	C20-conf /// SSC1
19	700	30	C20-conf /// SSC1
20	700	10	C20-conf /// SSC1
21	700	-10	C20-conf /// SSC1
22	700	-30	C20-conf /// SSC1
23	525	-47	C20-conf /// SSC1
24	525	-63	C20-conf /// SSC1
25	525	-78	C20-conf /// SSC1
26	525	-93	C20-conf /// SSC1
27	140	-103	C20-conf /// SSC1

From an engineering perspective, the necessary regions were defined individually, and fiber hinges were assigned in the ETABS software accordingly. This modeling was performed separately for each structural element, and fiber hinges were assigned individually to each of them.

The required fiber hinge assignments have been made for both column and beam elements in the regions where plastic hinge formation is expected. These assignments are illustrated in **[Figure 4.11]** for columns and **[Figure 4.12]** for beams.

Table 4.11: Fiber hinge for column

Area(cm2)	Coord3(cm2)	Coord2(cm2)	Material
1	1,5	11,8	Rebar
2	1,5	11,8	Rebar
3	1,5	-11,8	Rebar
4	1,5	-11,8	Rebar
5	1,5	0	Rebar
6	1,5	0	Rebar
7	86,6	-15,1	C20 Beam /// SSC1
8	78,7	0	C20 Beam /// SSC1
9	86,6	15,1	C20 Beam /// SSC1
10	135	-15,1	C20 Beam /// SSC1
11	135	15,1	C20 Beam /// SSC1
12	86,6	-15,1	C20 Beam /// SSC1
13	78,7	0	C20 Beam /// SSC1
14	86,6	15,1	C20 Beam /// SSC1
15	131,7	-9,7	C20 Beam /// SSC1
16	131,7	0	C20 Beam /// SSC1
17	131,7	9,7	C20 Beam /// SSC1
18	131,7	-9,7	C20 Beam /// SSC1
19	263,4	0	C20 Beam /// SSC1
20	131,7	9,7	C20 Beam /// SSC1
21	131,7	-9,7	C20 Beam /// SSC1
22	131,7	0	C20 Beam /// SSC1
23	131,7	9,7	C20 Beam /// SSC1

Table 4.12 Fiber hinge for beam

No	Coord3(cm2)	Coord2(cm2)	Material
1	1,5	11,8	Rebar
2	1,5	11,8	Rebar
3	1,5	-11,8	Rebar
4	1,5	-11,8	Rebar
5	1,5	0	Rebar
6	1,5	0	Rebar
7	86,6	-15,1	C20 Beam /// SSC1
8	78,7	0	C20 Beam /// SSC1
9	86,6	15,1	C20 Beam /// SSC1
10	135	-15,1	C20 Beam /// SSC1
11	135	15,1	C20 Beam /// SSC1
12	86,6	-15,1	C20 Beam /// SSC1
13	78,7	0	C20 Beam /// SSC1
14	86,6	15,1	C20 Beam /// SSC1
15	131,7	-9,7	C20 Beam /// SSC1
16	131,7	0	C20 Beam /// SSC1
17	131,7	9,7	C20 Beam /// SSC1
18	131,7	-9,7	C20 Beam /// SSC1
19	263,4	0	C20 Beam /// SSC1
20	131,7	9,7	C20 Beam /// SSC1
21	131,7	-9,7	C20 Beam /// SSC1
22	131,7	0	C20 Beam /// SSC1
23	131,7	9,7	C20 Beam /// SSC1

5. SEISMIC PERFORMANCE ASSESSMENT BEFORE AND AFTER STRENGTHENING

In this section, the previously determined suitable nonlinear analysis method for the structural system, the pushover analysis, will be performed. The nonlinear definitions required for the pushover analysis were provided in the previous section, and this part will now proceed with the implementation stages of the pushover analysis. In addition to pushover analysis, time history analysis will also be performed to provide a more comprehensive assessment of the seismic behaviour.

Pushover analysis is a nonlinear static method used to evaluate the seismic performance of buildings. In this analysis, horizontal loads are gradually applied to the structure while vertical loads remain constant. As the lateral loads increase, some structural elements start to yield, and plastic hinges form. This helps engineers understand how the structure behaves during an earthquake, including when and where damage might occur. The results are shown with a capacity curve, which gives an idea about the strength and deformation capacity of the building.

Pushover analysis is especially useful for checking whether existing buildings meet the performance levels required by seismic codes like TBDY 2018.

5.1 Generation of Pushover Curves Based on Target Displacement

This section determines the expected displacement demand of the building by considering its spectral response to seismic effects. This analysis is conducted within the framework of the procedures outlined in Annex 5B of the Turkish Seismic Code **[TBDY-2018]**

To examine the relationship between the base shear force and the roof displacement, graphs have been generated in both directions. These graphs provide a clearer insight into the building's lateral behaviour throughout the pushover process. **[Figure 5.1]**
[Figure 5.2]

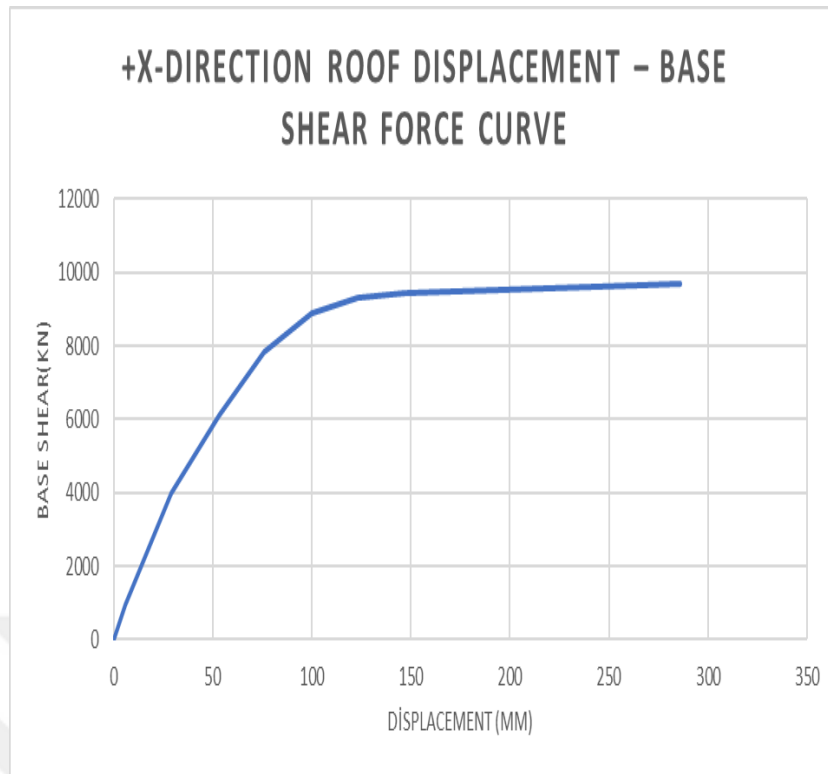


Figure 5.1: X-direction base shear- displacement

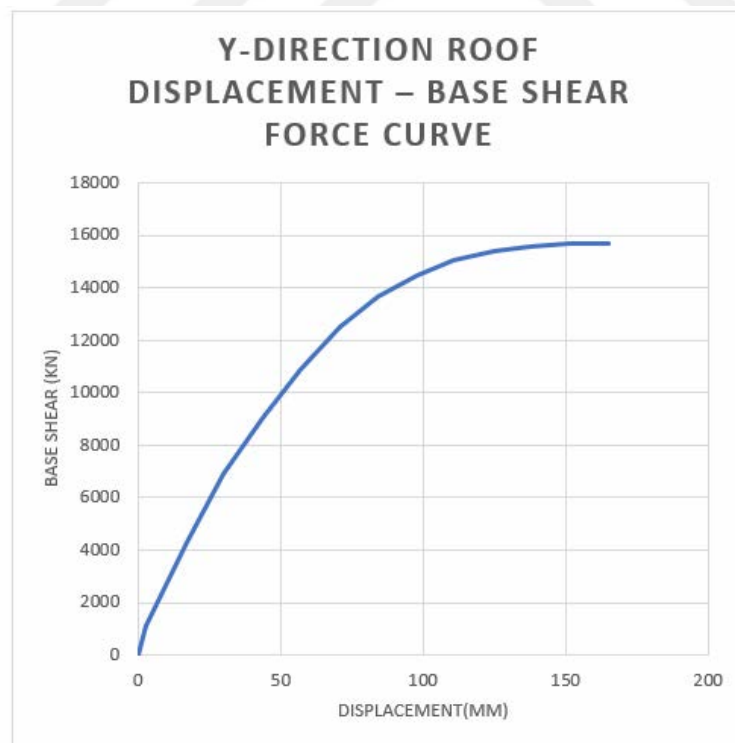


Figure 5.2 Y-direction base shear- displacement.

After determining the roof displacement, a modal capacity diagram is obtained to proceed with the pushover analysis. At this stage, various parameters are used to determine the target displacement. First, T_b (DD1) represents the characteristic period for the DD1 level earthquake and defines the upper limit of the short-period range in the design spectrum. The structural period (T_x) refers to the natural vibration period of the building, which reflects its dynamic behavior. The corresponding $S_{ae}(T_x)$ value indicates the spectral acceleration at this period, representing the seismic demand on the structure.

After obtaining the modal capacity parameters, the pushover analysis spectra for both directions were generated and are presented below for comparison and evaluation.

[Figure 5.3]

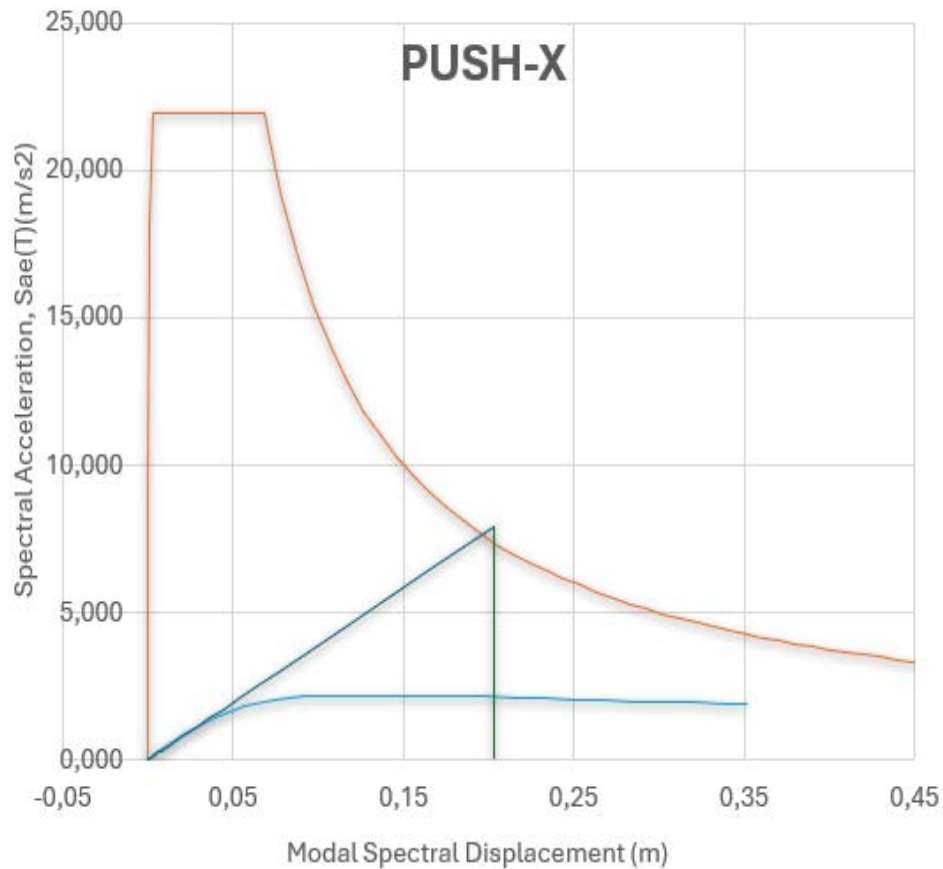


Figure 5.3: Acceleration–displacement – Modal capacity diagram-X direction

The necessary calculations have been carried out in accordance with the procedures specified in the seismic code, and a summary of the results for the X-direction is provided below. [Table 5.1]

Table 5.1 Summary of pushover-X

Tb (DD1)	0,355010661
Structural Period, Tx	1,046
CR $\geq$ 1	1,00
New Target Disp. (CR*Sde)	286,56

The same procedures have been applied for the Y-direction as well, and both the corresponding graph and detailed information have been provided to illustrate the results clearly. [Figure 5.4] [Table 5.2]

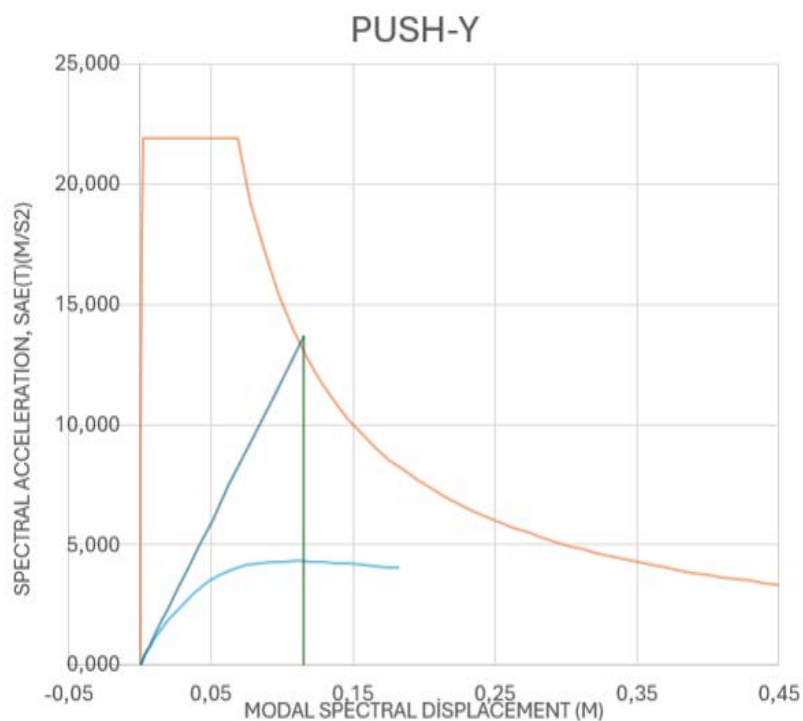


Figure 5.4: Acceleration–displacement – Modal capacity diagram-Y direction

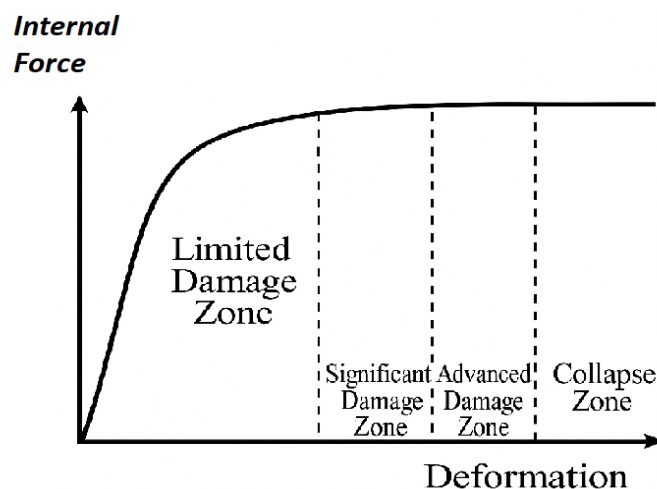
Table 5.2 Summary of pushover-Y

Tb (DD1)	0,355010661
Structural Period, Ty	0,779
CR $\geq$ 1	1,00
New Target Disp. (CR*Sde)	18,60

5.2 Performance Levels of Frame Elements (Beam&Column)

In this section, the number of damaged structural elements was identified based on the target displacement level. The assessment was carried out separately for each story and each element. The results are presented in separate tables for columns and beams. No significant damage was observed in the shear wall elements.

Structural elements that do not reach the SH threshold are classified within the Limited Damage Zone. When the demand falls between SH and KH, the element is considered to be in the Significant Damage Zone. If the response lies between KH and GÖ, it indicates the Advanced Damage Zone. Exceeding the GÖ threshold corresponds to the Collapse Zone. [Figure 5.5] [TBDY-2018]

**Figure 5.5:** Damage zones of structural sections

Push-X Damage Levels – Columns & Beams

The damage states observed in the beam elements at the target displacement level as a result of the Push-X analysis are presented below. For each story, the percentage distribution of beam elements across different damage levels was evaluated separately. The distribution of hinge states for the 1st, 2nd, and 3rd stories is presented in Figures [Figure 5.6], [Figure 5.7], and [Figure 5.8], respectively.

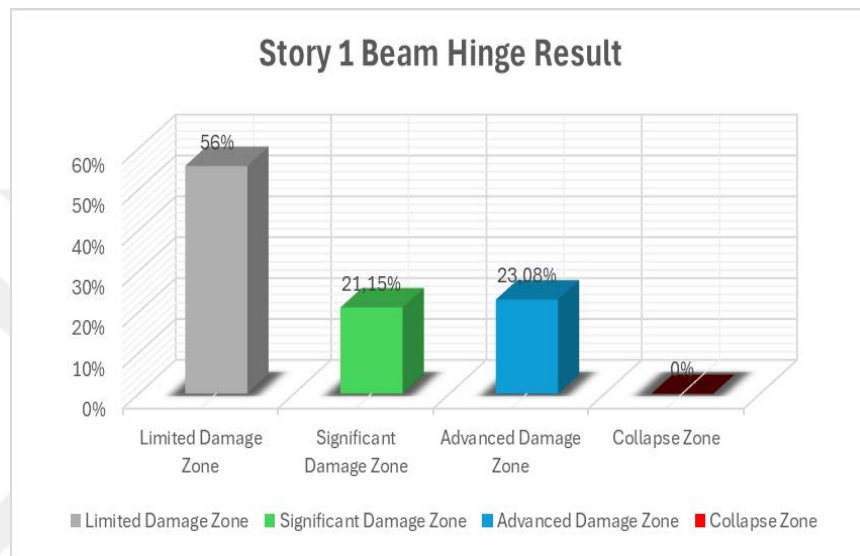


Figure 5.6: 1. Story beam hinge result push-X

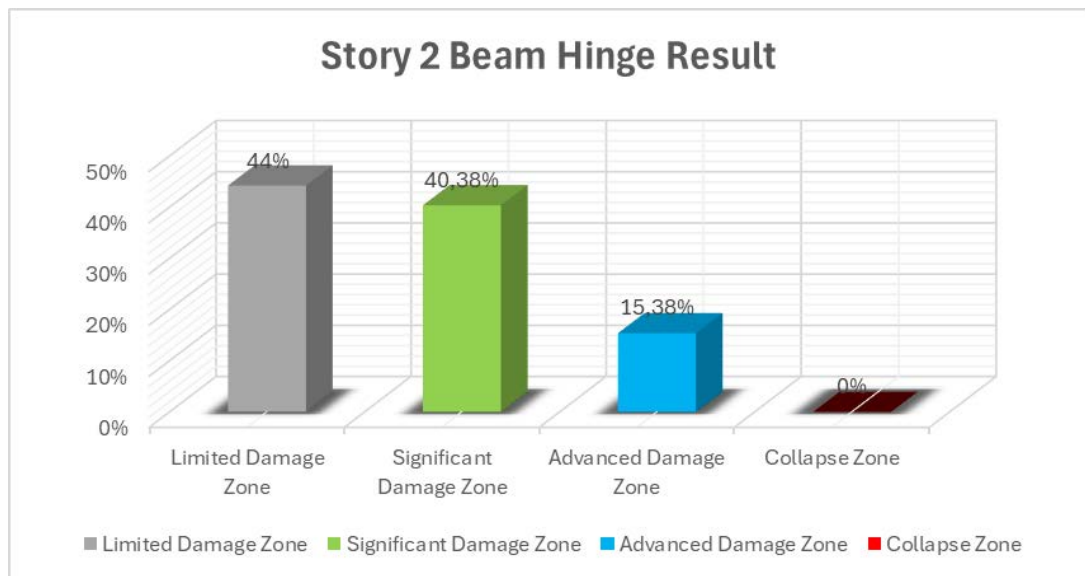


Figure 5.7: 2. Story beam hinge result push-X

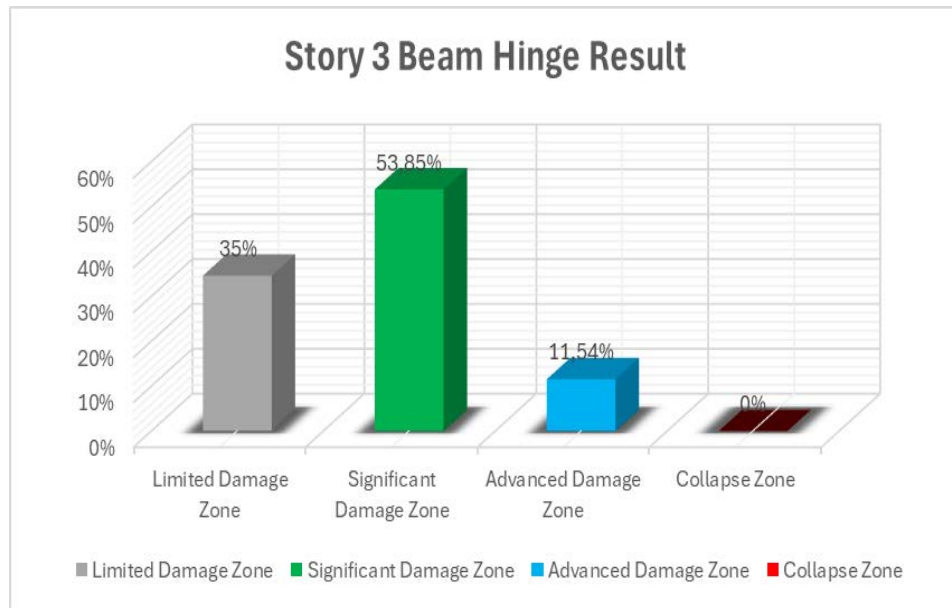


Figure 5.8: 3. Story beam hinge result push-X

The percentage distribution of column hinge states in the 1st story — the only story where significant damage was observed — is presented in [Figure 5.9].

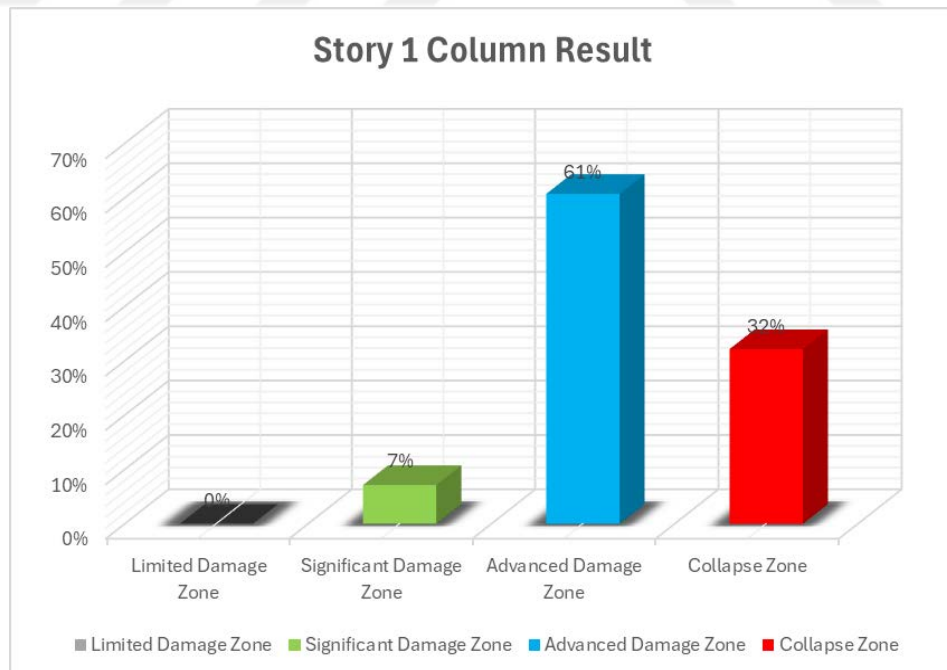


Figure 5.9 1. Story column hinge state push-X.

Push-Y Damage Levels – Columns & Beams

The Push-Y analysis revealed the hinge states in the beam elements at the target displacement level. The percentage distribution of elements across various damage levels was assessed independently for each story. The hinge state distributions for the beams in the 1st, 2nd, and 3rd stories are presented in Figures [Figure 5.10], [Figure 5.11], and [Figure 5.12], respectively. Beam Hinge State Push-Y

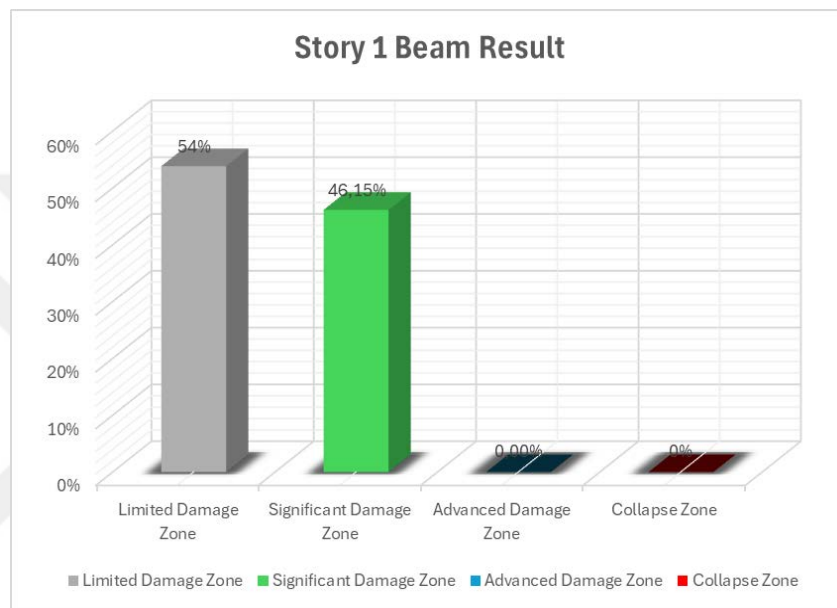


Figure 5.10: 1. Story beam hinge result push-Y

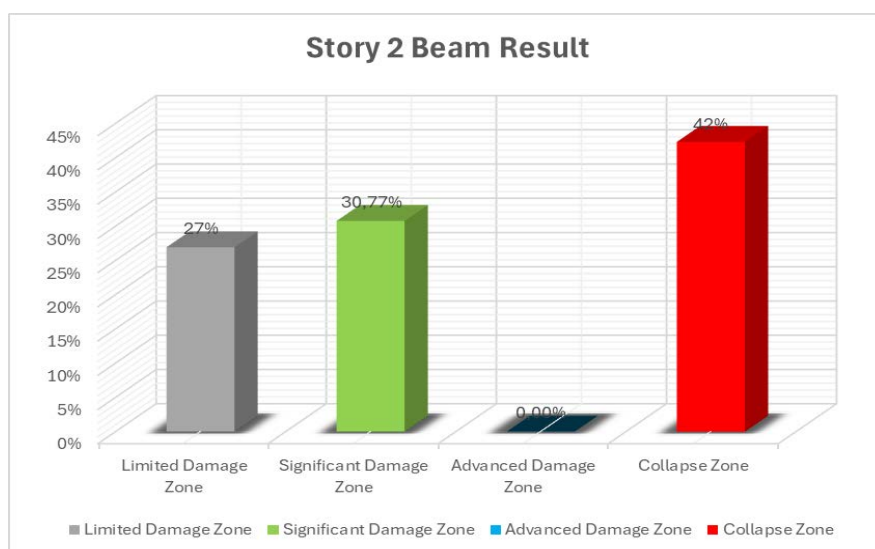


Figure 5.11: 2. story beam hinge result push-Y

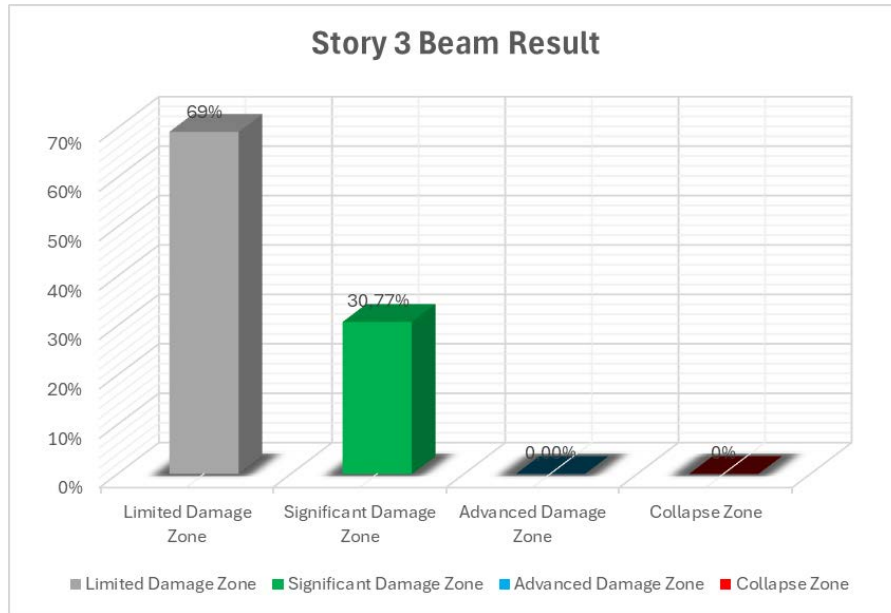


Figure 5.12: 3. story beam hinge result push-Y

The hinge states for the column elements are presented in **[Figure 13]**. Column damage was observed only in the 1st story, and even then, at a minimal level. This is illustrated in the figure. No damage was detected in the other stories, as all columns remained within the code-defined limit states.

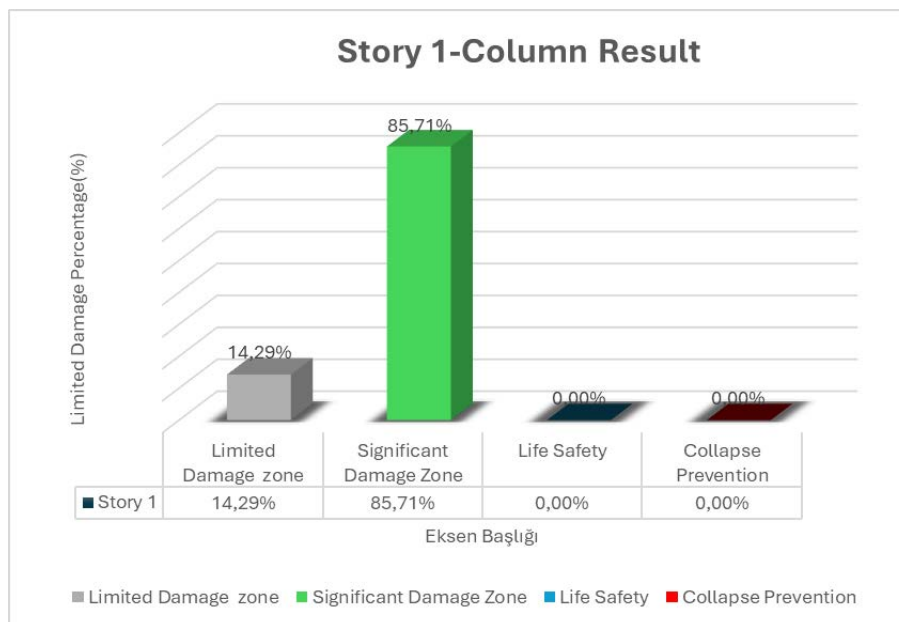


Figure 5.13: 1. Story column hinge state push-Y

5.3 Story Drift Ratio Values – X and Y Directions

Story drift ratio, which is one of the critical parameters in earthquake engineering, has been evaluated separately for each axis. The values resulting from the pushover analysis in the X-direction are clearly presented in **[Figure 5.14]**.

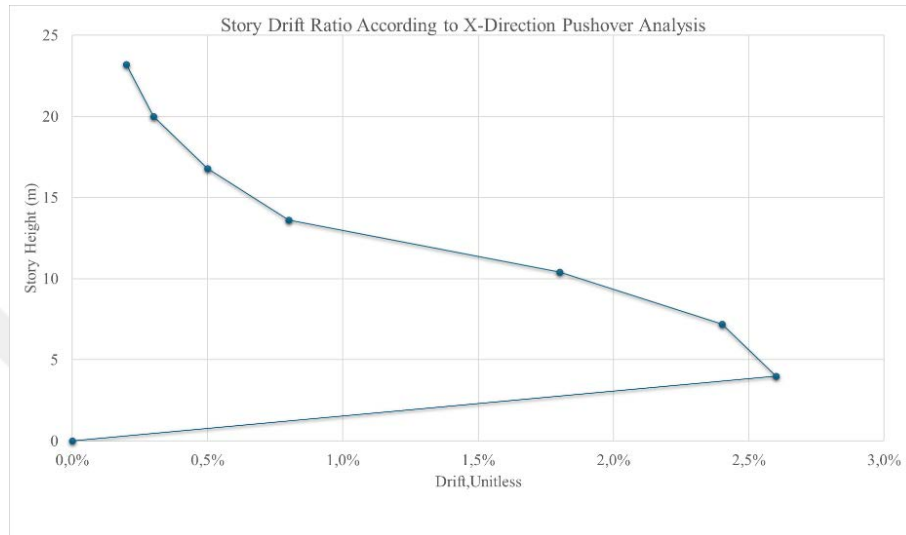


Figure 5.14 Story drift ratio according to X-direction pushover analysis

In the pushover analysis, high story drift ratios were observed in the lower stories due to plastic deformations in the shear wall elements. For the Y-direction, the story drift ratios are presented in **[Figure 5.15]**, expressed as percentages. The story drift ratio values observed here were comparatively lower than those in the X-direction.

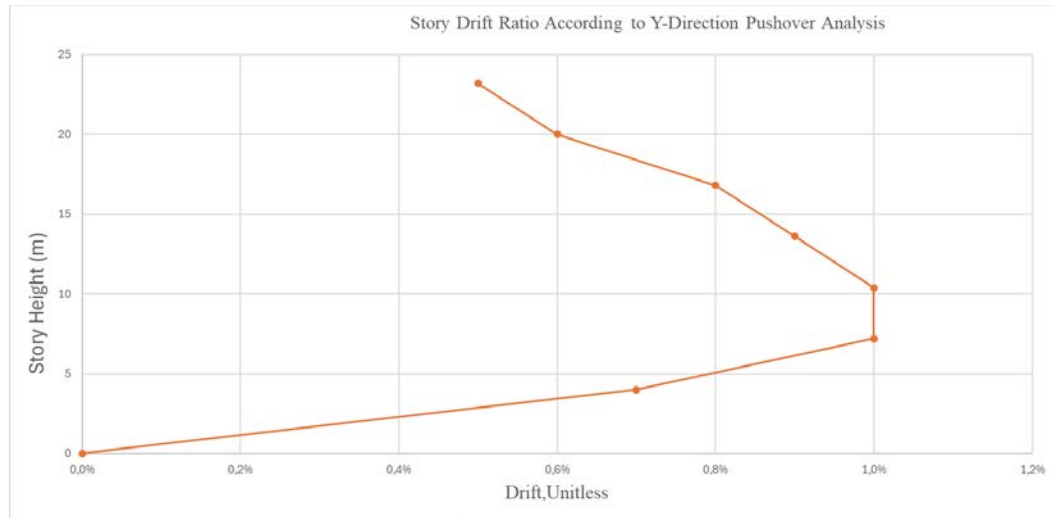


Figure 5.15: Story drift ratio according to Y-direction pushover analysis

5.4 Assessment of the Necessity for Global Strengthening

As detailed in the tables, significant damage levels were observed in the columns of the first story along the X-direction. This story has a height of 4 meters, reflecting the general conditions of existing and newly constructed buildings in Turkey. When performing the pushover analysis in the X-direction, noticeable damage occurred predominantly in beams within the first three stories, while column damage in upper stories was not significant. No significant damage was observed in the shear wall elements.

In the Y-direction, the presence of shear walls providing resistance against seismic forces resulted in substantially lower damage levels compared to the X-direction.

When examining the story drift ratio values, it was observed that the values reached in the first three stories were significantly high in the X-direction, whereas the building did not exhibit such behavior in the Y-direction.

Based on the damage assessment conducted according to the limits defined in TBDY 2018, a global retrofit was deemed necessary. However, rather than a bidirectional strengthening, a single-direction strengthening along the X-direction was found adequate. It was concluded that the resistance provided by shear wall elements in the Y-direction is sufficient, and thus the strengthening method aims to enhance seismic performance predominantly in the X-direction.

5.5 Definition of the Selected Retrofitting Method: (BRBs)

In this section, the reason for selecting the BRB system and the definition of the BRB element will be explained. As a global retrofit strategy, this hospital structure could have been strengthened using various scenarios. However, within the scope of this thesis, it was intended to strengthen the building along the outer axes to allow continued use of the interior spaces. Additionally, instead of a continuous retrofit along the entire axis, a more segmented approach was adopted to remain within economic constraints. In line with this strategy, the BRB (Buckling-Restrained Brace) retrofit method was deemed appropriate and implemented in the building

BRBs are defined to carry only axial loads. In this study, the CoreBrace catalogue was used as a reference, and calculations were made accordingly. First, the following formulas were used to determine the appropriate cross-section.

$$F = k \times u \quad (5.1)$$

$$K = \frac{E \times A}{L} \quad (5.2)$$

$$P_y = \frac{E \times A}{L} \times u \quad (5.3)$$

$$0,9 \times f_{y_{sc}} \left(\frac{N}{mm^2} \right) \times A_{sc} (mm^2) = \frac{E \times A_{sc}}{L} \times u \quad (5.4)$$

We obtain u from the equation where $f_{y_{sc}}$ is taken from the CoreBrace BRB catalogue. This value represents the yield strength and is given as 262 MPa. A_{sc} refers to the cross-sectional area of the BRB, E is the elastic modulus, and L is the brace length, which varies depending on the span length. Therefore, two different BRBs were modelled for the upper and lower storeys.

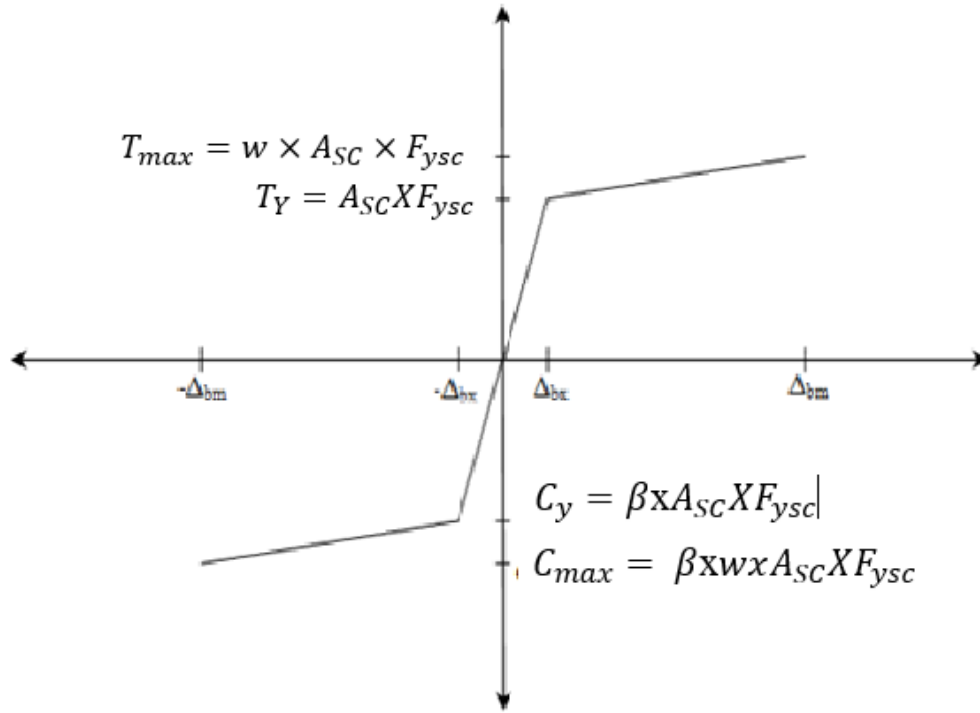


Figure 5.16 Axial force–displacement diagram of the BRB [Doran et al,2013]

The strain-hardening axial compression and tension forces of the BRB element were calculated using the relevant formulas. The parameters w and β were derived from the values provided in the CoreBrace catalogue. **[Figure 5.16]**

$$T_{max} = w \times A_{SC} \times F_{ysc} \quad (5.5)$$

$$T_Y = A_{SC} \times F_{ysc} \quad (5.6)$$

$$C_Y = \beta \times A_{SC} \times F_{ysc} \quad (5.7)$$

$$C_{max} = \beta \times w \times A_{SC} \times F_{ysc} \quad (5.8)$$

Two separate BRBs were defined due to the first story having a height of 4 meters, while the other stories are 3.2 meters high. For the BRB model used, $w = 1.35$ and $\beta = 1.15$ were provided. The properties of the BRB defined for the first story are presented

in the table. The values calculated based on the formulas and the material property data are provided in [Table 5.5] and [Table 5.6].

Table 5.3: Properties of the first story BRB

$A_{sc} (in^2)$	3
$A_{sc}(cm^2)$	1935
F_y	262
f_i	0,9
P_y	456,27
L_x	5,00
L_y	4,00
L_{br}	6,40
E	200000
U_y	8,4
U_u	111,6
R_y	1,20
β	1,15
ω	1,35

$A_{sc} (in^2)$ =cross-sectional area of the BRB (in^2)

$A_{sc} (cm^2)$ = cross-sectional area of the BRB (cm^2)

L_x = Length of the axis to be aligned in the X direction

L_y = Length of the axis to be aligned in the Y direction

L_{br} = Total Length of the BRB

E = Modulus of Elasticity (MPa)

F_y = Yield strength of BRB

P_y = Force(kN)

U_y = Yield Displacement (mm)

U_u = Ultimate Displacement (mm)

R_y = Material Overstrength Factor

β = Strain Hardening Ratio

w = Overstrength Factor

As shown in the formulas above and based on the values obtained from the catalogue, the calculated limit values for the BRB model are presented in [Table 5.6].

Table 5.4 Limit values of the BRB.

	m	P(Kn)	kN
Uu	-0,112	Pu	-850,037
Uy	-0,008	Py	-456,273
	0,000		0
Uy	0,008	Py	456,273
Uu	0,112	Pu	739,162

The BRB behavior obtained based on the calculated values is presented in [Figure 5.17].

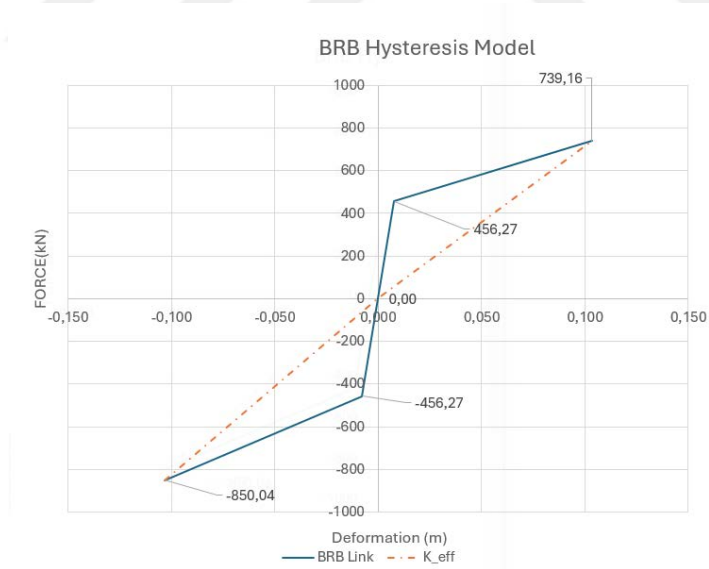


Figure 5.17: BRB hysteresis model

$$\frac{1}{K_{eff}} = \frac{1}{k_1} + \frac{1}{k_2}$$

(5.9)

First floor: The effective stiffness K_{eff} of the first-storey BRBs was calculated.

$$K_{eff} = 7619,435 \text{ kN/m}$$

A different BRB was defined for the upper stories, and its properties were calculated using the same formulas. The relevant values were obtained from the catalogue, and the results are presented in [Tables 5.7] and [Table 5.8].

Table 5.5 Properties of the BRB used in the upper stories

$A_{sc} (in^2)$	3
$A_{sc} (cm^2)$	1935
F_y	262
f_i	0,9
P_y	456,27
L_x	5,00
L_y	3,20
L_{br}	5,94
E	200000
U_y	7,8
U_u	103,4
R_y	1,20
β	1,15
ω	1,35

Table 5.6: Limit Values of the BRB for Upper Stories

	m	P(kn)	kN
Uu	-0,103	Pu	-850,037
Uy	-0,008	Py	-456,273
	0,000		0
Uy	0,008	Py	456,273
Uu	0,103	Pu	739,162

First floor: The effective stiffness K_{eff} of the first-storey BRBs was calculated.

$$K_{eff} = 8218,58 \text{ kN/m}$$

5.6 Application of the BRB to the Structural Element

The BRB elements were implemented in the structural system as shown in [Figure 5.18], and their use was deemed necessary only in the first three stories due to the concentration of damage at these levels. By limiting the application to these stories, the aim was to remain economically feasible while also assessing whether this innovative retrofit method could provide sufficient additional lateral stiffness to improve the building's overall seismic performance and meet the requirements set by the seismic code.

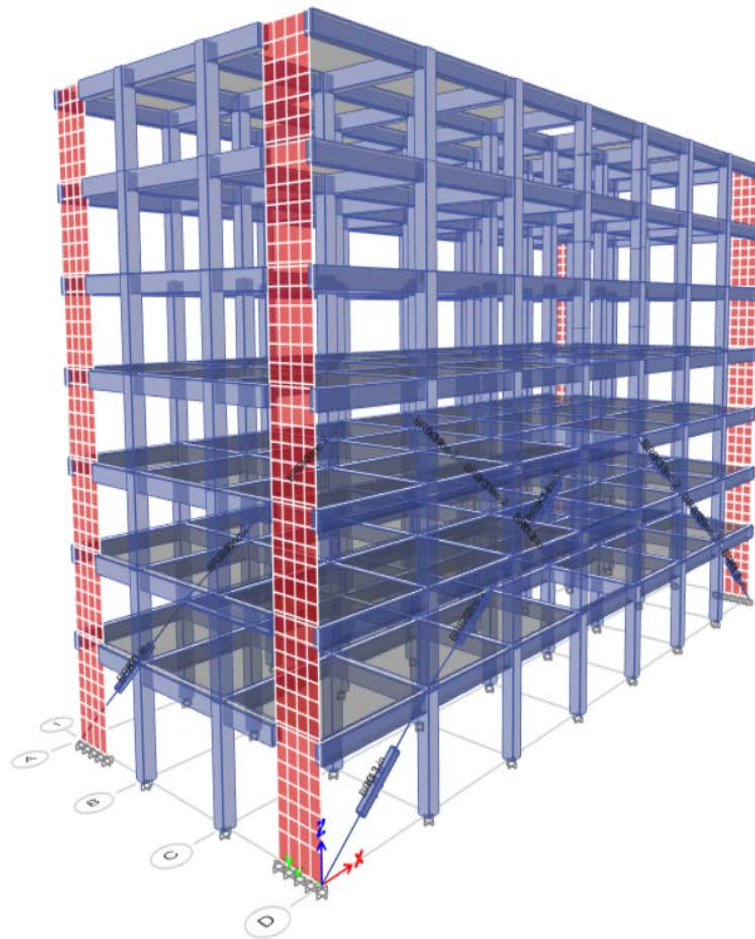


Figure 5.18: 3D view of the structure with BRBs

Additionally, instead of placing BRBs between every axis on the selected three stories, it was deemed appropriate to install them only in the X-direction, as explained in the

previous section. In the first story, BRBs were applied between axes 1–2 and 7–8 along the outermost axes A and D, starting from the building's outer edges. In the second story, BRBs were placed between axes 2–3 and 6–7. Finally, in the third story—which is the last story where BRBs were implemented—single-direction BRBs were installed between axes 3–4 and 5–6. A sectional view of the BRB placement between the axes is presented in [Figure 5.19].

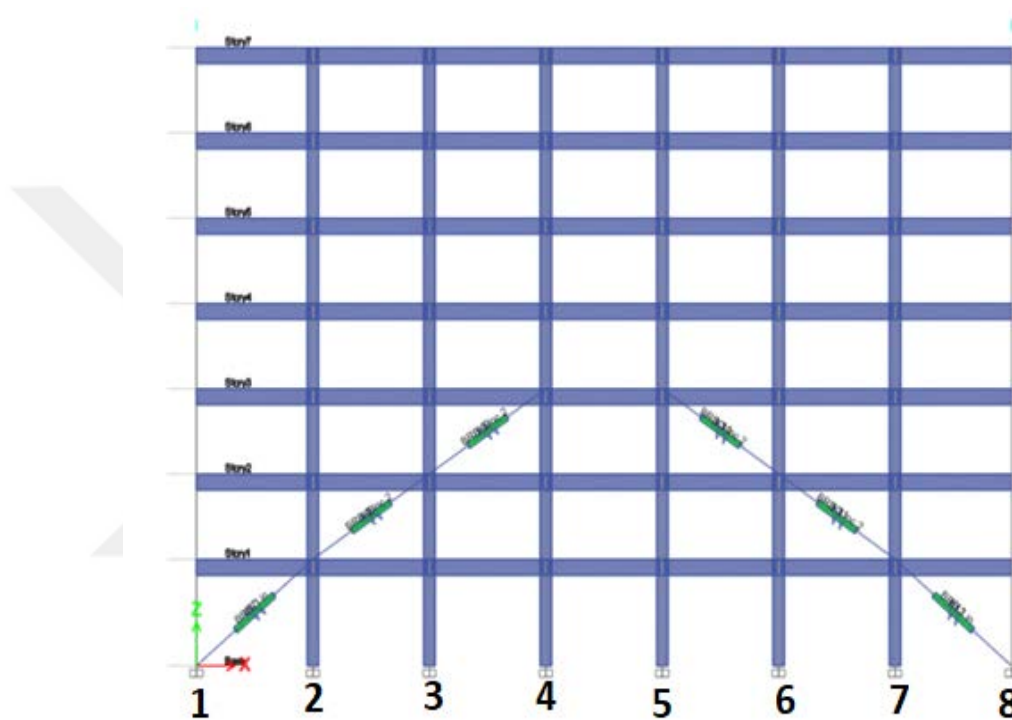


Figure 5.19: BRB placement detail on axis-D

During the placement of BRBs, three main criteria were considered:

1. **Ensuring uninterrupted building use:** BRBs were installed in a way that did not interfere with the interior functionality, by placing them along the exterior axes.
2. **Developing a cost-effective solution:** The number of BRBs was limited by applying them only on the necessary stories and between selected axes, reducing material and installation costs.

3. **Providing safe distribution of additional axial forces:** One of the key concerns was the potential increase in axial loads, particularly in the first-story columns, due to the addition of BRBs. To prevent this undesired outcome, careful planning of BRB placement was carried out from the beginning to ensure a safe distribution of these additional forces.

In addition to these features, the arrangement of BRBs from the outermost axes inward provided a visually balanced and aesthetically pleasing appearance to the structure. A 3D representation of this layout is presented in [Figure 5.20].

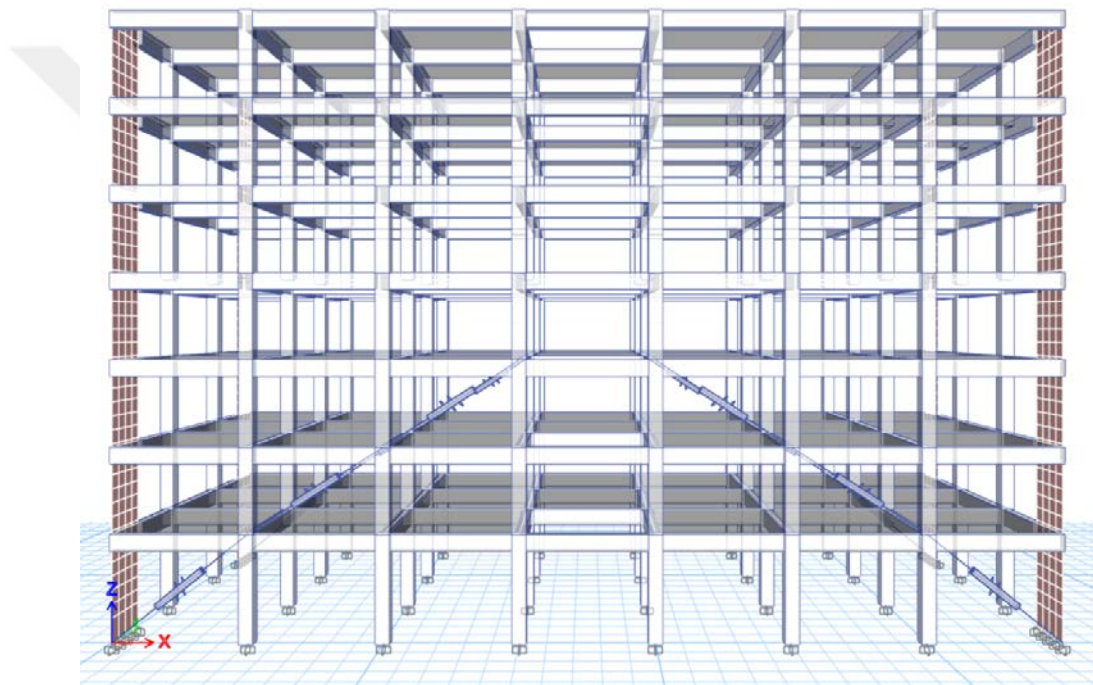


Figure 5.20 Front elevation

5.7 Pushover Analysis After Retrofitting with BRBs

After the BRB element was defined and implemented in the structure as described in the previous section, a pushover analysis was conducted in the X direction. Since the structure required strengthening only in one direction, the retrofitting was applied accordingly. When the structure was re-pushed to the target displacement, the maximum base displacement was observed to decrease from 286 mm to 160 mm. This indicates that the addition of BRBs effectively reduced the peak displacement in the X

direction. The pushover curve obtained by pushing the structure to the target displacement can be seen in [Figure 5.21].

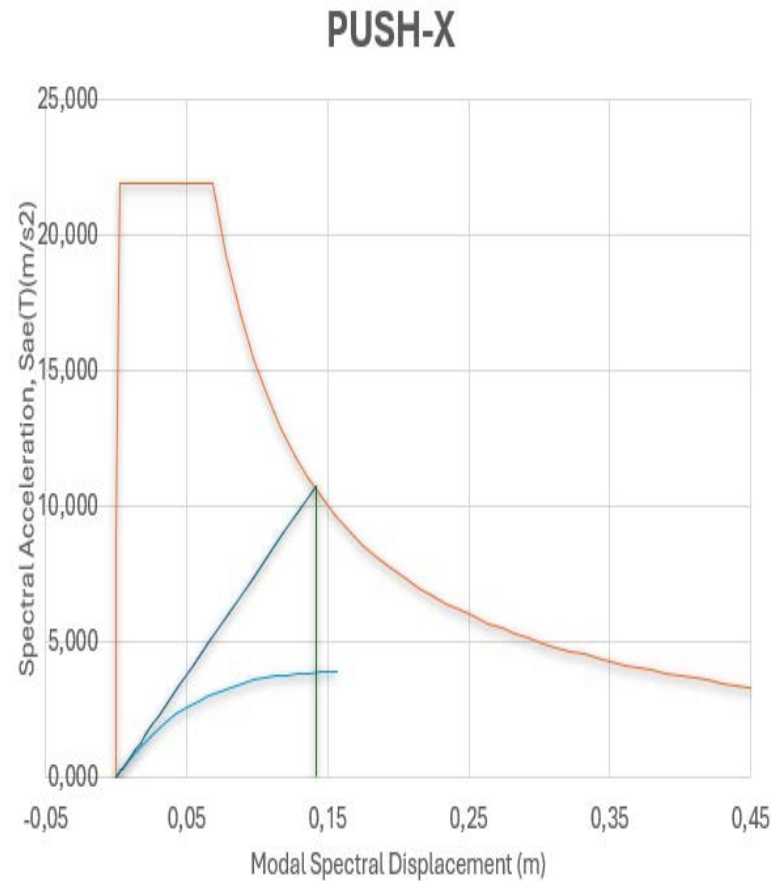


Figure 5.21 Sae– Modal capacity Modal Capacity (with BRBs)

As a result of the pushover analysis, the performance levels of the structural elements were examined individually. In the first story, 93% of the columns remained within the Limited Damage Zone, and compared to the initial condition, no elements were observed to have reached collapse, advanced damage, or failure. The strengthening method has achieved the performance targets required by the code. [Figure 5.22]

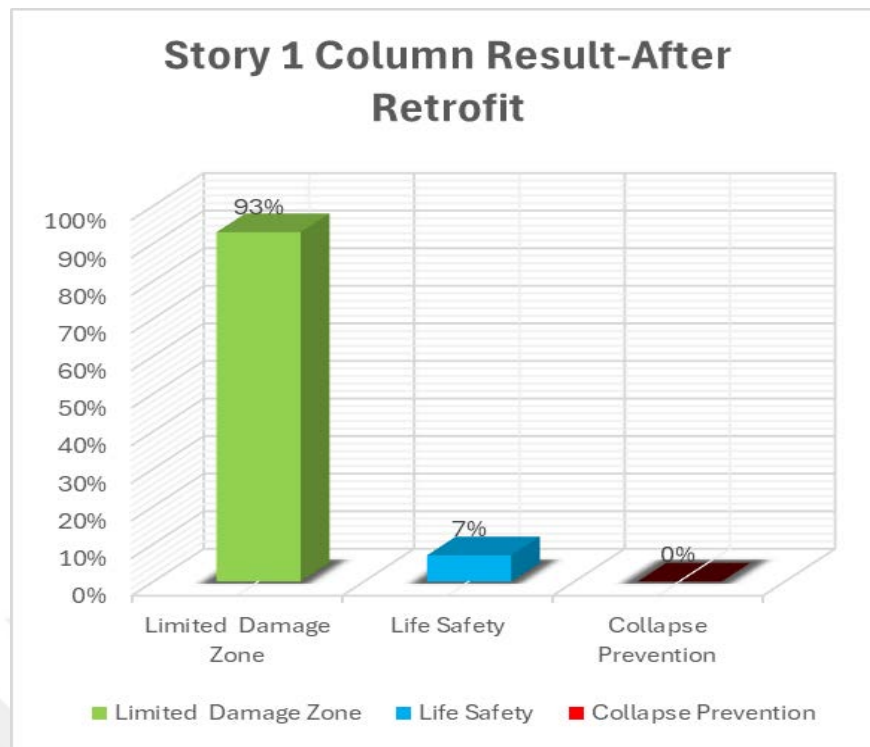


Figure 5.22: Column result after retrofit

Similarly, the hinge states of the beams in the first three stories—where strengthening was applied and damage was previously observed—were examined, as shown in **[Figure 5.23]**. According to the results, all beams remained within the code-specified limits after strengthening. No damage was observed in the upper stories.

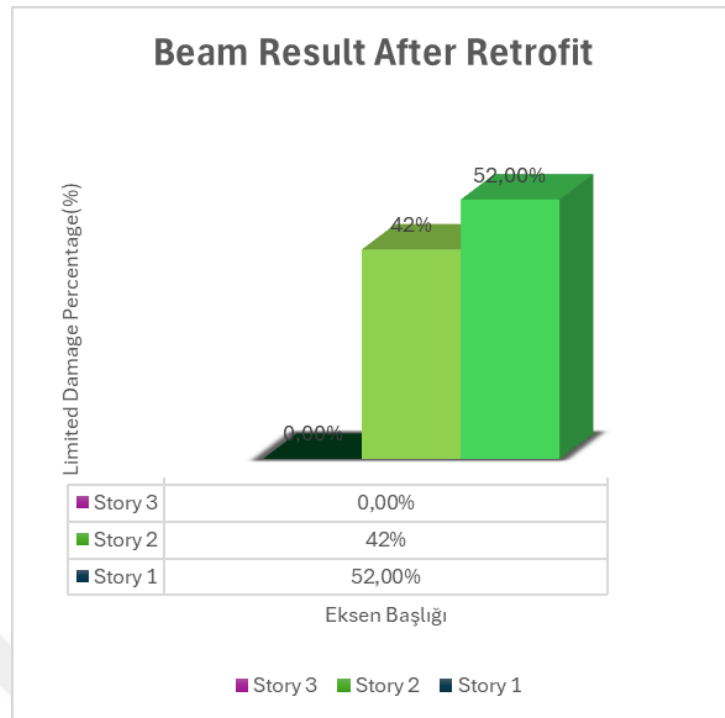


Figure 5.23: Post-retrofit beam results

[Figure 5.24] provides detailed story drift ratio values following the implementation of the strengthening method. These results demonstrate the effect on structural displacements and indicate that the story drift ratios remained within the limits defined by the seismic code.

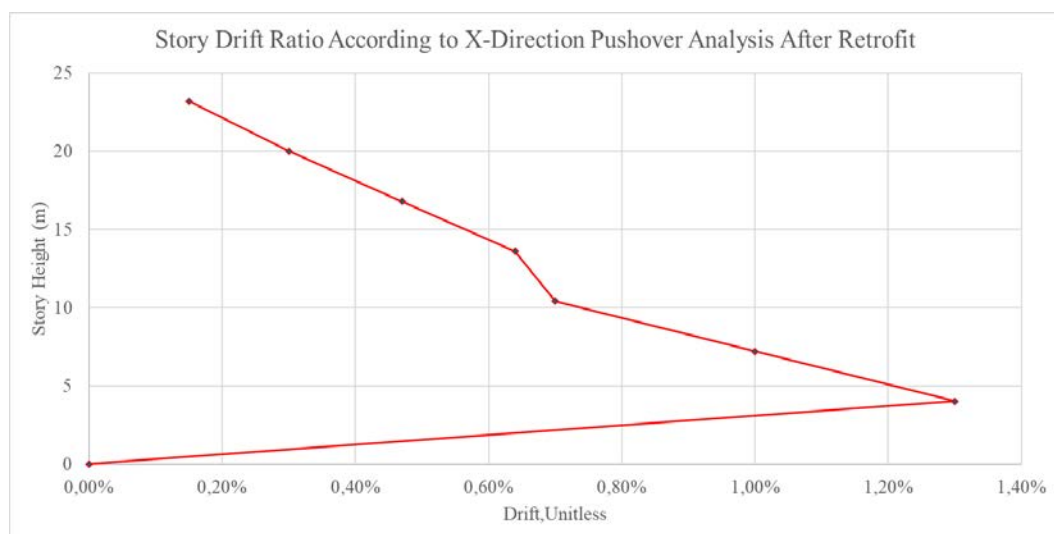


Figure 5.24: Story Drift Ratio from X-Direction Pushover After Retrofit

Comparison of Plastic Hinge Formation Before and After Retrofitting

The plastic hinge states in Sections A and D, where BRBs were added after retrofitting, are of particular importance. Before the installation of BRBs, A and D axes were the most critical regions in terms of damage concentration. Therefore, the hinge states in these axes prior to retrofitting are presented to clearly illustrate the effects of BRBs in the corresponding areas.

The hinge distribution and damage states of Axis A before retrofitting are presented. [Figure 5.25].

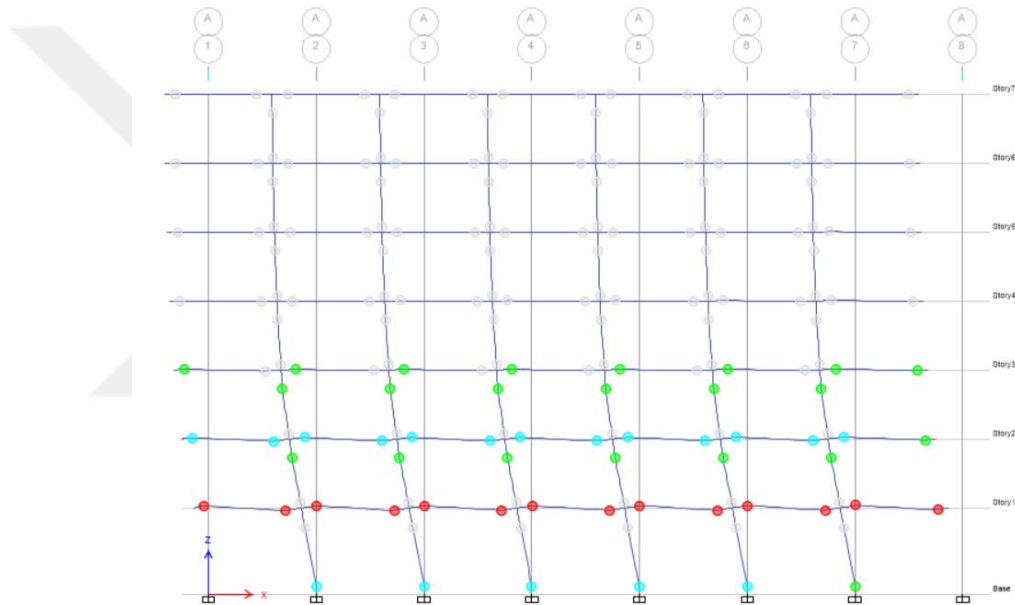


Figure 5.25: Hinge states of axis A before retrofitting

The condition of Axis A after the addition of BRBs is illustrated below. This representation highlights the changes in hinge distribution and damage states, allowing a clear assessment of the improvements achieved through the retrofitting process. [Figure 5.26]

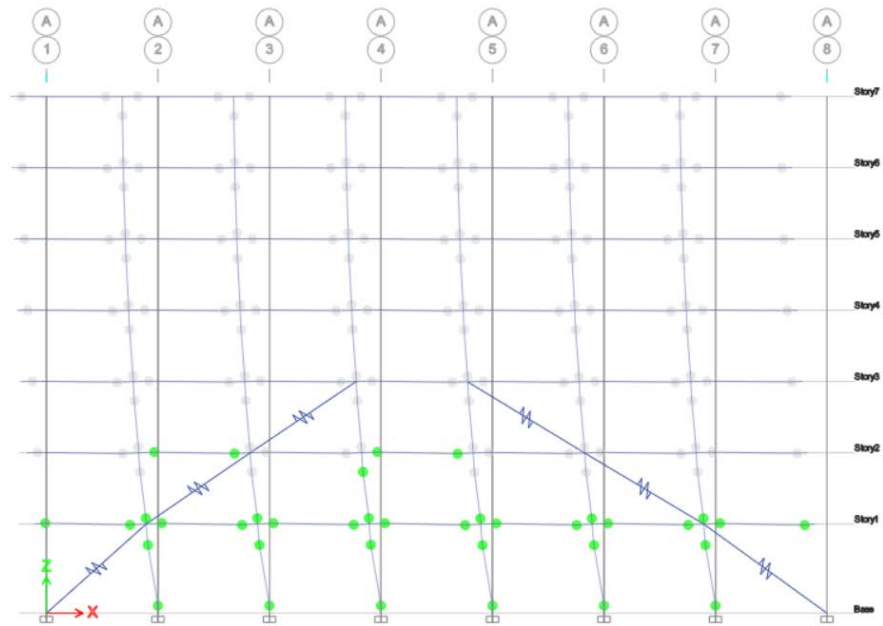


Figure 5.26 Hinge states of axis A After retrofitting

Similarly, a detailed evaluation was also deemed appropriate for Axis D, as it is the outermost axis where BRBs were installed. This allows for a clear understanding of how the initial damage conditions were addressed. [Figure 5.27] presents the condition of the structure before retrofitting.

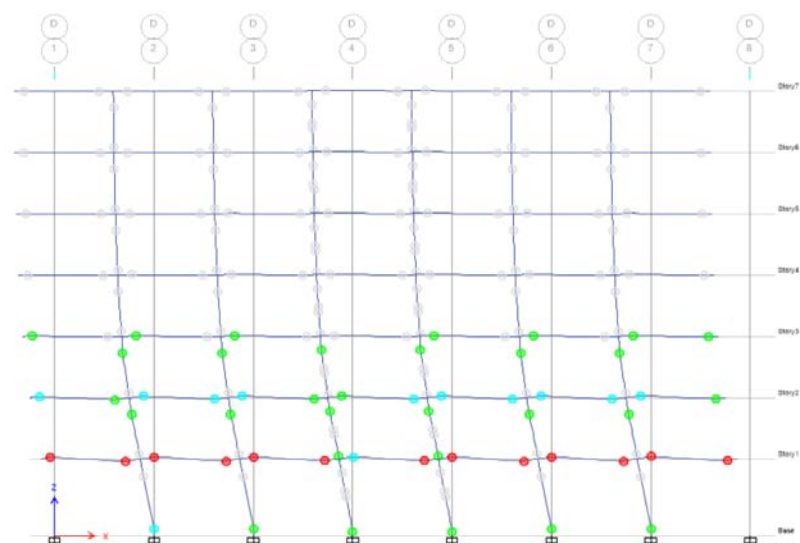


Figure 5.27: Hinge states of axis D before retrofitting

The final condition of Axis D after retrofitting is presented in [5.28].

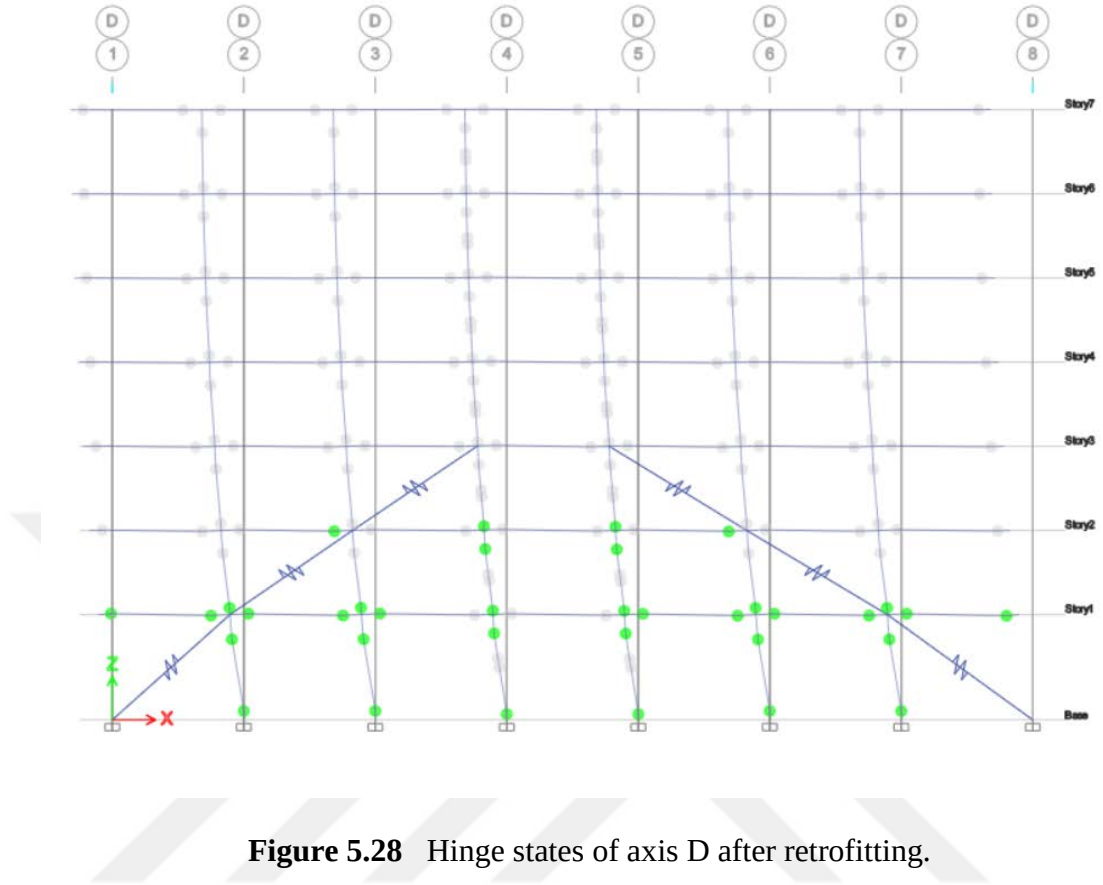


Figure 5.28 Hinge states of axis D after retrofitting.

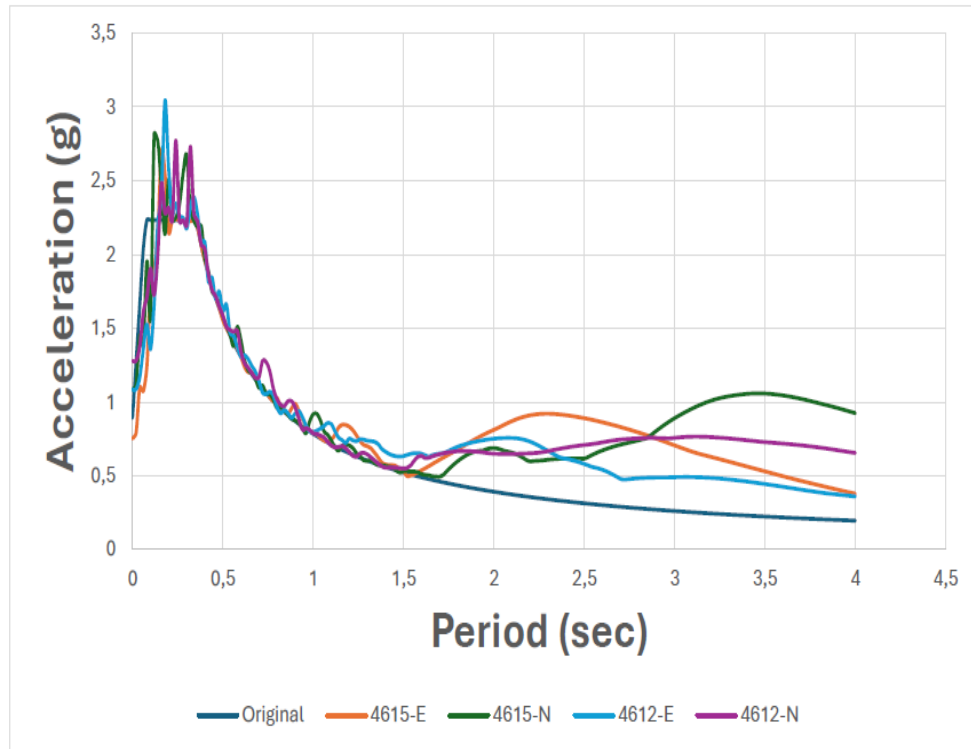
5.8 Time History Analysis

Within the scope of this study, two ground motion records were selected from the devastating earthquakes that struck the Pazarcık and Elbistan districts in Türkiye on February 6, 2023. These earthquakes caused significant damage across a wide region and were considered highly representative in terms of seismic demand. Therefore, the selected records were evaluated in detail to better understand the structural response under realistic earthquake scenarios. The detailed characteristics of these records are presented in [Table 5.10].

Table 5.7: Pazarcık Elbistan ground motion records

EventDate	Station	Location	Latitude	Longitude	Depth	Type	Magnitude	PGA
202302060117	4615	Pazarcık-Kahramanmaraş	37,288	37,043	8,6	MW	7,7	582
202302061024	4612	Elbistan-Kahramanmaraş	38,089	37,239	7	MW	7,6	635

For each earthquake record, the east–west and north–south components were obtained from the official database of the Disaster and Emergency Management Authority (AFAD) via the website tadas.afad.gov.tr. These records were processed using SeismoSignal for baseline correction and further adjusted using SeismoMatch to match the site-specific DD-1 design spectrum. The resulting spectra are illustrated in [Figure 5.29].

**Figure 5.29:** Spectral matching of ground motions to DD-1 design spectrum

For both the Pazarcık and Elbistan earthquakes, two components (east–west and north–south) were considered for each record, and the results are illustrated in [Figure 5.30], [Figure 5.31], [Figure 5.32], [Figure 5.33].

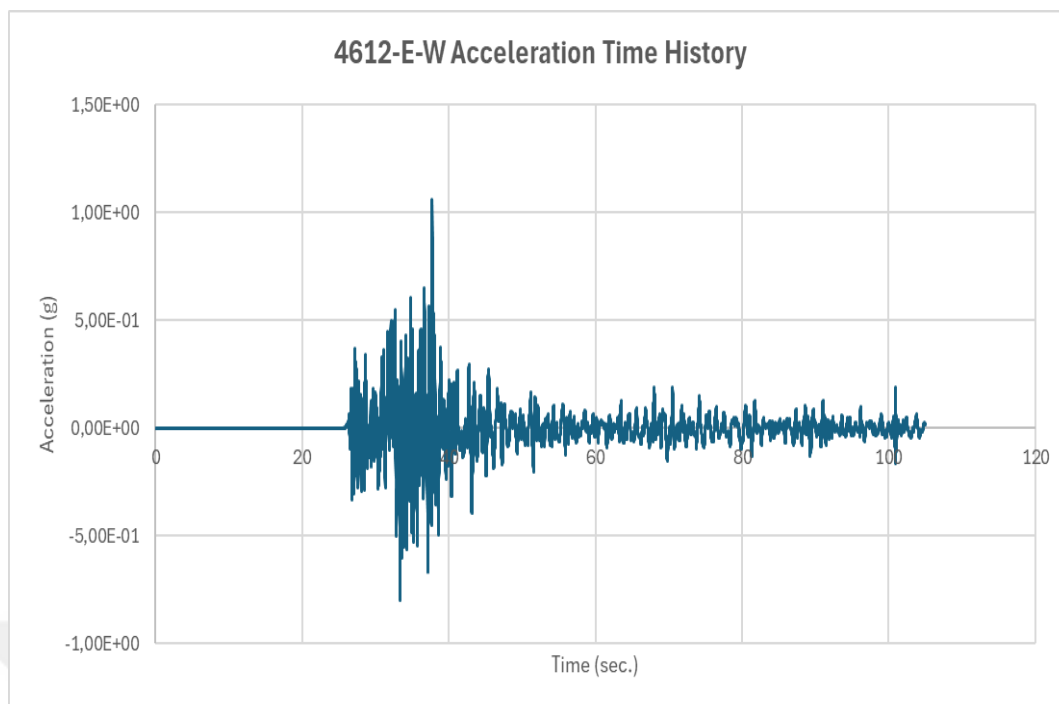


Figure 5.30: 4612- E-W acceleration time history

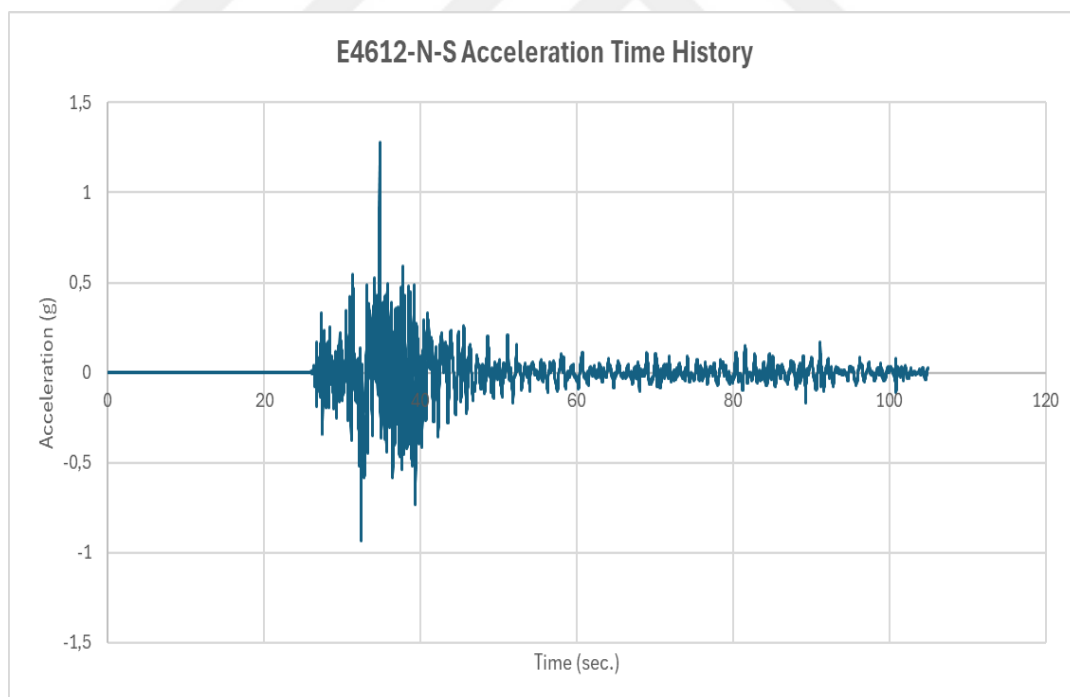


Figure 5.31: 4612- N-S acceleration time history

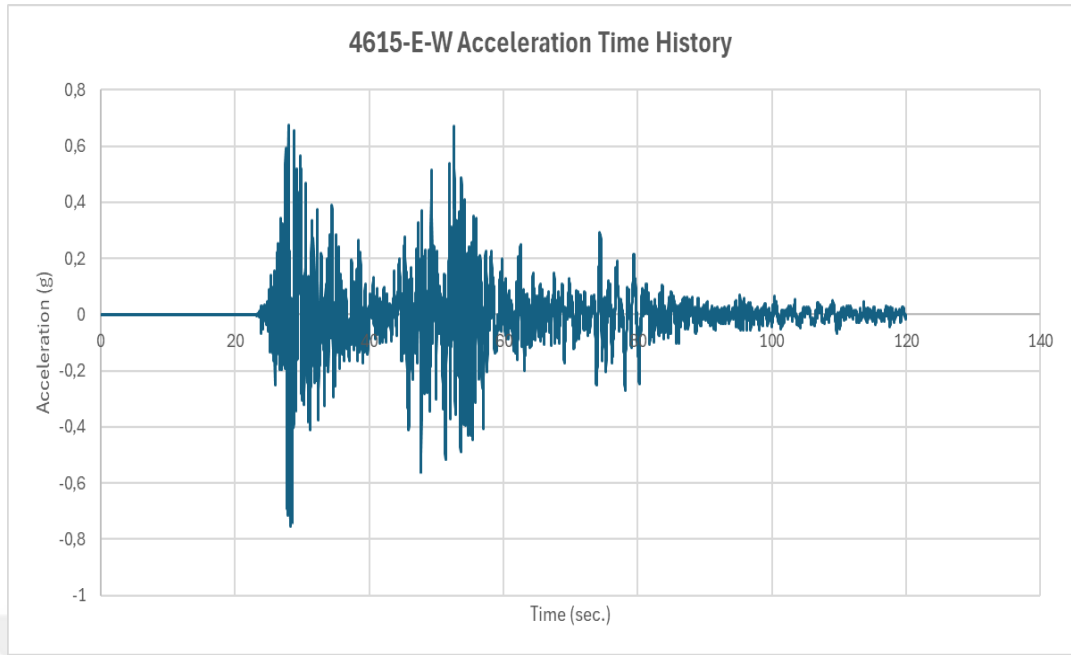


Figure 5.32; 4615- E-W acceleration time history

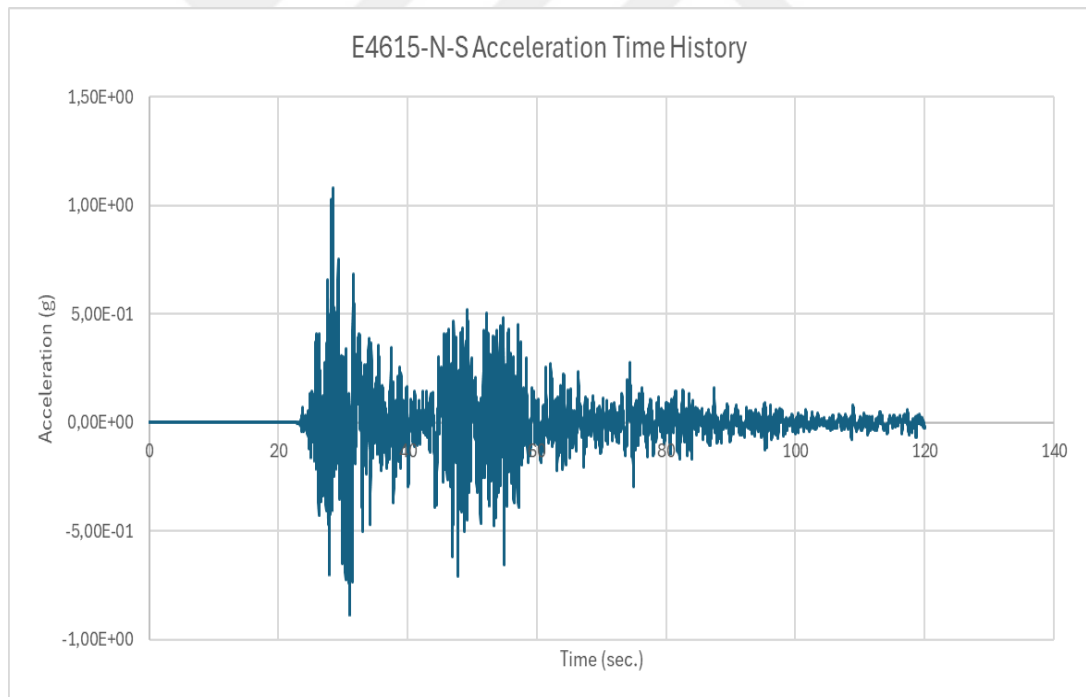


Figure 5.33: 4615- N-S acceleration time history

The earthquakes were defined in the ETABS software, and the hysteresis curve of the Buckling-Restrained Brace (BRB) element in the structure is presented in **[Figure 34]**.

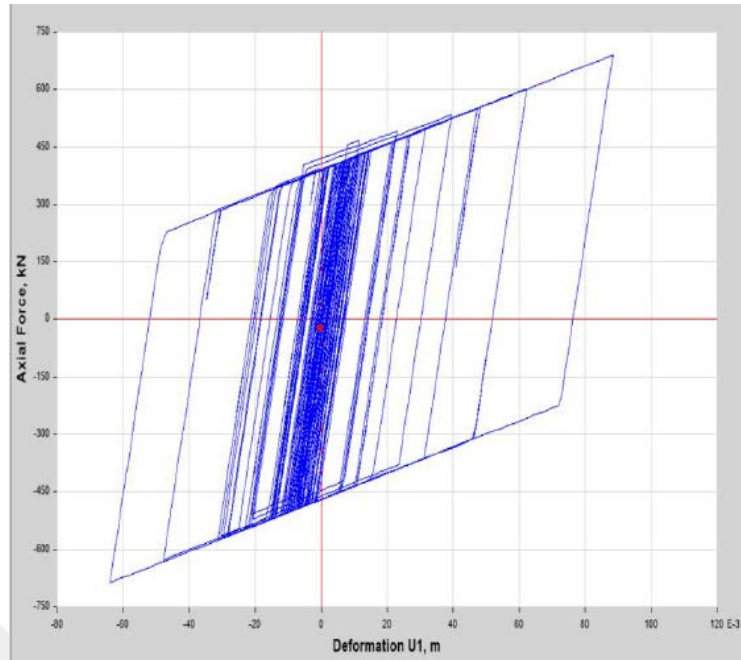


Figure 5.34: Hysteresis curve of the Buckling-Restrained Brace (BRB)

As a result of the analyses, the story drift ratios were calculated separately for each earthquake record and are presented in [Figure 35], [Figure 36], [Figure 37], and [Figure 38].

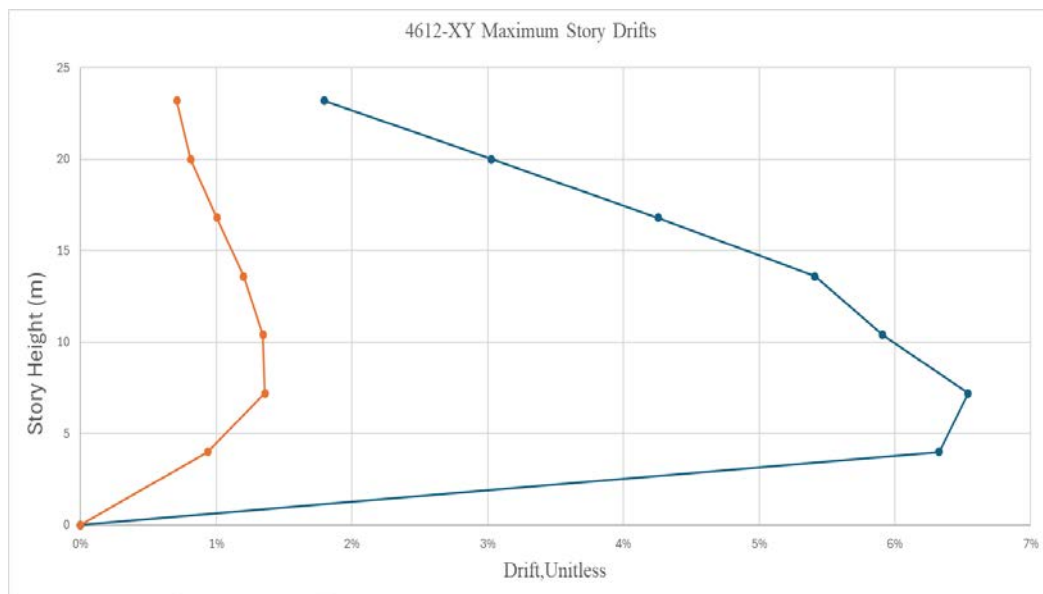


Figure 5.35 4612-XY maximum story drift ratio (%).

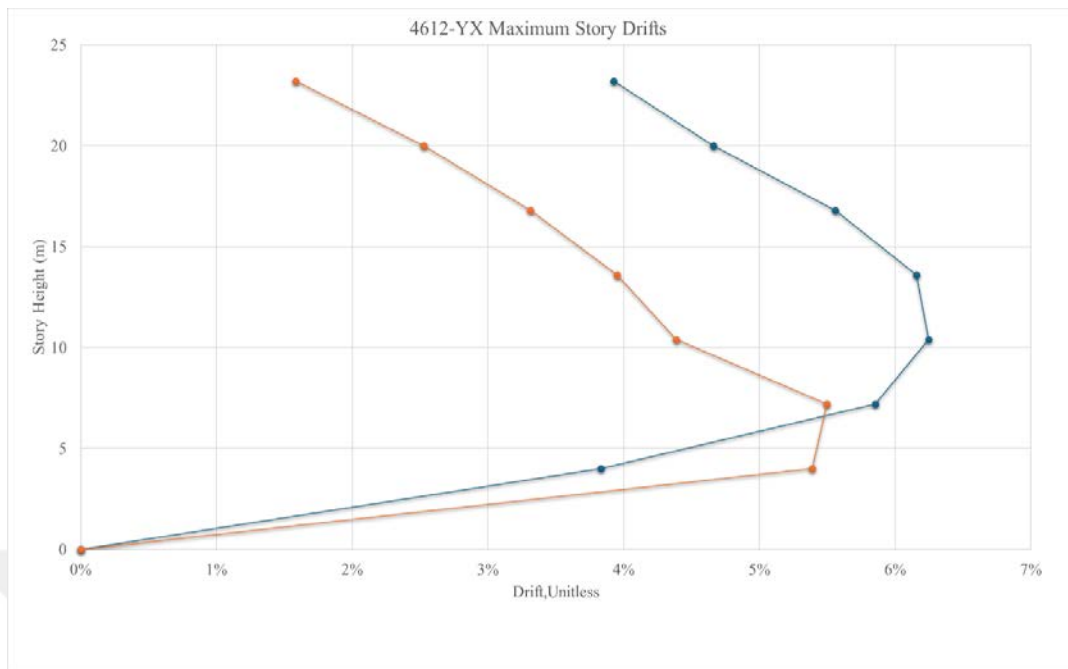


Figure 5.36: 4612-YX maximum story drift ratio (%)

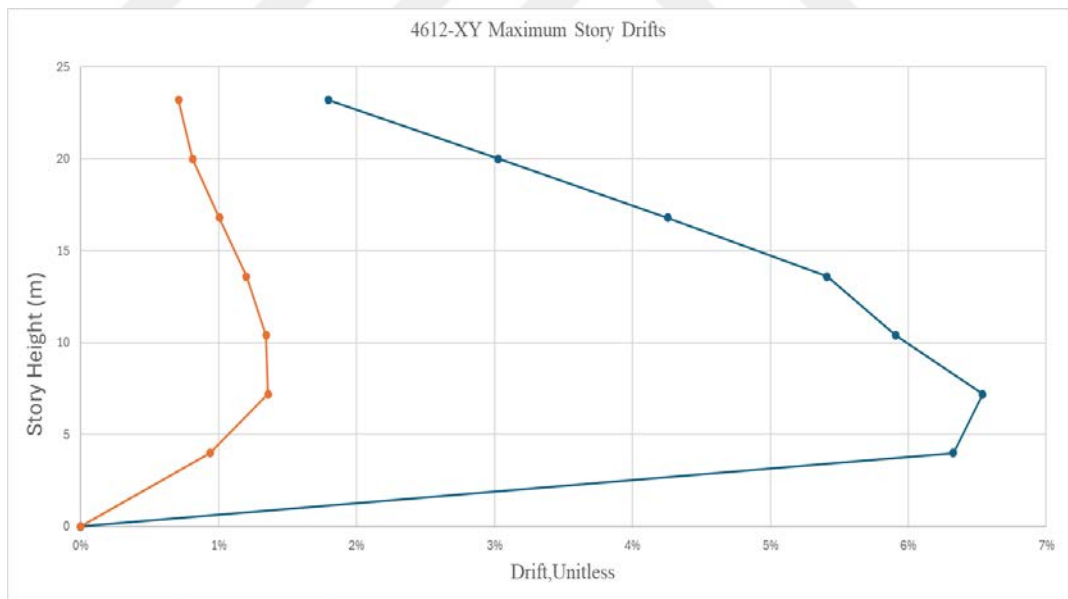


Figure 5.37 4615-XY maximum story drift ratio (%)

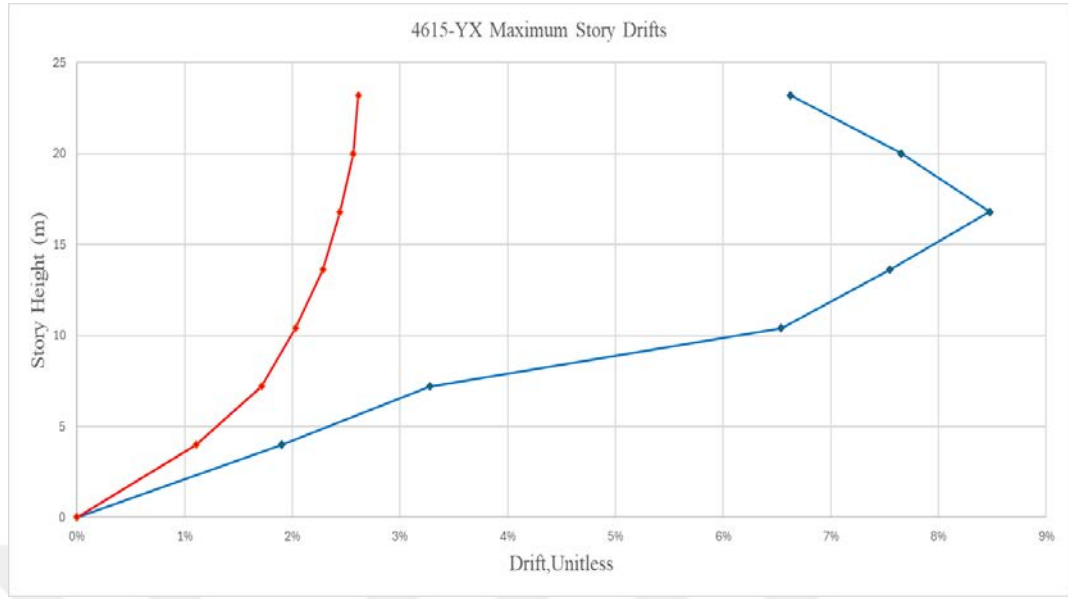


Figure 5.38 4615-XY maximum story drift ratio (%).

As part of this thesis, time history analyses were conducted using the ground motion records of the Pazarcık (Mw 7.7) and Elbistan (Mw 7.6) earthquakes, two of the most powerful and destructive seismic events ever recorded in Türkiye, which struck the southeastern region on February 6, 2023. These two major seismic events occurred along the East Anatolian Fault and caused widespread devastation. Based on the analyses conducted, it was observed that these strong ground motions led to damage in some structural elements.

6. CONCLUSION AND RECOMMENDATIONS

The structural system reflects the problematic building stock commonly found in Türkiye. It is a hospital building, and due to its critical function, all analyses were conducted based on the highest seismic level, DD1, in accordance with the Turkish Seismic Code. The site soil classification is ZC. In such a critical structure, BRBs were proposed to address the damage observed in the bottom-story columns, which are most exposed to seismic forces. Damage assessments indicated that global strengthening was necessary, as outlined in TBDY-2018. Within this scope, the BRB system—considered one of the increasingly common and innovative retrofitting methods in Türkiye—was selected and implemented.

For the installation of BRBs, the outermost axes were chosen. This allowed the strengthening to be carried out externally, without interrupting the building's interior use, thus eliminating the need for evacuation. BRBs were placed starting from the outer axes and extending inward. This layout helped to distribute the axial loads more evenly on the columns and prevented potential collapse due to axial force concentration.

The implementation of BRBs (Buckling-Restrained Braces) proved especially effective in reducing lateral displacements in the first three stories of the structure, where the most critical damage had been observed. According to the story drift ratio results obtained through pushover analysis, these stories initially showed significantly high drift ratios: 2.4% at the first story, 2.6% at the second story, and 1.8% at the third story. After BRB retrofitting, the drift ratios in these stories were reduced to 1.3%, 1.0%, and 0.7%, respectively. These reductions represent a substantial improvement and directly reflect the enhanced seismic performance of the structure in its most vulnerable regions.

The first story, in particular, stands out due to its atypical height of 4 meters, which makes it structurally weaker compared to the upper stories that have regular story heights of approximately 3.2 meters. This height difference introduced a vertical irregularity, commonly referred to as a soft-story condition, which made the base of the structure more susceptible to damage under seismic loads. Before retrofitting, this

irregularity contributed significantly to the excessive story drift ratio and high damage observed at the first story.

To address this issue, BRBs were strategically added to the first, second, and third stories—focusing especially on the first story to mitigate the effects of its increased flexibility and vulnerability. The placement started from the outermost axes and moved inward, optimizing the load paths and contributing to a more balanced structural response. This approach not only reduced story drift ratios but also distributed axial forces more effectively, preventing concentration of stress in the columns, particularly in the lower levels where axial loads are already higher.

Through this targeted strengthening strategy, the irregularity at the first story was effectively corrected, and the transition in stiffness between the stories was smoothed out. As a result, the system's behavior became more uniform and predictable during lateral loading. These outcomes demonstrate that the selective application of BRBs in the lower stories can both reduce structural irregularities and significantly improve the overall seismic resilience of reinforced concrete buildings.

In the Y direction of the structure, shear walls located along axes 1A, 1D, 8A, and 8D in each bay provided sufficient lateral resistance, as revealed by the pushover analysis. It was observed that no additional strengthening was required in this direction, clearly demonstrating the effectiveness and importance of shear walls in reinforced concrete buildings. However, in the X direction, the extent of damage necessitated global strengthening, as required by the Turkish Seismic Code **[TBDY-2018]**. For this reason, retrofitting was carried out unidirectionally, focusing solely on the X direction.

According to the PUSH-X analysis results, the strengthening work effectively brought all columns within the performance level limits defined by TBDY-2018 under the DD1 earthquake scenario. Before retrofitting, the first story showed severe vulnerability: 7% of the columns were in the significant damage zone, 61% in the advanced damage zone, and 32% were in the collapse zone. After retrofitting with BRBs, all columns in this story were successfully shifted to the limited damage zone.

In the second story, a similar improvement was observed, where 100% of the columns were also moved into the limited damage zone. In the upper stories, from the third to the seventh, no damage was recorded. These findings confirm that the application of BRBs significantly enhanced the lateral stiffness of the structure and effectively

prevented excessive damage in the columns, contributing to a more controlled and resilient structural response under severe seismic conditions.

According to the pushover analysis results, the total building height was 23.2 meters. Roof displacement was measured as 0.29 meters before retrofitting and reduced to 0.16 meters after the implementation of BRBs, indicating a 55% improvement. This significant reduction demonstrates the positive impact of BRBs on limiting lateral displacements and improving global structural behavior.

Moreover, BRBs notably increased the lateral stiffness of the building without causing a significant rise in axial loads. The BRB layout was carefully optimized to minimize additional axial force effects on the columns, particularly in the lower stories where axial demands are more critical. This balanced design approach ensured effective retrofitting without introducing new vulnerabilities to the structural system.

Within the scope of this thesis, time history analyses were carried out using ground motion records from the Pazarcık (Mw 7.7) and Elbistan (Mw 7.6) earthquakes, which stand among one of the most severe and destructive seismic events in Türkiye's history. Occurring on February 6, 2023, in the southeastern part of the country, these earthquakes caused significant disruption over a broad region. Based on Time History analyses conducted, it was observed that these strong ground motions led to damage in some structural elements.

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BIOGRAPHY

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During high school, she served as the head of the school sports club, founded an entrepreneurship club, and contributed as a writer to a publication. At Yıldız Technical University, she organized major engineering events such as Civil Istanbul and İlk Baretim and participated in several social responsibility projects.

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