

DESIGN OF SEISMIC RETROFITTING USING INNOVATIVE TECHNIQUES:
A CASE STUDY ON A SCHOOL BUILDING

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ABSTRACT

DESIGN OF SEISMIC RETROFITTING USING INNOVATIVE TECHNIQUES: A CASE STUDY ON A SCHOOL BUILDING

Seismic retrofitting plays a critical role in ensuring the safety and resilience of structures in earthquake-prone regions. This study investigates the performance of fluid viscous dampers (FVDs) as an alternative retrofitting technique for improving the seismic performance of buildings. The Büyük Halkalı Primary School Building in Istanbul, originally constructed in 1997 and retrofitted twice using conventional methods (in 2006 and 2019), was selected as a case study. The study aims to propose an innovative retrofitting solution that minimizes construction time, operational disruptions, and associated costs while enhancing seismic resilience. A comparative analysis was conducted between the conventional retrofit approach (reinforced concrete shear walls) and the proposed alternative retrofit solution incorporating FVDs. The evaluation considered seismic performance, cost-effectiveness, serviceability, and architectural impact. Nonlinear time-history analyses were performed using TBDY-2018 and ASCE 41-23 standards, with a particular focus on near-fault effects, including forward directivity effects that were not considered in previous retrofits. The results demonstrated that the incorporation of FVDs significantly improved seismic performance by reducing inter-story drift and structural damage. Furthermore, the alternative retrofit method resulted in shorter construction periods and minimized the necessity for building evacuation, ensuring continuous serviceability. The findings highlight that using FVDs in seismic retrofitting can be a viable solution, particularly for structures requiring uninterrupted functionality during construction, such as school buildings.

ÖZET

YENİLİKÇİ TEKNİKLER KULLANARAK SİSMİK GÜÇLENDİRME TASARIMI: BİR OKUL BİNASI ÖRNEĞİ

Deprem riski yüksek bölgelerde yapıların deprem güvenliğini ve dayanıklılığını sağlamak için deprem güçlendirmesi kritik bir öneme sahiptir. Bu çalışma, binaların deprem performansını artırmak amacıyla alternatif bir güçlendirme yöntemi olarak viskoz sönümleyicilerin performansını incelemektedir. Bu kapsamda İstanbul'da bulunan Büyük Halkalı İlköğretim Okulu Binası vaka çalışması olarak seçilmiştir. İncelenen bina 1997 yılında inşa edilmiş olup 2006 ile 2019 yıllarında geleneksel yöntemlerle iki kez güçlendirilmiştir. Bu çalışmanın amacı, inşaat süresini, operasyonel kesintileri ve ilgili maliyetleri azaltırken sismik dayanıklılığı artıran yenilikçi bir güçlendirme çözümü önermektir. Geleneksel güçlendirme yöntemleri (betonarme perde duvarlar) ile önerilen alternatif güçlendirme çözümü (viskoz sönümleyicilerin kullanıldığı sistem) arasında karşılaştırmalı bir analiz gerçekleştirilmiştir. Değerlendirme kapsamında deprem performansı, maliyet etkinliği, kullanılabilirlik ve mimari etkiler ele alınmıştır. Özellikle önceki güçlendirme uygulamalarında dikkate alınmayan yakın fay etkilerinden olan yönelim etkisine de odaklanılarak TBDY-2018 ve ASCE 41-23 standartları çerçevesinde doğrusal olmayan zaman-tanım alanında analizler yapılmıştır. Elde edilen sonuçlar, viskoz sönümleyicilerin katlar arası ötelenmeyi ve yapısal hasarı azaltarak yapının deprem performansını önemli ölçüde artırdığını göstermiştir. Ayrıca, geleneksel güçlendirme ile kıyaslandığında, alternatif güçlendirme yönteminin daha kısa inşaat süreleri sağlayacağı sonucuna varılmış ve bina tahliye ihtiyacının en aza indirilerek sürekli kullanım olanağı sunulabileceğine kanaat getirilmiştir. Bulgular, özellikle okul binaları gibi inşaat sürecinde kesintisiz işlevsellik gerektiren yapılar için, viskoz sönümleyicilerin deprem güçlendirmesinde kullanılmasının uygulanabilir ve etkili bir çözüm olduğunu ortaya koymaktadır.

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LIST OF SYMBOLS

b	Section width for columns or beams
B_1	Damping coefficient used to adjust spectral response for the effect of viscous damping,
C	Damping constant
C_0	Damping coefficient for the device
C_1	Regression coefficient
C_2	Regression coefficient
$\dot{D}$	Relative velocity between each end of the device
d	Cover thickness for reinforced concrete elements
F	Damping force
f_1	Fundamental frequency of the building
f_{ywe}	Average (expected) yield strength of transverse rebar
f_{ye}	Average (expected) yield strength of longitudinal rebar
h	Section height for columns or beams
L_F	Distance to the fault plane in km
k_i	Story stiffness obtained from static analysis of the building
M_w	Moment magnitude of earthquake
n	Number of longitudinal reinforcements
n_i	Number of dampers in each direction
n_x	Number of stirrup legs in X-direction
n_y	Number of stirrup legs in Y-direction
s	Spacing of transverse reinforcement
sgn	Signum function which defines the sign of the term
T	Fundamental period of the building in related direction
V	Relative velocity between two ends of the damper
$SPAF$	System property adjustment factor
X	Length ratio (fraction of the fault along strike that ruptures toward the site)
Y	Width ratio (fraction of the fault up-dip that ruptures toward the site)

α	Damping exponent
β	Effective viscous damping ratio of the structural system expressed as a decimal (as opposed to percent)
ϕ	Zenith angle between the fault plane and the ray path to the site
γ_F	Near-fault coefficient
ζ	Viscous damping ratio
λ_{ae}	Property variation factor caused by the individual aging and environmental effects
λ_{max}	Property modification factor for calculation of the maximum value of the energy dissipation device property of interest, used to account for all sources of property variability
λ_{min}	Property modification factor for calculation of the minimum value of the energy dissipation device property of interest, used to account for all sources of property variability
λ_{spec}	Variation on the average of the manufacturing production test values from the nominal design value
λ_{test}	Lambda factors obtained from testing
θ	Azimuth angle between the fault plane and the ray path to the site
θ_i	Angle of the dampers with the ground

LIST OF ACRONYMS/ABBREVIATIONS

AFAD	Afet ve Acil Durum Yönetimi Başkanlığı
BOQ	Bill of Quantities
BRB	Buckling Restrained Braces
Caltrans	California Department of Transportation
CBF	Chevron Braced Steel Frames
CFRP	Carbon Fiber Reinforced Polymer
FEMA	Federal Emergency Management Agency
FVD	Fluid Viscous Dampers
IPCU	Istanbul Project Coordination Unit
ISMEP	Istanbul Seismic Risk Mitigation and Emergency Preparedness Project
KOERI	Kandilli Observatory and Earthquake Research Center
MCE _R	Risk-targeted maximum considered earthquake ground motion response acceleration
MCEER	Multidisciplinary Center for Earthquake Engineering Research
MTA	Maden Tetkik Arama
NAF	North Anatolian Fault
NAFZ	North Anatolian Fault Zone
NDP	Nonlinear Dynamic Procedure
NEHRP	National Earthquake Hazards Reduction Program
NLTH	Non-linear Time-History Analysis
NTCS-17	Norma Técnica Complementaria Para El Diseño Por Sismo
PEER	Pacific Earthquake Engineering Research Center
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Calculations
RC	Reinforced Concrete
ŞGDT	Şekildeğiştirmeye Göre Değerlendirme ve Tasarım (<i>Deformation-Based Assessment</i>)
TBDY-2018	Türkiye Bina Deprem Yönetmeliği
UBC	Uniform Building Code
USA	United States of America
VDD	Velocity Dependent Dampers

1. INTRODUCTION

1.1. Need for Innovative Techniques in Seismic Retrofitting

Seismic retrofitting is an economical solution in enhancing the resilience of structures against seismic forces, safeguarding both lives and property. It is widely used in earthquake prone countries i.e. Japan, USA, New Zealand, Italy and Türkiye for decades. Especially, 1999 İzmit Earthquake (Mw7.4) had become a milestone for seismic retrofitting for Türkiye. After the 1999 İzmit Earthquake (Mw7.4), many buildings were retrofitted nationwide.

Seismic performance assessment for existing structures was added to Turkish Seismic Design Codes in 2007. In 2018, the code was totally renewed, and performance assessment chapter was enhanced. Retrofit criteria were defined in more detail covering modern analysis methods. In spite of that, regarding retrofitting subject, code covers only conventional techniques such as addition of Reinforced Concrete (RC) or steel shearwalls, column-jacketing, beam wrapping etc. Energy dissipators such as viscous dampers, Buckling Restrained Braces(BRB)'s, visco-elastic dampers, friction dampers, tuned mass dampers etc. are not mentioned in the code. Therefore, in the retrofit projects, engineers mostly use conventional techniques which demand evacuating the buildings for months. Just for this reason, many building owners willing to safeguard their structures have refrained from retrofitting their building. Innovative retrofit design examples are relatively less in practice but are much-needed by construction sector in Türkiye. Hence, in order to increase case studies of innovative techniques, an alternative viscous damper solution has been proposed using a conventionally reinforced structure. The new solution is compared with the existing one in terms of structural performance, business interruption durations and construction costs. The first findings on this comparison were presented and published in 2023 [1].

In this context, a school building is selected for a case study. It was retrofitted two times within the scope of Istanbul Seismic Risk Mitigation and Emergency Preparedness Project (ISMEP) of Istanbul Project Coordination Unit (IPCU). IPCU of Republic of Türkiye had started the ISMEP with the loans from the World Bank, European Investment Bank, Council of Europe Development Bank, Islamic Development Bank, KfW Development

Bank, Asian Infrastructure Investment Bank, Ecobank in 2006, and 932 school buildings were retrofitted in Istanbul within this project.

This study focuses on the seismic retrofitting of the Büyük Halkalı Primary School Building in İstanbul, a structure constructed in 1997 and previously retrofitted (within the scope of ISMEP project) in 2006 using conventional methods (by adding RC shearwalls). After publishment of the latest Seismic Code for Buildings in Türkiye (TBDY-2018) [2]], the structure's seismic performance was reevaluated in 2019, and further enhancements were made, including the addition of new shear walls to improve structural integrity. Therefore, the structure was retrofitted two times (in 2006 and 2019) which results in the evacuation of the building for two times leaving it out of service. Besides, all the construction works including displacement of the electrical/mechanical equipment and renewing the architectural finishings had been done two times.

The main objective of this study is to propose an alternative innovative retrofit solution with external interference only which limits the construction works inside the building at minimum, so the service of the retrofitted structure can continue without interruption during construction works. Also, the proposed solution is compared with the applied conventional solutions in terms of structural performance, business interruption times and construction costs.

1.2. Near Fault Effects

Near-fault ground motions exhibit unique characteristics that differ significantly from far-field seismic effects. These effects can lead to higher spectral acceleration values, large velocity pulses, and increased displacement demands. These characteristics of the near-fault ground motions were observed initially by Housner and Trifunac [3] in the records of 1966 Parkfield earthquake at Cholame.

The primary near-fault effects include “Forward Directivity Effect” and “Fling Step Effect”. Forward directivity effect becomes dominant when rupture propagation is directed toward a site. The accumulation of seismic wave energy generates high-velocity, pulse-like

ground motions, increasing structural demands. Fling-Step Effect is the permanent ground displacement that causes large baseline shifts. [4], [5].

It is known that direction of the fault rupture affects the amplitude (and duration) of the ground motions around faults, which may result an increase in accelerations for periods higher than 0.6s, around faults in the direction of rupture [4]. However, in TBDY-2018, near fault effects including forward directivity or fling effects are not covered. Therefore, these effects are not taken into consideration during the retrofits applied to the selected school building under review although it is 17 km away from the North Anatolian Fault and is in the direction of the fault that is expected to rupture.

In this study, these effects are also evaluated and taken into account during the design of the alternative retrofit solution.

2. LITERATURE SURVEY

2.1. Viscous Dampers

In conventional structural systems, the energy released by the earthquakes is dissipated by the damage (yielding, plastic strains) in the structural elements. For instance, in a structure consisting of reinforced concrete frames, a considerable amount of earthquake energy is absorbed at the beam-column joints.

One of the most effective ways to reduce earthquake damage in structures is through energy dissipation systems. Dampers are developed based on the principle that the earthquake energy affecting the structure should be absorbed by energy absorbing devices designed for this purpose instead of absorbing with the plastic behavior of the structural load-bearing elements through damage. Energy dissipation systems operate on the same principle as shock-absorbing dampers in cars. They ensure that earthquake energy is dissipated by energy-dissipating devices (seismic dampers) rather than causing damage to structural elements.

Although seismic dampers are not yet widely used in Türkiye and guidelines for damping systems are not covered in the design code TBDY-2018 [2], these devices have been extensively used for many years in the United States, Mexico, Italy and Japan.

Seismic dampers are generally categorized into active and passive systems. Active damping systems utilize sensors to regulate their response dynamically, whereas passive systems operate independently, without real-time adjustments to external effects. Passive seismic energy dissipation devices can be classified into two main categories: (1) displacement-dependent devices, which dissipate energy based on the relative displacement between their ends—such as friction dampers and hysteretic devices (e.g., yielding dampers); and (2) velocity-dependent devices (VDDs), where energy dissipation is governed by the relative velocity across the damper [6]. This classification framework is adopted in various seismic design standards, including Chapter 18 of ASCE 7-22 [7], Appendix B of NTCS-17 [8], and the European guidelines outlined in EN 15129 [9].

This section will provide information and a summary of studies in literature about **fluid viscous dampers** (FVDs) due to the intended use of viscous dampers for strengthening the selected school structure.

Viscous dampers absorb seismic energy by transforming it into heat via fluid flow through orifices. Fluids such as silicone, oil, or lead can be utilized within viscous dampers. They are affected by ambient temperature depending on the properties of the fluid material, but there are also varieties capable of functioning in harsh outdoor environments.

The fundamental design components of a fluid damper are limited in number. Nevertheless, the specifics of these components can vary significantly and, in certain instances, can be both challenging and complex. Figure 2.1 illustrates a standard fluid damper and its various parts [10].

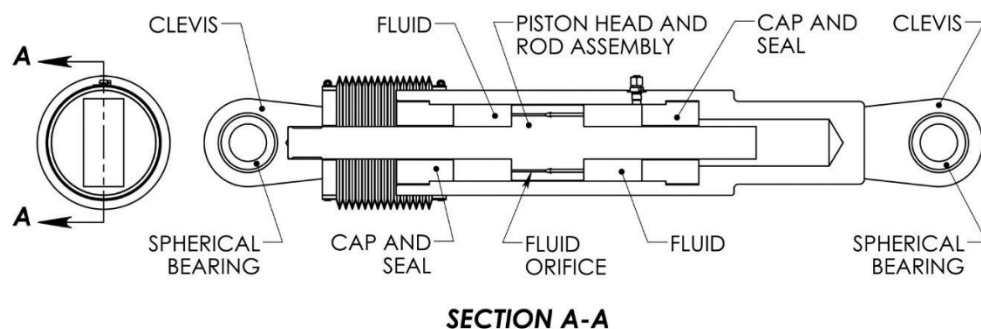


Figure 2.1. Typical fluid damper & parts [10].

Viscous dampers function by generating a resisting force exclusively during motion, without contributing stiffness to the structure or bearing any static loads. Nonetheless, stiffness can be integrated into a damper if specified. In a viscous damper, a piston cycles through a viscous fluid, creating high pressure. This piston is equipped with specially designed orifices that establish an optimized correlation, creating a pressure (force) that changes with velocity. Consequently, higher velocities result in higher resisting forces. This relationship is typically characterized as

$$F = CV^\alpha. \quad (2.1)$$

In this expression, F is the damping force, C is the damping constant, V is the relative velocity between two ends of the damper and α is referred to as the damping exponent. Typically, this exponent is adjusted from 0.2 to 2.0 [11]. However, producers can adjust α to 0.1 depending on the specific use case. For seismic applications in buildings, values of α are commonly set between 0.3 and 0.5 [10]

Referring to Equation 3.1, when comparing the energy absorbed by a viscous damper with sinusoidal input per cycle to its damping exponent, it is evident that dampers with lower damping exponents absorb more energy each cycle than those with higher exponents.

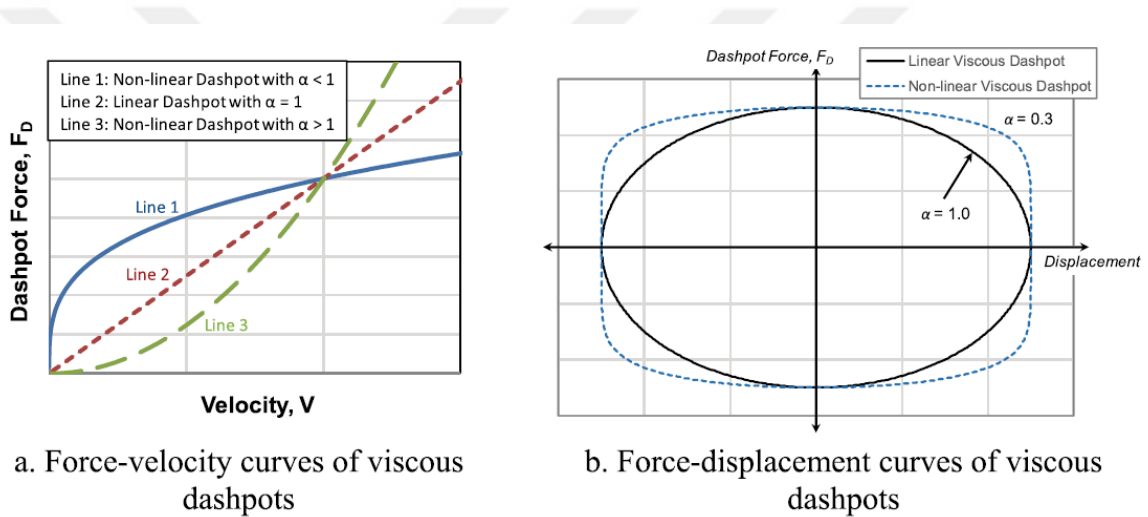


Figure 2.2. Linear and nonlinear viscous dashpot force–velocity and force–displacement relationships. (a) Force–velocity curves of viscous dashpots. (b). Force–displacement curves of viscous dashpots [12].

In this study, the damping exponent α is taken as 0.1 in order to limit the damper forces while increasing the damping ratio.

The initial noteworthy study concerning usage of fluid viscous dampers in buildings in order to increase the seismic performance of the structures is the work of Constantinou [13] which is a combined experimental and analytical study of an energy absorbing system for structures, consisting of fluid viscous dampers. The study conducted various component tests to establish the dampers' mechanical characteristics and assess their frequency and temperature dependencies. Experiments were carried out on both one- and three-story model

structures featuring different damper configurations. These findings were incorporated into a mathematical model, which was then used to analyze the seismic response of one-story and three-story steel structures equipped with dampers. Additionally, simplified models and analytical methods were devised and assessed, demonstrating results that closely matched the experimental data. Results demonstrated that the inclusion of these dampers substantially reduced inter-story drifts and shear forces, confirming the model's accuracy and the dampers' effectiveness in mitigating seismic impacts. Integrating fluid viscous dampers into the tested structures led to decreases in story drifts ranging from 30% to 70%. These reductions are similar to those attained by other energy-dissipating systems, including viscoelastic, friction, and yielding steel dampers. Additionally, the use of fluid dampers resulted in reductions in story shear forces by 40% to 70%, a level of performance that other energy-absorbing devices failed to significantly achieve.

Several studies have led to the pursuance of passive energy dissipation systems in new and existing constructions after Constantinou [13].

The study by Uriz and Whittaker [14] delves into retrofitting pre-Northridge steel moment-resisting frames with fluid viscous dampers. It underscores the use of linear fluid viscous damping devices for seismic retrofit on a three-story steel-framed building, focusing on reducing beam plastic rotations and displacement during seismic events. By employing FEMA 273 procedures, the study achieved significant reductions in frame displacement with an added damping of about 40%, though complete elimination of plastic rotations was not fully realized. Moreover, the retrofit led to increased base shear and column axial forces, necessitating potential strengthening of structural components. The research highlights the complexity of retrofitting with fluid viscous dampers, noting that while they reduce seismic response effectively, they require careful integration into existing systems to manage increased loads and ensure overall structural integrity.

The study by Miyamoto and Singh [15] explores the efficacy of passive energy dissipators, particularly fluid viscous dampers, in enhancing the seismic resilience of steel moment frame buildings. Focusing on buildings of 1, 5, and 11 stories designed to meet the UBC-97's strength and drift requirements, the study investigates how these structures behave under various seismic intensities with and without damping systems. Both linear and

nonlinear time history analyses were used, revealing that structures equipped with energy dissipation systems generally remained elastic under seismic events that did not exceed the UBC Zone 4 response spectrum. Specifically, the inclusion of fluid viscous dampers was found to significantly improve immediate occupancy performance levels during high-intensity earthquakes, and enhance the overall seismic performance compared to frames without such systems. The study also noted that dampers could reduce seismic demands considerably, suggesting a potential shift towards their integration in performance-based design codes.

The research conducted by Pavlou and Constantinou (2004) [16] explores the seismic performance of both elastic and inelastic structural systems incorporating damping mechanisms, with a particular emphasis on their behavior under near-fault and soft-soil ground motion excitations. The study evaluates the efficiency of both linear and nonlinear viscous dampers through simplified models and nonlinear response-history analysis. One of the primary conclusions is that damping systems can significantly reduce ductility demands, even when the structures are designed for base shear forces below those specified for conventional buildings in accordance with the 2000 NEHRP Provisions. The findings underscore the effectiveness of supplemental damping in mitigating the adverse effects of near-fault and soft-soil ground motions by reducing lateral displacements and internal force demands in the structural system.

Subsequently, in their 2006 publication, Pavlou and Constantinou [17] focused on the seismic response of nonstructural components within buildings equipped with damping systems. Using analytical models of three-story steel frame structures designed in compliance with the 2000 NEHRP Provisions, the study investigated the dynamic behavior of secondary systems subjected to various ground motion scenarios, including far-field, near-fault, and soft-soil records. The analysis revealed that the incorporation of viscous dampers led to notable reductions in floor spectral accelerations and absolute floor velocities, thereby enhancing the protection of nonstructural elements. These results affirm that damping systems not only improve the seismic performance of primary structural components but also offer substantial resilience benefits for nonstructural systems during strong ground shaking.

The research by Dicleli and Mehta (2007) [18] investigates the impact of near-fault ground motion characteristics and damper parameters on the seismic response of chevron braced steel frames with fluid viscous dampers. The study utilizes nonlinear time history analyses to explore how various velocity pulse periods and damper settings influence frame performance under near-fault conditions. The results underscore that frames without fluid viscous dampers exhibit poor performance and increased sensitivity to near-fault ground motions due to brace buckling. In contrast, the inclusion of dampers markedly enhances frame behavior, maintaining elastic performance and significantly mitigating seismic demands. Frames equipped with dampers featuring lower velocity exponents and higher damping ratios showed the best performance, highlighting the effectiveness of fluid viscous dampers in reducing seismic vulnerabilities in steel framed structures exposed to near-fault seismic activities.

The study by Murat Dicleli (2007) [19] investigates the seismic performance of chevron braced steel frames (CBFs) with and without fluid viscous dampers (FVDs) as influenced by ground motion and damper characteristics. The research, utilizing nonlinear time history (NLTH) analyses, demonstrates that the inclusion of FVDs significantly enhances the seismic resilience of CBFs by maintaining their elastic behavior during seismic events. It was found that frames with FVDs equipped with smaller velocity exponents and larger damping ratios exhibited better seismic performance. Frames without FVDs showed poor performance and were highly sensitive to the frequency characteristics of ground motion due to brace buckling effects. The study also found that FVDs are particularly effective in reducing seismic demands, highlighting their potential for integration into performance-based seismic design standards.

The study conducted by Kitayama and Constantinou (2018) [20] critically evaluates the seismic performance of buildings equipped with fluid viscous damping systems, as outlined by the procedures of ASCE/SEI 7-16. The research meticulously analyzes the influence of various design parameters on the seismic resilience of structures, particularly focusing on the required strength of the brace-damper-connection system, the base shear force for frame design excluding the damping system, the level of damping provided, the damper displacement capacity, and the type of damping which includes the utilization of fluidic self-centering devices. The study's findings reveal that the most critical factors for

enhancing building resilience against collapse and minimizing the probability of exceeding specific residual story-drift thresholds are the design's base shear force, the robustness of the brace–damper–connection system, and the extent of damping implemented. These parameters significantly affect the buildings' ability to withstand seismic events, reducing both the risk of structural failure and the magnitude of drift experienced during such incidents. Moreover, the research underscores that while buildings designed with viscous dampers as per ASCE/SEI 7-16's minimum criteria do not typically exhibit reduced drift or less inelastic action compared to conventional buildings, they do benefit from decreased floor accelerations and response spectra due to the dampers. This advantage is particularly noted when higher levels of viscous damping are employed, highlighting the potential of viscous dampers in seismic retrofitting and new constructions for enhanced performance in earthquake-prone regions.

2.1.1. Design Codes That Can be Used for FVD's

As explained above, Fluid Viscous Dampers (FVDs) are velocity-dependent devices designed to dissipate seismic energy in buildings. Generally, while the level of effective damping within a structure increases, the responses in terms of forces, displacements, stresses, and drift ratios decrease. Typically, buildings possess an inherent damping ratio (β) ranging from 2% to 5% of critical. By strategically placing FVDs throughout the height of a building, it is possible to achieve significantly higher levels of effective damping.

Over the past several decades, a range of guidelines for designing structures with anti-seismic devices has emerged, initially focusing on seismic isolation for structural systems and later expanding to encompass structures equipped with passive energy dissipation devices [6].

Mostly followed design standards for structures with damping devices are a) the ASCE/SEI Minimum Design Load for Buildings and Other Structures 7-22 (ASCE 7-22) [7], b) the European Standard for Anti-seismic Devices EN 15129 [9], and c) the Technical Regulations for Seismic Design, known as Normas Técnicas Complementarias para Diseño por Sismo (NTCS-17) [8], which forms part of the Mexico City Building Code. These standards represent frameworks from the United States, Europe, and Mexico, respectively.

ASCE 41-23 Chapter 15 [21] also includes design requirements for existing structures to be retrofitted with supplemental energy dissipation. In Turkey, TBDY-2018 [2] does not cover energy dissipating systems, however in section 1.1.9 of TBDY-2018 [2], it is stated that: “Until specific regulations are established for the design of buildings and building-type structures that fall outside the scope of this Regulation (TBDY-2018), the provisions provided in the relevant Turkish Standards, along with internationally recognized equivalent standards, regulations, and other technical arrangements, or technical rules determined by institutions, may be utilized in accordance with the principles outlined in this Regulation.” [2]. Therefore, one of the design codes mentioned above (ASCE 7-22, ASCE 41-23, EN 15129 or NTCS-17) can be used in Turkey while designing a structure with energy dissipation system.

The provisions stipulated in various seismic design codes differ notably in terms of minimum requirements and placement criteria for energy dissipation devices. ASCE 7-22, under Clause 18.2.4.6, prescribes that if a given story of a structure contains fewer than four energy dissipation devices in either of the two principal directions, or if fewer than two devices are positioned on each side of the story’s center of stiffness along any principal axis, then all installed devices must be designed to accommodate displacements of at least 130% of the maximum expected device deformation under Maximum Considered Earthquake (MCER) loading. The MCER level referenced herein corresponds to the DD-1 seismic hazard level as defined in the TBDY-2018 (Turkish Seismic Code). In contrast, the NTCS-17 and EN 15129 standards do not enforce any explicit minimum requirements concerning the number or distribution of energy dissipation devices within structural stories, thereby allowing greater design flexibility in this context.

ASCE 41-23 Chapter 15 [21] is selected as a reference for the design of the fluid viscous dampers (FVDs) in the scope of this study while the level of earthquake and acceptance criteria for the reinforced concrete elements are selected according to TBDY-2018[2]. The reason for selecting ASCE 41-23 Chapter 15 [21] is that it is a specialized code for evaluation and retrofit design of existing buildings.

2.1.2. Requirements for Retrofit of Structures using FVDs according to ASCE41-23

Chapter 15 of ASCE 41-23 [21] outlines the requirements for the systematic evaluation and retrofit of buildings using supplemental energy dissipation systems, which include fluid viscous dampers (FVDs). Some requirements are stated below:

- Nominal design properties for FVDs must be established (Section 15.3.1) [21].
- Maximum and minimum properties (velocity coefficients, stiffness, strength, and energy dissipation) should be defined to account for variations in behavior due to loading conditions using following formulas:

$$\text{Maximum Design Property} = \text{Nominal Design Property} * \lambda_{max} \quad (2.2)$$

$$\text{Minimum Design Property} = \text{Nominal Design Property} * \lambda_{min} \quad (2.3)$$

where λ_{max} and λ_{min} factors shall be calculated in accordance with following equations for each device by the designer (Section 15.3.2) [21].

$$\lambda_{max} = \lambda_{test\ max} * \lambda_{spec\ max} * (1 + SPAF(\lambda_{ae\ max} - 1)) \geq 1.20 \quad (2.4)$$

$$\lambda_{min} = \lambda_{test\ min} * \lambda_{spec\ min} * (1 - SPAF(\lambda_{ae\ max} - 1)) \leq 0.85 \quad (2.5)$$

where λ_{test} is the lambda factors obtained from testing, λ_{spec} is the variation on the average of the manufacturing production test values from the nominal design value, λ_{ae} is the property variation factor caused by the individual aging and environmental effects, and $SPAF$ is the system property adjustment factor which is equal to 0.67 for all performance objectives.

- Multi-axis Movement: Dampers must be capable of resisting forces in all directions relevant to the seismic response of the building (Section 15.2.3.2) [21].
- Inspection and Testing: Periodic inspection and testing of dampers is required, ensuring compliance with established performance criteria (Section 15.2.3.3) [21].
- The Nonlinear Dynamic Procedure (NDP) is the required method for designing velocity-dependent dampers such as FVDs (Section 15.4) [21].

- Linear methods are not permitted unless the effective damping ratio does not exceed 30% of critical damping (Section 15.4) [21].
- The seismic-force-resisting system (SFRS) and damping system must be explicitly modeled in the analysis (Section 15.5.1) [21].
- The damping system must be modeled with its nonlinear force-velocity-displacement characteristics.
- The plan and vertical distribution of the dampers must be accurately represented in the model (Section 15.5.1) [21].
- The fluid viscous dampers which do not exhibit stiffness in the frequency range $0.5f_1$ to $2.0f_1$, the force in the fluid viscous device shall be computed in accordance with the following equation:

$$F = C_0 |\dot{D}|^\alpha * \text{sgn}(\dot{D}) \quad (2.6)$$

where C_0 is the damping coefficient for the device, $\dot{D}$ is the relative velocity between each end of the device, sgn is the signum function, which in this case, defines the sign of the relative velocity term and α is the velocity exponent of the device. This formula is the same formula as Equation (2.1) given above (Section 15.5.2.2.3) [21].

- The nominal design properties such as stiffness and damping properties of the FVDs must be confirmed during prototype tests and validated through production testing (Section 15.8) [21].
- Each damper type must undergo prototype testing to validate the assumed force-velocity properties (Section 15.8.1) [21].
- Every damper must be production-tested to ensure compliance with the assumed design properties.
- Variations in damping properties in the production tests must not exceed $\pm 10\%$ for average values and $\pm 15\%$ for individual units (Section 15.8.2) [21].
- A third-party structural engineer must review and verify mathematical modeling, seismic force calculations and the testing compliance. This review is mandatory for all retrofit projects with dampers (Section 15.7) [21].

It should be noted that the performance objective for existing school buildings are defined differently in ASCE41-23[21] and TBDY-2018 [2]. School buildings are defined as

Risk Category 4 in ASCE41-23 and the target performance objective is defined as “Life Safety” under BSE-2E earthquake level (975 years return period) and “Immediate Occupancy” under BSE-1E earthquake level (224 years return period). However, for the same building, the performance objective is defined as “Life Safety” under DD1 earthquake level (2475 years return period) and “Immediate Occupancy” under BSE-1E earthquake level (475 years return period) in TBDY-2018. In other words, **existing buildings** which are required to maintain functionality (schools, hospitals etc.) **are evaluated according to a higher level of earthquake according to TBDY-2018 compared to ASCE41-23.**

2.1.3. Design and Modeling of FVDs

While designing a structure with FVDs according to ASCE41-23 [21], spectral response is decreased by a coefficient of B_I , which is defined as damping coefficient used to adjust spectral response for the effect of viscous damping.

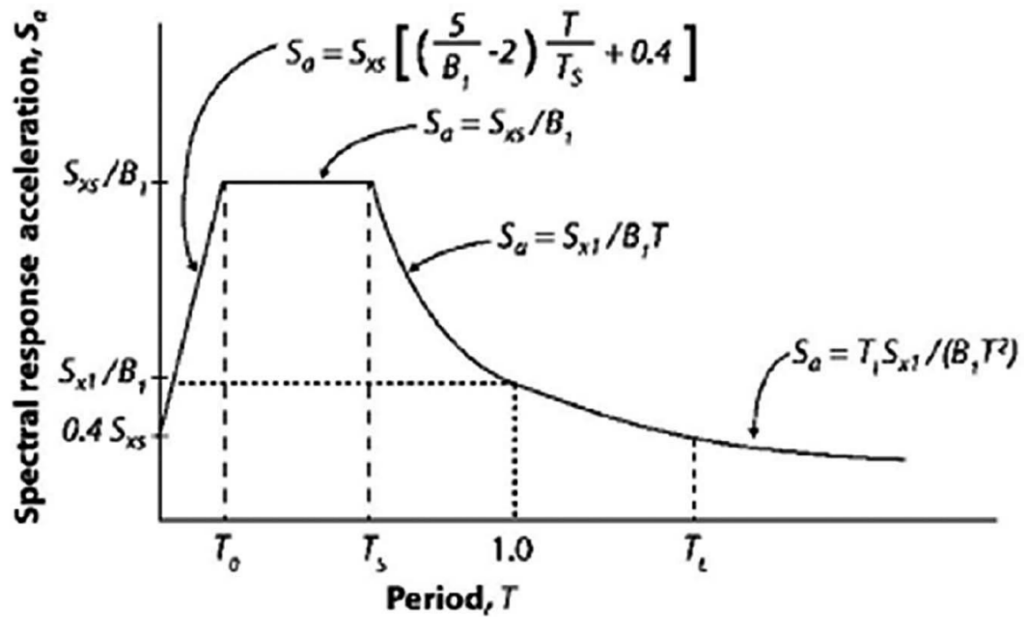


Figure 2.3. General horizontal response spectrum [21].

An estimate in the reduction of dynamic response can be determined using the damping coefficient B_I , defined by ASCE41-23 section 2.3.2.1.[21].

$$B_1 = 4/(5.6 - \ln(100 * \beta)) \quad (2.7)$$

Using the above formula, the following table can be created for different damping ratio values:

Table 2.1. Numerical coefficients for various damping ratios.

Effective Damping, β	Damping Coefficient, B_1
2%	0.82
5%	1.00
10%	1.21
20%	1.54
30%	1.82
37%	2.00
40%	2.09
50%	2.37

According to the demand of the building, which is to be retrofitted, a suitable target effective damping is to be selected using the above equation. Many methods and procedures have been developed and can be used for preliminary sizing and configuration of dampers for buildings [10]. MCEER Technical Report 00-0010 [22] is used as a source document for this study which outlines the procedure for implementation based on the modal strain energy method. This approach necessitates only a modal analysis and a presumed damping configuration to formulate a conceptual design aimed at achieving specific performance objectives. In this study, stiffness proportional damping is used to obtain the preliminary size of dampers. Stiffness proportional damping is a method derived from the modal strain energy method, and it follows the following constitutive relation for determining linear ($\alpha=1$) damper constraints at each floor level in each direction:

$$C_j = \zeta * \frac{k_i}{n_i} * \frac{T}{\pi} * \frac{1}{\cos^2 \theta_j} \quad (2.8)$$

where ζ is the viscous damping ratio, k_i is the story stiffness obtained from static analysis of the building, n_i is the number of dampers in each direction, T is the fundamental period of

the building in related direction, θ_i is the angle of the dampers with the ground (note that this formula assumes all dampers are at the same angle for a particular floor level) [10].

Once the preliminary damper sizes are determined based on the demand of the building, one can define dampers in ETABS software. In this study, ETABS v22.5.0 Software [23] is used for analysis and retrofit design. In ETABS, dampers are modeled using non-linear link elements. For fluid viscous dampers, link type is selected as “Damper-Exponential”. The FVDs will dissipate energy in U1 local axis only. Therefore, only U1 direction is selected for the directional properties. “Effective Stiffness” for “Linear Properties” is selected very small (close to zero stiffness) so that the dampers will not affect the modal properties of the structure. Nonlinear properties of the FVDs are defined in the model and finalized using the values determined by the iterative analyses.

2.2. Near Fault Effects

Ground motions within approximately 20 kilometers of the fault rupture are identified as near-fault ground motions. These motions differ from standard (far-field) ground motions in that they exhibit high-amplitude, long-period pulses within their velocity time histories [24]. This distinct characteristic of near-fault ground motions was first observed by Housner and Trifunac [3] in the seismic records from the 1966 Parkfield earthquake at Cholame as stated above at Section 1.2. In the mentioned study of Housner and Trifunac [3], researchers utilized an extensive array of five accelerographs and fifteen seismoscopes across the San Andreas fault at Cholame and they compare the data taken from each accelerographs during 1966 Parkfield Earthquake in terms of response velocity spectrum and time-history data. One of the major findings from the study is the rapid attenuation of ground motion with distance from the fault, and the transformation of the ground motion's character from pulse-like near the fault to more isotropic as the distance increases.

To develop a design method for the pulse-type ground motions, studies have been carried out on understanding the response of structures to different types of simplified pulses.

Near-fault ground motions are categorized into two types: (a) fling-step motions and (b) forward-directivity motions. Fling-step motions are characterized by a one-sided pulse

in their velocity time-history, leading to a permanent static displacement of the ground. Conversely, forward-directivity motions feature two-sided pulse(s) in their velocity time-history and do not result in any permanent ground displacement. The occurrence of fling-step motions is contingent upon the tectonic displacement along the fault, whereas forward-directivity motions are influenced by the rupture mechanism and the direction of slip relative to the site [24].

Somerville et al. (1997) [4] discusses how forward rupture directivity affects ground motion amplitude and duration and how these effects can be incorporated into empirical attenuation relations. According to Somerville et al. [4] rupture directivity effects cause spatial variations in ground motion amplitude and duration, influencing the strike-normal and strike-parallel components of horizontal ground motion. These effects become significant at periods longer than 0.6 seconds and generally increase with period. The mentioned study develops modifications to empirical strong ground motion attenuation relations to account for the effects of rupture directivity on:

- Average horizontal response spectral acceleration
- Acceleration time history duration
- Ratio of strike-normal to strike-parallel spectral acceleration

The authors introduce model parameters that depend on

- Fraction of the fault rupture that lies between the hypocenter and the site
- Angle between the fault plane and the path from the hypocenter to the site

These parameters allow directivity effects to be included in probabilistic seismic hazard calculations (PSHA).

According to Somerville et al. [4] the forward directivity effect occurs when:

- The rupture propagates toward the site.
- The fault slip direction is aligned with the site.

Somerville et al. [4] introduces mathematical models to quantify these effects based on empirical data.

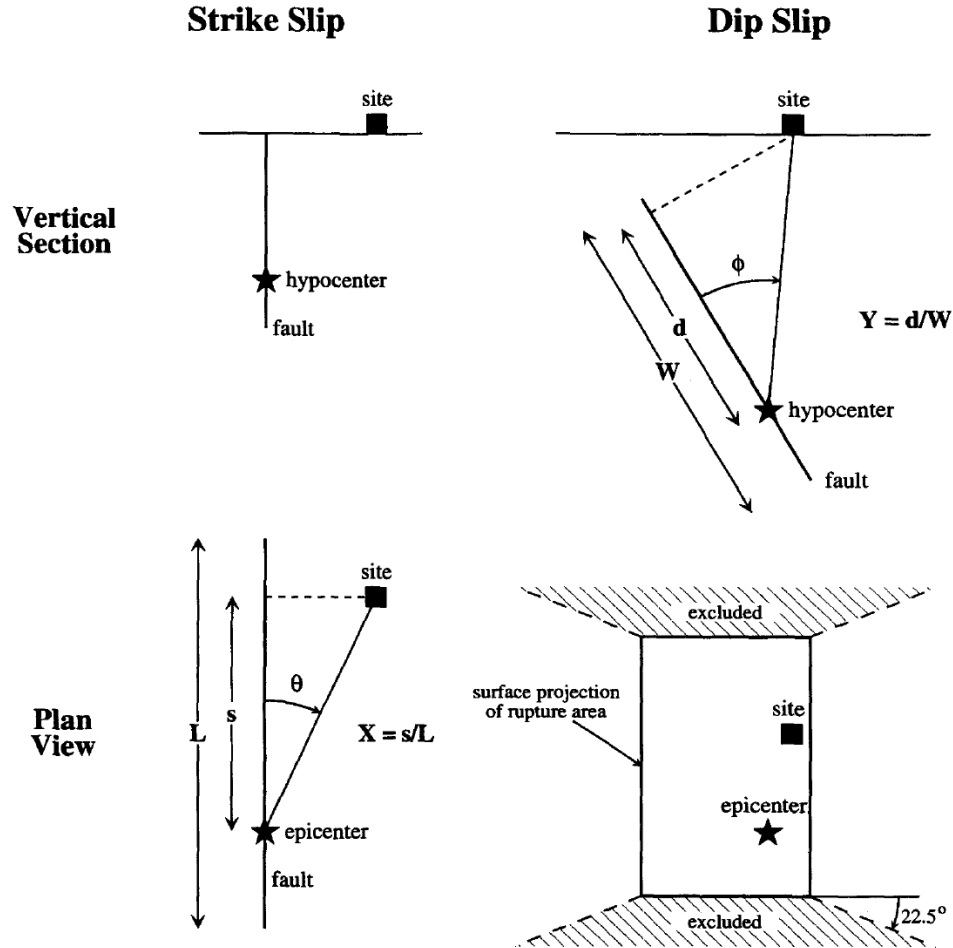


Figure 2.4. Definition of rupture directivity parameters θ and X for strike-slip faults, and ϕ and Y for dip-slip faults, and region off the end of dip-slip faults excluded from the model [4].

The average horizontal response spectral acceleration is modified using the following relations:

$$y = C_1 + C_2 X \cos \theta \quad \text{for strike-slip faults} \quad (2.9)$$

$$y = C_1 + C_2 Y \cos \phi \quad \text{for dip-slip faults} \quad (2.10)$$

where X is the length ratio (fraction of the fault along strike that ruptures toward the site), Y is the width ratio (fraction of the fault up-dip that ruptures toward the site), θ is the azimuth angle between the fault plane and the ray path to the site, ϕ is the zenith angle between the fault plane and the ray path to the site. C_1 and C_2 are regression coefficients that are given in Somerville et al. [4] depending on the period of the structure. The modifications increase ground motion amplitude in forward directivity cases, particularly for strike-slip faults where rupture propagates along the fault plane.

Abrahamson (2000) [5] offers a modification to Somerville [4]'s rupture directivity scaling model. The key modifications include:

- Distance Taper Function: The directivity effect decreases at distances greater than 30 km and vanishes beyond 60 km.

$$T_d(r) = \begin{cases} 1, & r < 30km \\ 1 - \frac{(r - 30)}{30}, & 30km < r < 60km \\ 0, & r > 60km \end{cases} \quad (2.11)$$

- Magnitude Taper Function: Directivity effects are significant for magnitudes $M \geq 6.5$ and vanish below $M = 6.0$.

$$T_m(m) = \begin{cases} 1, & M \geq 6.5 \\ 1 - \frac{(M - 6.5)}{0.5}, & 6.0 \leq M < 6.5 \\ 0, & M < 6.0 \end{cases} \quad (2.12)$$

- Directivity Function Modification: The original directivity model of Somerville et al. [4] is based on X (fault length fraction) and θ (azimuth angle). Abrahamson (2000) [5] states that the empirical data showed that $X \cos(\theta) > 0.6$ was not well-constrained in Somerville [4]'s model. Therefore, Abrahamson (2000) [5] offers that the directivity effect reaches a peak at $X \cos(\theta) = 0.4$ instead of increasing linearly beyond this value.

$$\gamma_{dir}(x, \theta, T) = \begin{cases} C_1(T) + 1.88C_2(T)x \cos(\theta), & X \cos(\theta) \leq 0.4 \\ C_1(T) + 0.75C_2(T), & X \cos(\theta) > 0.4 \end{cases} \quad (2.13)$$

where $C1(T)$ and $C2(T)$ are period-dependent coefficients given in Abrahamson (2000) [5].

Considering the fact that the recordings from past earthquakes reveal that ground shaking at locations near a rupturing fault can be substantially more damaging than from further distances, “California Department of Transportation (Caltrans)” has increased the design response spectrum at locations less than 15 km from a fault to account for the potential severity of shaking since 1992. Near-fault adjustment factors are given in Appendix B of Caltrans Seismic Design Criteria [25].

In Caltrans Seismic Design Criteria [25], near-fault phenomena are commonly combined into a single “near-fault” adjustment factor. As depicted in Figure 2.5, this factor is entirely applied at sites located 15 km or closer to the fault rupture plane, and it linearly decreases to zero at a distance of 25 km. The adjustment involves a 20% enhancement of the spectral values for periods exceeding one second, with this increase progressively reducing to zero at a period of 0.5 second.

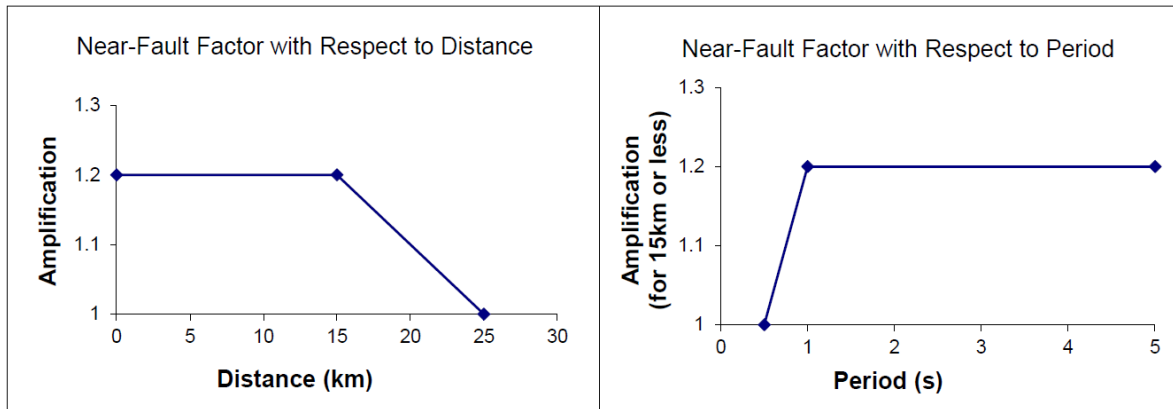


Figure 2.5. Near-Fault adjustment factor as a function of distance and spectral period. The distance measure is based on the closest distance to any point on the fault plane [25].

In Turkish Seismic Code for Buildings (TBDY-2018 [2]) near-fault effects are not covered. However, in the draft version of the mentioned code which was published in 2016 [26], near-fault effects were governed in Section 2.3.3 by using a “near-fault coefficient”

calculated depending on the distance to the fault planes. The coefficient was applied to the DD1 and DD2 level earthquakes only. No criteria for the period is given in this draft version of TBDY-2018, so this γ_F was applied to the whole response spectrum.

$$\gamma_F = \begin{cases} 1.2, & L_F \leq 15km \\ 1.2 - 0.2(L_F - 15), & 15km < L_F \leq 25km \end{cases} \quad (2.14)$$

where L_F is the distance to the fault plane in km, and γ_F is the near-fault coefficient. This section of the draft version of Turkish Seismic Code on Buildings - 2016 [26] was removed in the published version TBDY-2018 [2].

3. METHODOLOGY

Near fault seismic ground motions are frequently characterized by intense velocity and displacement pulses of relatively long period. In order to take these effects into consideration in engineering designs, various approaches are available in the literature. In this study, for selection/matching of the ground motion records, the target acceleration spectrum is determined after evaluation of 4 of these approaches; Sommerville 1997 [4], Abrahamson 2000 [5], Caltrans 2019 [25] and draft version of Turkish Seismic Code 2016 [26]. Ground motion records are selected from the PEER Ground Motion Database [27].

For retrofit design of the building, since the aim was keeping the building under service during construction works not to interrupt the education in the school, viscous dampers are used which can be applied externally to the building and which can eliminate the foundation retrofit works allowing to minimize the construction works inside the building.

CSI ETABS v22.5.0 [23] is used to perform the non-linear time history and pushover analyses. In order to calculate the plastic deformations in the structural elements, the nonlinear behavior of the structural elements was modeled with the concentrated plastic behavior model. Fiber hinge models are used in order to find the cross-sectional properties of plastic hinges in the deformation-controlled analysis approach. FVDs are modeled using link elements. Since relative displacement and related earthquake damage are not expected in the basement floor, shearwalls at the basement are modeled as linear shell elements.

TBDY-2018 Chapter 15 is used for both assessment and design. Since dampers are not covered in TBDY-2018, dampers are modeled and designed according to ASCE41-23 [21] Chapter 15.

4. CASE STUDY: BÜYÜK HALKALI PRIMARY SCHOOL BUILDING

Büyük Halkalı Primary School building was constructed in 1997, with an area of 4,200m². It has 30 classrooms with a capacity of 880 students. The plan dimensions of the building are 45.60x17.80m. The structural system is formed with reinforced concrete moment frames in both directions in the original design. The columns have dimensions of 30x60cm and beams have dimensions of 30x70cm. The building has a very uniform structural system with 3.6m spans in X (N-S) direction. In the Y (E-W) direction, the spans are 6.80m, 4.10m and 7.20m respectively. The building consists of a basement floor, a ground floor and 3 normal floors.

4.1. Information From the Previous Assessment and Retrofit Works

Building's structural performance was assessed in 2006 and shearwalls were added to the structure. In 2019 it was reassessed and additional shearwalls were added to the structure. The assessment and retrofit designs of the building were both performed by Ülker Mühendislik Müşavirlik Ltd. company. The assessment report includes all the information about the building's original situation such as material quality, dimensions of the structural members, detailing in the rebars etc. are taken from and drawings [28].



Figure 4.1. Outside view of the investigated Büyük Halkalı Primary School building

According to Ülker Mühendislik's assessment report [28], following information is given;

Concrete grade : 12.5 MPa (calculated using DBYBHY-2007 [29] Ch 7.2.6.3)

Steel grade : S220

Site class : ZC

Corrosion : N/A

PGA (DD-1) : 0.671 g

PGA (DD-3) : 0.160 g

Site Class : ZC

Allowable Soil Stress : 2.00 kg/cm²

Liquefaction : No liquefaction risk

Subgrade Modulus : 2400 t/m³

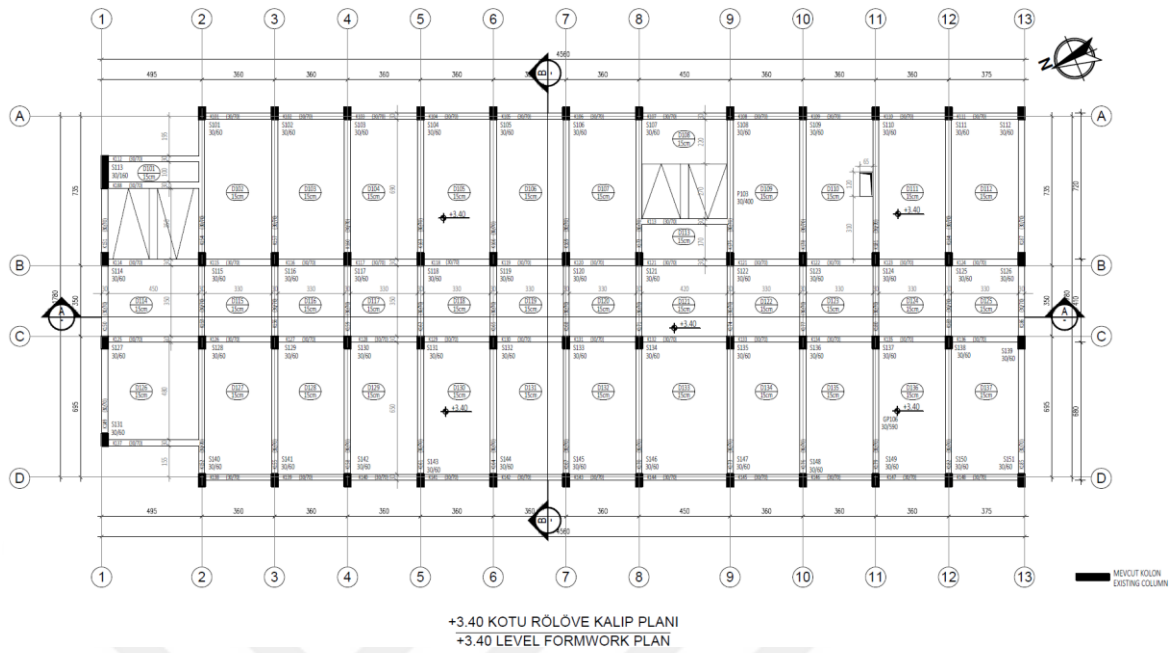


Figure 4.2. Formwork plan view of the Büyük Halkalı Primary School building's original case before retrofit works [28].

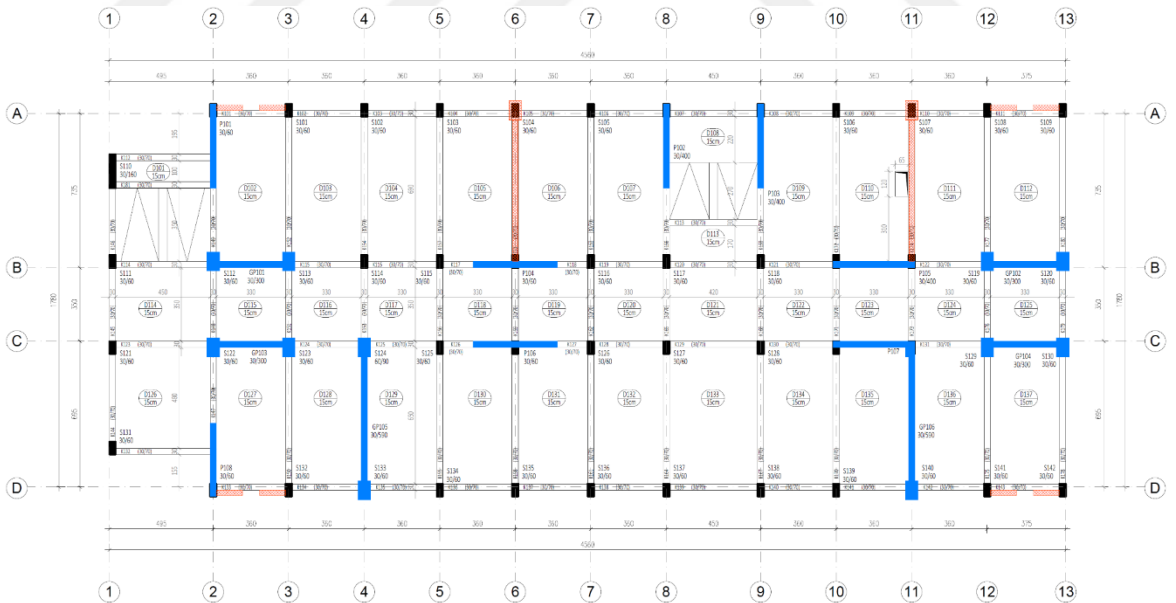


Figure 4.3. Formwork plan view of the Büyük Halkalı Primary School building after shearwalls were added during the retrofit works. Shearwalls added in 2006 are shown with blue color, shearwalls added in 2019 are shown in red color [28].

Service loads on the building are defined as follows:

Table 4.1. Vertical loads on the building defined in the structural model [28].

Dead Loads (G)		
12cm Slab Finishing Weight	COVER	2.00 kN / m ²
20 cm Partition Walls' Weight	WALL	1.50 kN / m ²
15cm Slab Weight	SLAB	3.75 kN / m ²
Roof Weight	ROOF	1.50 kN / m ²
Live Loads (Q)		
Classroom Live Loads	FLL	3.50 kN / m ²
Roof Live Loads	SNOW	0.75 kN / m ²

4.2. Determination of Seismic Parameters

The school building is located in Küçükçekmece, İstanbul, Türkiye. The coordinates of the building are 41.035153°N, 28.790781°E (Figure 4.4). This area is situated in a region that is seismically active. The seismicity of this region is due to the stresses in the fault zone in Northern Anatolia, particularly in the Marmara Region.

The North Anatolian Fault (NAF) is a right-lateral strike-slip transform fault system which separates the Eurasian and Anatolian plates. The North Anatolian Fault Zone (NAFZ) cuts across all of Northern Anatolia, extending from Eastern Anatolia to the Aegean Sea. The NAF zone can be classified into three main parts—the Eastern, Central and Western. The Western NAF bifurcates into two main strands namely the northern and southern strands. The northern strand is the main fault accommodating plate boundary motion through the Marmara Sea, Saros Bay and the North Aegean Trough [30].

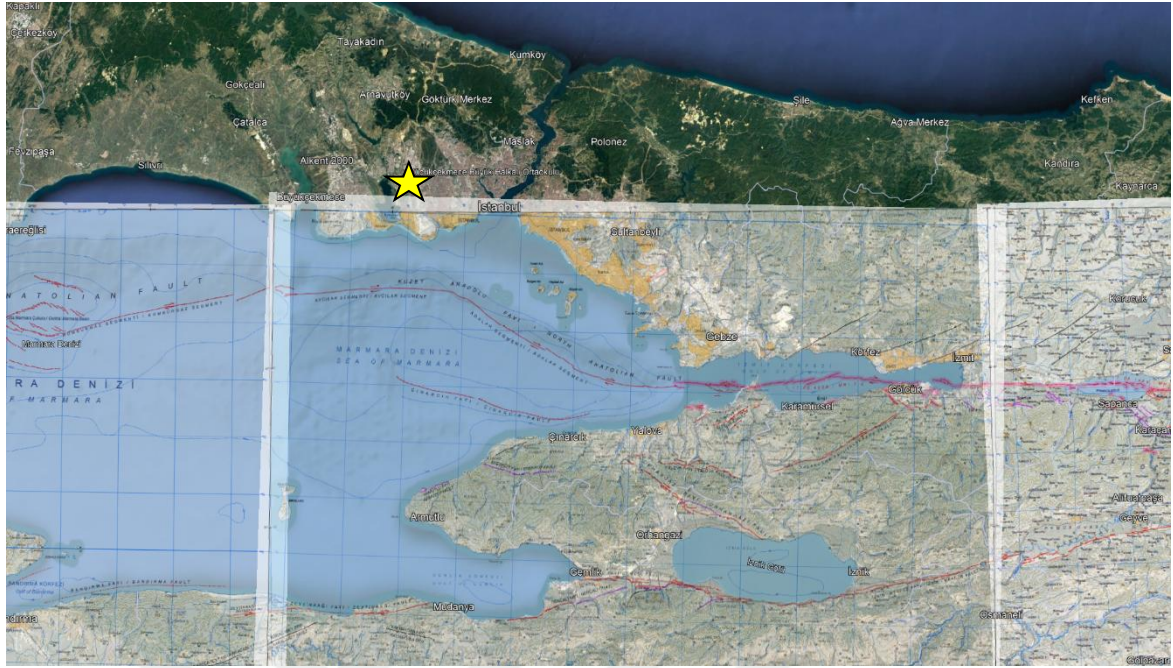


Figure 4.4. Satellite view of Marmara Region taken from Google Earth [31] and superposed Active Fault Map from MTA [32]. Location of investigated Büyük Halkalı Primary School building is shown with yellow star.

The western segment of the NAFZ's northern branch within the Sea of Marmara was ruptured during the 1912 Şarköy-Mürefte earthquake ($M_S=7.4$), while its eastern segment broke during the 1999 Kocaeli earthquake ($M_w=7.6$) [30]. Consequently, it is anticipated that an earthquake in the Sea of Marmara affecting Istanbul would occur on one or several of the as-yet unruptured central segments of this branch [33].

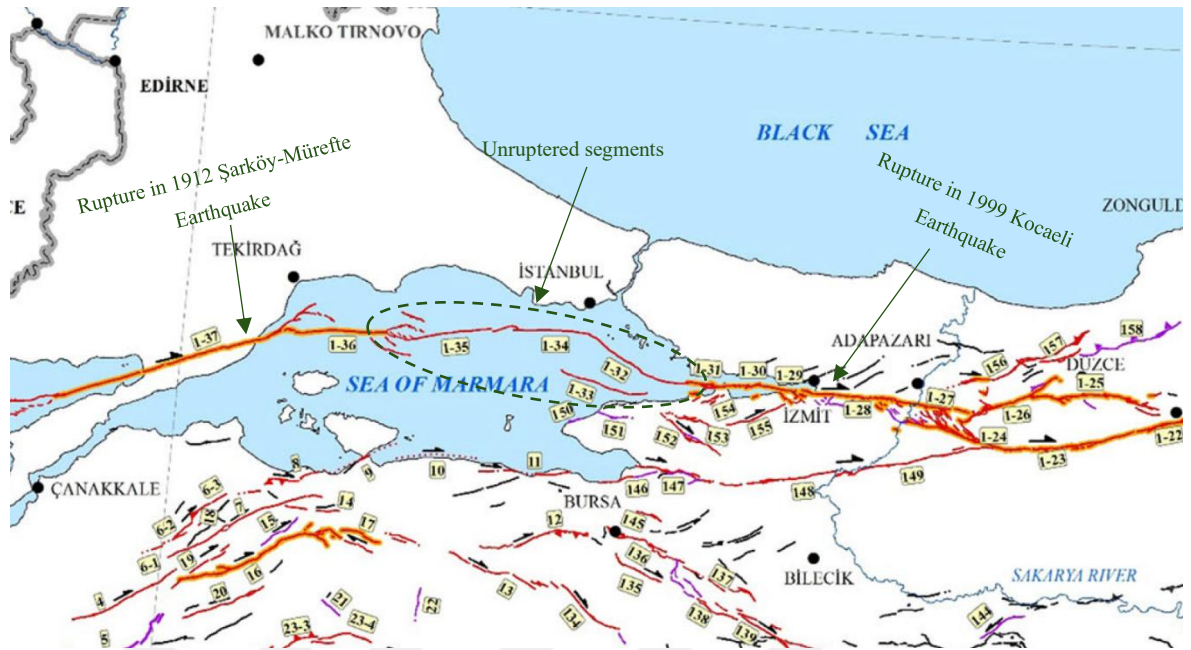


Figure 4.5. Spatial distribution of the active faults of Marmara Region [30].

Table 4.2. Fault parameters for the western segment of the NAFZ's northern branch [30].

Fault					Length	Trend (RHR)		Dip (Degree)		Slip rate (mm/year)		Depth (km)		Magnitude		References
Segment	Type	ID No	AC	(km)	Min	Max	Min	Max	Min	Max	1	2	Obs.	Est.	Mw	
North Anatolian Fault Zone	1999 İzmit Earthquake segment															
	Karadere S	RL	1-26		33,5											
	Arifiye S	RL	1-27		31											
	Tepetarla S	RL	1-28	ER	145	34	228	300	90	90		18	17	1999	7.58	11;12;13;15;16;
	Gölcük S	RL	1-29		15											18;19;20
	Karamürsel S	RL	1-30		14											
	Darıca S	RL	1-31	H	15	265	290	87	90			18	15		6.48	11;12;15;16;19;
	Adalar S	RL	1-32	H	44	92	131	55	85						7.00	20
	Çınarcık S	NN-RL	1-33	H	35	264	295	55	85	22	27	13	15		6.90	20
	Avcılar S	RL	1-34	H	30	64	107	65	85						6.81	11;12;15;16;19;
	Kumburgaz S	RL	1-35	H	39	263	274	85	90			18	17		6.94	20
	1912 Earthquake segment															
	Tekirdağ S	RL	1-36	ER	140	35	263	275	80	90		18	19	1912	6.89	
	Ganos S	RL	1-37			90	247	263	75	90		18	20	Ms 7.4	7.35	11;12;15;16;19;20

In the deterministic ground motion modeling study of the KOERI (2018) [33] earthquake loss estimation, a scenario of an earthquake with a magnitude of Mw 7.5 occurring on the segments of the Main Marmara Fault that have not recently ruptured (shown in Figure 4.5) was used. This scenario is essentially a worst-case scenario that considers the

simultaneous rupture of all fault segments near Istanbul that have not experienced an earthquake in the recent past.

Research has revealed that a 110 km segment of the NAF within the Sea of Marmara has a high probability of generating an earthquake of magnitude 6.5 or greater ([34], [35]).

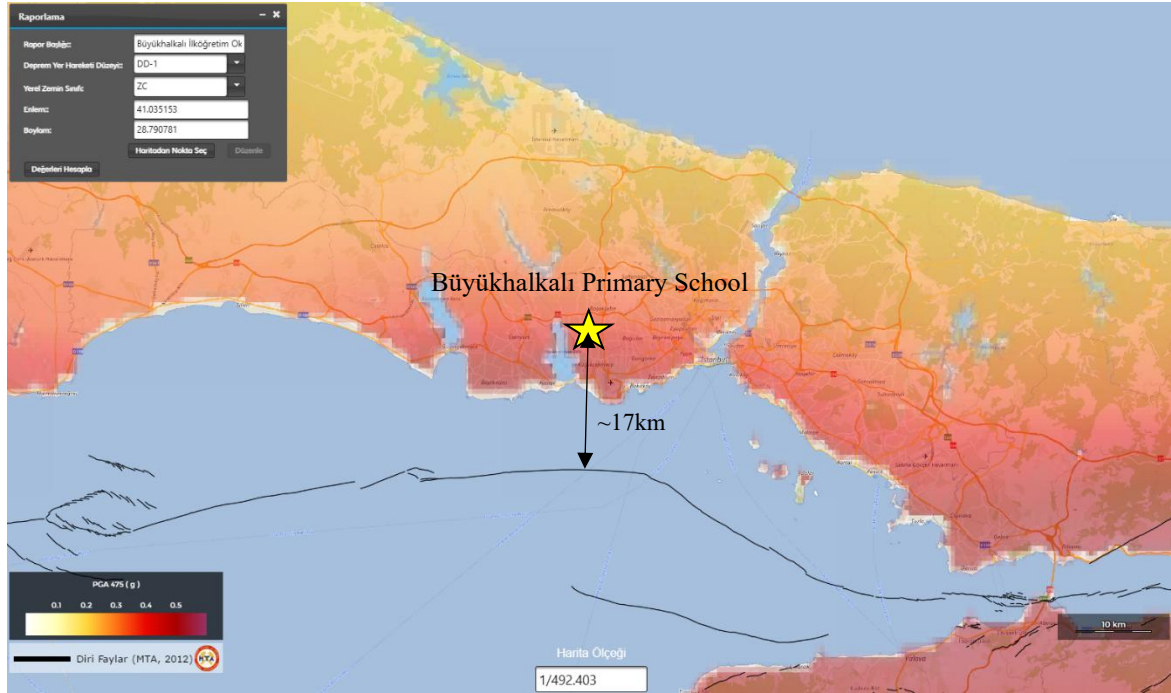


Figure 4.6. The seismic hazard of the region according to the Turkey Earthquake Hazard Map [36] and superposed Active Fault Map from MTA [32].

The design acceleration spectrum has been verified according to the new earthquake hazard maps (shown in above Figure 4.6) published by AFAD under the Turkish Building Seismic Code [2], which came into effect on January 1, 2019. For the Büyükhalkalı School Building located at the coordinates (41.035153°N, 28.790781°E), the expected spectral accelerations for a DD1 level earthquake are as follows: on bedrock S_S : 1.686g, S_1 : 0.472g, and for "C" class soil, the amplified spectral accelerations are S_{DS} : 2.023g, S_{D1} : 0.708g. For a DD3 level earthquake, the accelerations on bedrock are S_S : 0.373g, S_1 : 0.105g, and for "C" class soil, the amplified spectral accelerations are S_{DS} : 0.485g, S_{D1} : 0.157g [36].

Since the investigated building is a school building, the structural performance is evaluated to comply “Controlled damage (KH)” under DD-1 (2475 years return period) earthquake level and “Limited damage (SH)” under DD-3 (72 years return period) earthquake level according to TBDY-2018 Chapter 3.5.1.

Table 4.3. Seismic parameters given by Türkiye Seismic Hazard Maps of AFAD [36] for the Büyük Halkalı School Building’s location.

Ground Motion Level	DD-1	DD-2	DD-3
S_s (g)	1.686	0.967	0.373
S₁ (g)	0.472	0.269	0.105
F_s	1.20	1.20	1.30
F₁	1.50	1.50	1.50
S_{DS} (g)	2.023	1.160	0.485
S_{D1} (g)	0.708	0.404	0.158
T_A	0.070	0.070	0.065
T_B	0.350	0.348	0.325

The elastic response acceleration spectrum considering above parameters is stated in below Figure 4.7.

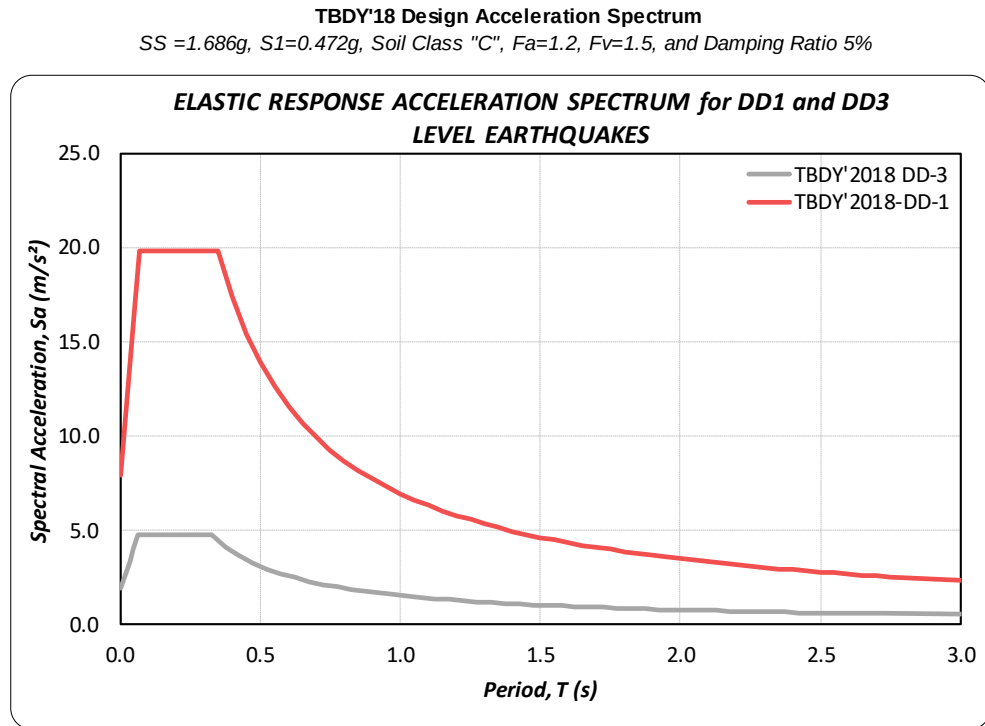


Figure 4.7. The elastic response acceleration spectrum defined according to TBDY-2018.

The school building evaluated is 17 km away from the North Anatolian Fault and is in the expected direction of the fault rupture. Therefore, it has been deemed necessary to consider the near-fault effects in the calculations conducted within the scope of this study.

In order to take forward-directivity effect into account, 4 rupture scenarios for the nearest faults are evaluated and the worst-case scenario is selected to be used in calculating the modified response acceleration spectrum according to Sommerville [4] and Abrahamson [5] models. The rupture of Adalar Segment (1-32) and Avcılar Segment (1-34) [30] are considered in the scenarios. Properties of these faults taken from Emre et al.[30] are given in the table below:

Table 4.4. Fault parameters for Adalar and Avcılar segments of the NAFZ [30].

	Adalar	Avcılar
Segment No	1-32	1-34
ϕ_{min}	55°	65°
ϕ_{max}	85°	85°

	Adalar	Avcılar
M	7.00	6.81
Length, L	44	30

C_1 and C_2 values given in Table 4 of Somerville (1997) [4] that are used in the calculations are listed as follows:

Table 4.5. C_1 and C_2 values given in Table 4 of Somerville (1997) [4].

Period (Sec.)	C_1 (Const.)	C_2 (Const.)
0.60	0.000	0.000
0.75	-0.084	0.185
1.00	-0.192	0.423
1.50	-0.344	0.759
2.00	-0.452	0.998
3.00	-0.605	1.333
4.00	-0.713	1.571
5.00	-0.797	1.757

4.2.1. Rupture Scenario 1

In the first scenario, it is assumed that the rupture starts from the west end of the Avcılar Segment and continues towards east (see below Figure 4.8). In this case, θ is measured as 36° and s is measured as $\sim 23\text{km}$. Since it is given in Somerville (1997) [4] that $X=s/L$, then $X = 0.767$ and $X\cos\theta = 0.620$.

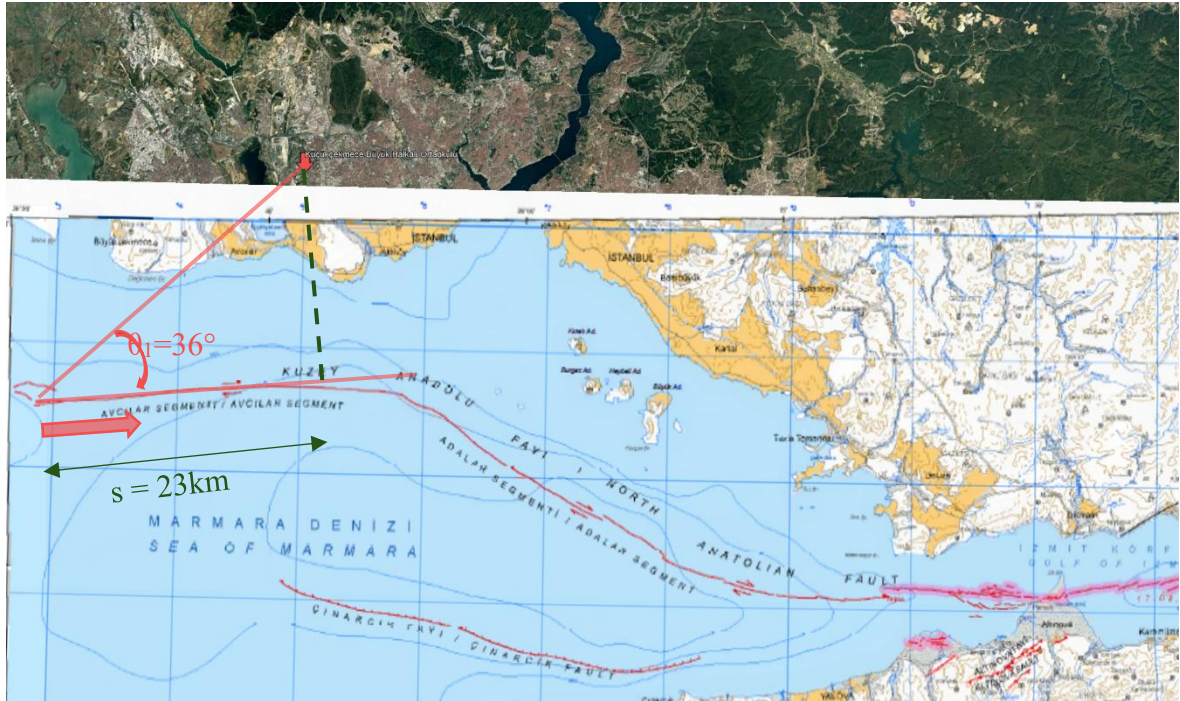


Figure 4.8. Satellite view of Marmara Region taken from Google Earth [31] and superposed Active Fault Map from MTA [32]. Rupture scenario 1 considered in order to calculate the forward directivity.

Using C_1 and C_2 values given in Table 4.5, the response spectrum modification factors are calculated as follows:

Table 4.6. Response spectrum modification factors for “Rupture Scenario-1” calculated according to Somerville (1997) [4] and Abrahamson (2000) [5].

Period (Sec.)	Somerville (1997) [4]		Abrahamson (2000) [5]	
	$\ln(y_1)$	y_1	$\ln(y_1)$	y_1
0.60	0.000	1.000	0.000	1.000
0.75	0.031	1.031	0.055	1.056
1.00	0.070	1.073	0.125	1.133
1.50	0.127	1.135	0.225	1.253
2.00	0.167	1.182	0.297	1.345
3.00	0.222	1.248	0.395	1.484
4.00	0.261	1.299	0.465	1.592
5.00	0.293	1.340	0.521	1.683

4.2.2. Rupture Scenario 2

In the second scenario, it is assumed that the rupture starts from the east of the Avclar Segment and continues towards west (see below Figure 4.9). In this case, θ is measured as 65° and s is measured as $\sim 8\text{km}$. Using these data, it is calculated that $X = 0.267$ and $X\cos\theta = 0.113$.

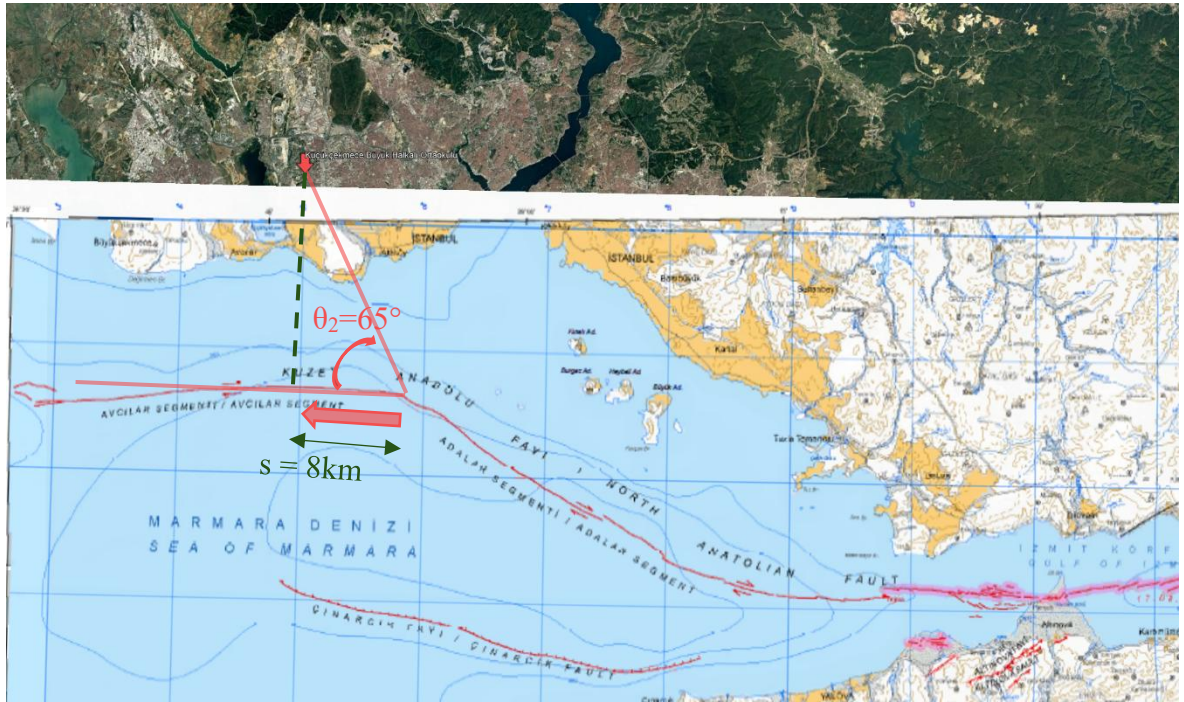


Figure 4.9. Satellite view of Marmara Region taken from Google Earth [31] and superposed Active Fault Map from MTA [32]. Rupture scenario 2 considered in order to calculate the forward directivity.

Using C_1 and C_2 values given in Table 4.5, the response spectrum modification factors are calculated as follows:

Table 4.7. Response spectrum modification factors for “Rupture Scenario-2” calculated according to Somerville (1997) [4] and Abrahamson (2000) [5].

Period (Sec.)	Somerville (1997) [4]		Abrahamson (2000) [5]	
	$\ln(y_2)$	y_2	$\ln(y_2)$	y_2
0.60	0.000	1.000	0.000	1.000
0.75	-0.063	0.939	-0.045	0.956
1.00	-0.144	0.866	-0.102	0.903
1.50	-0.258	0.772	-0.183	0.833
2.00	-0.340	0.712	-0.241	0.786
3.00	-0.455	0.635	-0.323	0.724
4.00	-0.536	0.585	-0.380	0.684
5.00	-0.599	0.549	-0.425	0.654

As can be seen in the table above, this scenario gives modification factors lower than 1.00 which means in this scenario near-fault effects will not govern for the location of Büyük Halkalı Primary School.

4.2.3. Rupture Scenario 3

In the third scenario, it is assumed that the rupture starts from the south of the Prince Islands at the east end of the Avcılar Segment and continues towards west (see below Figure 4.10). In this case, θ is measured as 18° and s is measured as $\sim 27\text{km}$. Using these data, it is calculated that $X = 0.614$ and $X\cos\theta = 0.584$.

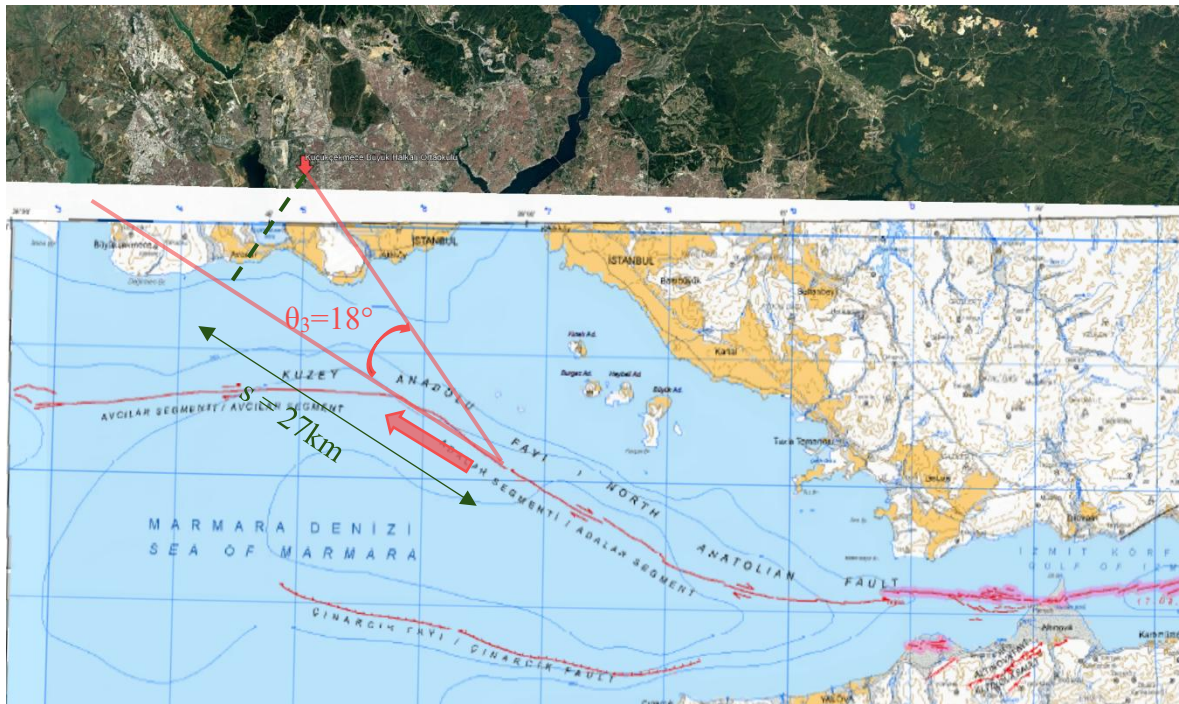


Figure 4.10. Satellite view of Marmara Region taken from Google Earth [31] and superposed Active Fault Map from MTA [32]. Rupture scenario 3 considered in order to calculate the forward directivity.

Using C_1 and C_2 values given in Table 4.5, the response spectrum modification factors are calculated as follows:

Table 4.8. Response spectrum modification factors for “Rupture Scenario-3” calculated according to Somerville (1997) [4] and Abrahamson (2000) [5].

Period (Sec.)	Somerville (1997) [4]		Abrahamson (2000) [5]	
	$\ln(y_3)$	y_3	$\ln(y_3)$	y_3
0.60	0.000	1.000	0.000	1.000
0.75	0.024	1.024	0.055	1.056
1.00	0.055	1.056	0.125	1.133
1.50	0.099	1.104	0.225	1.253
2.00	0.130	1.139	0.297	1.345
3.00	0.173	1.189	0.395	1.484
4.00	0.204	1.226	0.465	1.592
5.00	0.228	1.257	0.521	1.683

As can be seen in the table above, this scenario gives modification factors lower than 1.00 which means in this scenario near-fault effects will not govern for the location of Büyük Halkalı Primary School.

4.2.4. Rupture Scenario 4

In the fourth scenario, it is assumed that the rupture starts from the east end of the Adalar Segment and continues towards west (see below Figure 4.10). The angle of directivity and the length s is measured from the point where the Adalar segment bends towards north-west to the Büyükhalkalı School Buildings's location. In this case, θ is measured as 15° and s is measured as ~ 43 km. Using these data, it is calculated that $X = 0.977$ and $X \cos \theta = 0.944$.

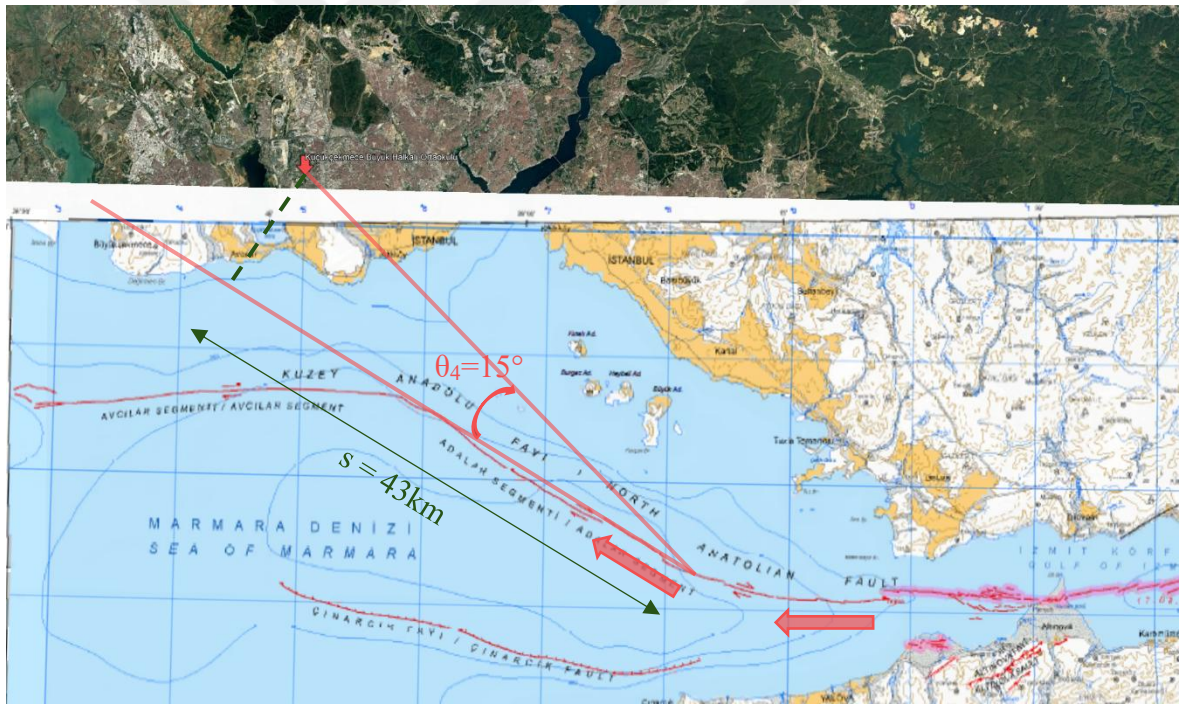


Figure 4.11. Satellite view of Marmara Region taken from Google Earth [31] and superposed Active Fault Map from MTA [32]. Rupture scenario 4 considered in order to calculate the forward directivity.

Using C_1 and C_2 values given in Table 4.5, the response spectrum modification factors are calculated as follows:

Table 4.9. Response spectrum modification factors for “Rupture Scenario-4” calculated according to Somerville (1997) [4] and Abrahamson (2000) [5].

Period (Sec.)	Somerville (1997) [4]		Abrahamson (2000) [5]	
	$\ln(y_4)$	y_4	$\ln(y_4)$	y_4
0.60	0.000	1.000	0.000	1.000
0.75	0.091	1.095	0.055	1.056
1.00	0.207	1.230	0.125	1.133
1.50	0.372	1.451	0.225	1.253
2.00	0.490	1.632	0.297	1.345
3.00	0.653	1.922	0.395	1.484
4.00	0.770	2.160	0.465	1.592
5.00	0.862	2.367	0.521	1.683

4.2.5. Near-Fault Effects According to Caltrans Seismic Design Criteria [25] and Draft Version of TBDY [26]

Since the distance of the building to the nearest fault segment is measured as 17km, the distance factor according to Figure 2.5 is calculated as 1,16 by interpolation. By multiplying the distance factor with the period factor, the resulting Near-Fault factors are calculated for each period value according to Caltrans Seismic Design Criteria [25] and given in below Table 4.10.

In the draft version of TBDY [26], the only criteria for calculating the near fault effects was the distance to the fault plane. Therefore, the Near-Fault factor γ_F is calculated as 1.16 for 17km distance.

Table 4.10. Near-fault factors according to Caltrans Seismic Design Criteria [25] and Draft Version of TBDY [26].

Period (Sec.)	Caltrans [25]	Draft Version of TBDY [26]
0.60	1.16	1.16
0.75	1.25	1.16
1.00	1.39	1.16
1.50	1.39	1.16
2.00	1.39	1.16
3.00	1.39	1.16

Period (Sec.)	Caltrans [25]	Draft Version of TB DY [26]
4.00	1.39	1.16
5.00	1.39	1.16

4.2.6. Resulting Elastic Acceleration Spectrum

The modified elastic response acceleration spectrum for DD-1 level earthquake considering near fault modification factors is stated in below Figure 4.12.

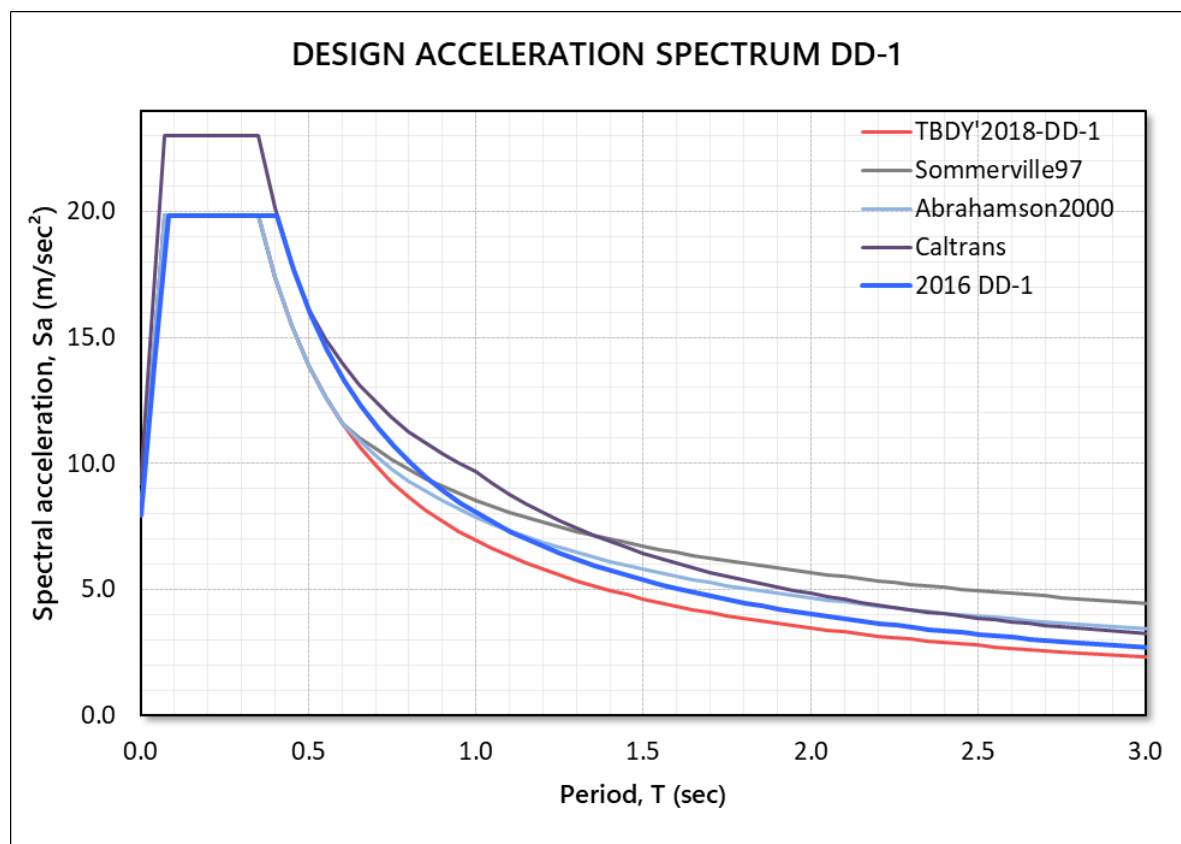


Figure 4.12. Comparison of the acceleration spectrum given in TB DY-2018 and the acceleration spectrum considering near fault effects according to different approaches.

The envelope of the above acceleration spectrum in Figure 4.12 is used as the target spectra in this study excluding Somerville 1997 [4] since it gives very high values after 1.5sec period.

Using the above-mentioned target spectra, 11 set of ground motion records compatible with the site and fault conditions are selected from PEER Ground motion Database NGA-West2 [27].

Table 4.11. Summary of PEER ground motion database search criteria [27].

Magnitude Min	6.5
Magnitude Max	7.5
Rjb Min (km)	0
Rjb Max (km)	35
Vs30 Min (m/sec)	360
Vs30 Max (m/sec)	700
Scale Factor Min	0.6
Scale Factor Max	3
Period Array	0.2,2,3
Weight Array	1,1,1
Pulse	3
Damping Ratio	0.05
Scaling Method	Minimize MSE
Suite Average	Arithmetic
Component	SRSS
Fault Type	SS+Reverse

Selected ground motion records are given in Table 4.12 below.

Table 4.12. Near-fault factors according to Caltrans Seismic Design Criteria [25] and Draft Version of TBDY [26].

Result ID	Rec. Seq. No	Earthquake Name	Year	M_w	R_{jb} (km)	V_{s30} (m/sec)	FM	SF	D_{5-95%}	I_A (m/sec)
1	RSN139	Tabas_ Iran	1978	7.4	0	472	RV	2.86	11.3	1.4
2	RSN779	Loma Prieta	1989	6.9	0	595	RO	1.06	10.2	7.9
3	RSN802	Loma Prieta	1989	6.9	7.58	381	RO	2.30	9.4	1.5
4	RSN825	Cape Mendocino	1992	7.0	0	568	RV	1.33	9.7	6
5	RSN983	Northridge-01	1994	6.7	0	526	RV	1.07	6.9	6.5
6	RSN3750	Cape Mendocino	1992	7.0	23.46	516	RV	2.86	15.3	0.9
7	RSN4031	San Simeon_ CA	2003	6.5	5.07	411	RV	2.29	10.3	1.9
8	RSN4219	Niigata_ Japan	2004	6.6	0.49	480	RV	1.08	8.4	8.8

Result ID	Rec. Seq. No	Earthquake Name	Year	M_w	R_{jb} (km)	V_{s30} (m/sec)	FM	SF	D_{5-95%}	I_A (m/sec)
9	RSN4228	Niigata_ Japan	2004	6.6	6.27	375	RV	1.99	12.2	2.2
10	RSN5813	Iwate_ Japan	2008	6.9	7.82	413	RV	2.74	40.6	2.1
11	RSN5818	Iwate_ Japan	2008	6.9	12.83	512	RV	1.44	15.1	7.3

Among the selected records, RSN802, RSN983 and RSN4228 include pulse effects.

Selected ground motion records are matched with the target spectra using Seismosoft SeismoMatch 2023 software [37] and stated in below Figure 4.13. Ground motion parameters for matched accelerograms are stated in below Table 4.13 and Table 4.14.

After matching, the velocity content of the accelerograms have changed so that the maximum velocities of the pulse-like records are not the highest in the list.

Table 4.13. Ground motion parameters for matched accelerograms. Pulse-like records are highlighted with red color.

Accelerogram	1- RSN139_X	2- RSN139_Y	3- RSN779_X	4- RSN779_Y	5- RSN802_X	6- RSN802_Y	7- RSN825_X	8- RSN825_Y	9- RSN983_X	10- RSN983_Y	11- RSN3750_X	12- RSN3750_Y
Max Acceleration (g)	0.97	1.08	1.02	0.91	1.17	1.02	1.33	1.42	1.01	1.07	1.00	0.79
Max Acceleration SRSS (g)	1.46		1.37		1.55		1.95		1.47		1.28	
Max Velocity (cm/sec)	93.95	83.39	120.73	67.94	96.72	166.52	96.89	88.67	126.50	71.11	134.49	87.95
Max Velocity SRSS (cm/sec)	125.62		138.54		192.57		131.34		145.12		160.70	
Max Displacement (cm)	38.34	30.55	56.89	24.42	38.20	121.39	60.50	71.86	159.97	29.27	148.70	56.93
Max Displacement SRSS (cm)	49.02		61.91		127.26		93.94		162.65		159.22	
Vmax/Amax (sec)	0.10	0.08	0.12	0.08	0.08	0.17	0.07	0.06	0.13	0.07	0.14	0.11
Acceleration RMS (g)	0.22	0.23	0.18	0.18	0.13	0.15	0.11	0.12	0.18	0.16	0.19	0.16
Velocity RMS (cm/sec)	27.99	29.51	29.40	21.22	17.98	34.14	14.90	16.49	33.50	20.96	60.81	30.61
Displacement RMS (cm)	20.69	12.36	15.75	7.89	9.99	32.69	35.95	41.78	80.00	10.21	61.44	24.41
Arias Intensity	15.29	16.50	12.69	12.78	10.11	13.41	5.88	6.77	14.00	11.29	15.53	11.89
Characteristic Intensity	0.46	0.49	0.39	0.39	0.29	0.36	0.21	0.23	0.40	0.34	0.43	0.36
Specific Energy Density (cm2/sec)	16432	18271	21612	11253	12931	46603	6657	8151	32126	12581	106027	26877
Cum. Abs. Velocity (cm/sec)	3272	3204	2799	2665	2538	3088	1524	1702	3081	2663	3775	3081
Acc Spectrum Intensity (g*sec)	0.91	0.98	0.91	0.92	0.89	0.92	0.88	0.90	0.89	0.91	0.89	0.91
Vel Spectrum Intensity (cm)	365	369	348	346	373	362	319	362	364	357	345	348
Sustained Max.Acceleration (g)	0.83	0.89	0.86	0.85	0.78	0.94	0.43	0.38	0.90	0.75	0.90	0.71
Sustained Max.Velocity (cm/sec)	68.76	77.84	101.61	63.12	54.96	149.30	46.49	52.88	97.17	65.27	128.21	83.10
Effective Design Acceleration (g)	0.91	0.98	0.93	0.78	1.08	0.94	1.38	0.91	0.91	0.89	0.91	0.70
Significant Duration (sec)	15.82	11.60	10.65	8.88	9.78	11.62	6.36	6.30	10.66	10.34	18.54	17.89

Table 4.14. Ground motion parameters for matched accelerograms. Pulse-like records are highlighted with red color. (cont.).

Accelerogram	13- RSN4031_X	14- RSN4031_Y	15- RSN4219_X	16- RSN4219_Y	17- RSN4228_X	18- RSN4228_Y	19- RSN5813_X	20- RSN5813_Y	21- RSN5818_X	22- RSN5818_Y
Max Acceleration (g)	0.85	1.01	0.88	1.02	1.20	1.13	0.91	1.05	1.03	0.86
Max Acceleration SRSS (g)	1.31		1.35		1.65		1.39		1.34	
Max Velocity (cm/sec)	82.89	65.03	70.09	108.96	93.52	81.68	68.96	93.99	99.92	101.59
Max Velocity SRSS (cm/sec)	105.35		129.56		124.17		116.57		142.49	
Max Displacement (cm)	32.27	30.04	30.49	36.99	22.29	32.09	30.01	49.21	48.39	48.76
Max Displacement SRSS (cm)	44.08		47.93		39.07		57.64		68.70	
Vmax/Amx (sec)	0.10	0.07	0.08	0.11	0.08	0.07	0.08	0.09	0.10	0.12
Acceleration RMS (g)	0.09	0.08	0.07	0.08	0.06	0.06	0.10	0.11	0.12	0.12
Velocity RMS (cm/sec)	12.75	11.49	8.73	9.70	7.01	8.19	16.91	21.12	12.11	17.20
Displacement RMS (cm)	6.14	4.89	3.26	4.59	6.28	4.16	8.94	11.70	8.63	9.07
Arias Intensity	13.11	10.47	15.04	15.82	8.82	8.54	18.42	21.89	24.97	26.30
Characteristic Intensity	0.28	0.24	0.27	0.28	0.18	0.18	0.35	0.39	0.43	0.45
Specific Energy Density (cm ² /sec)	16427	13356	13708	16934	8834	12077	34327	53532	17595	35480
Cum. Abs. Velocity (cm/sec)	3605	3162	4126	4326	2687	2947	6336	6474	6085	6214
Acc Spectrum Intensity (g*sec)	0.89	0.89	0.92	0.89	0.90	0.89	0.88	0.89	0.90	0.97
Vel Spectrum Intensity (cm)	356	352	354	348	351	365	339	334	348	362
Sustained Max.Acceleration (g)	0.76	0.84	0.84	0.84	0.87	0.80	0.80	0.97	0.90	0.86
Sustained Max.Velocity (cm/sec)	71.58	54.20	62.35	64.19	49.44	68.78	60.80	80.94	73.40	89.37
Effective Design Acceleration (g)	0.77	0.88	0.82	1.00	1.19	1.01	0.82	0.87	0.93	0.85
Significant Duration (sec)	18.13	19.37	16.01	22.42	9.00	12.40	42.46	37.63	20.35	20.51

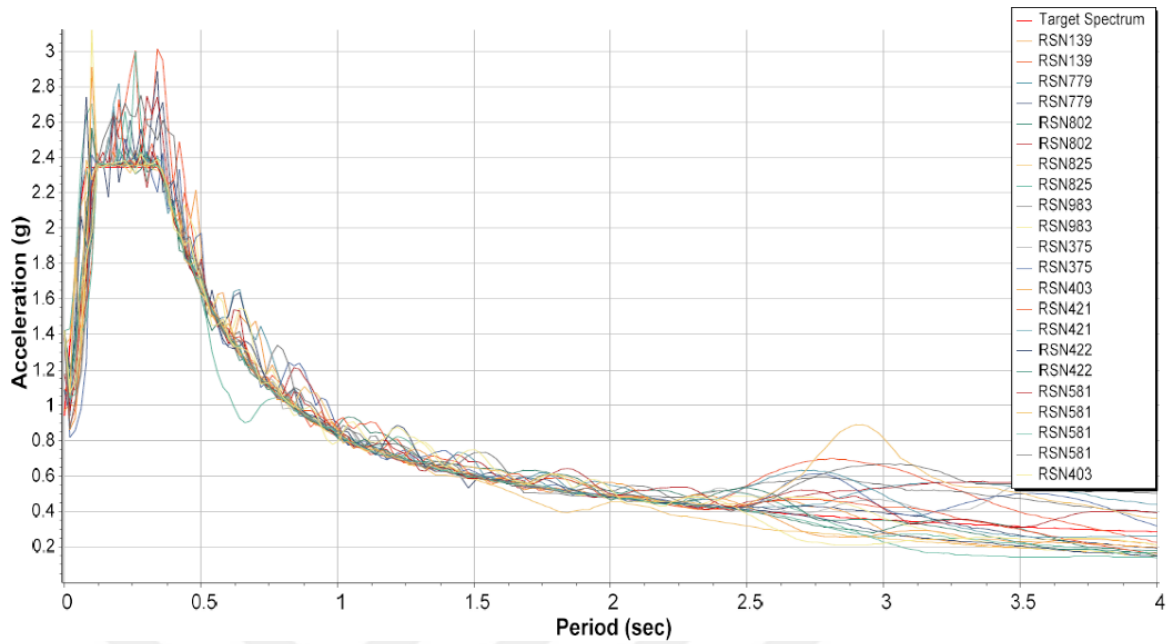


Figure 4.13. Acceleration spectrum of the matched ground motion records [37].

4.3. Modelling and Analysis of the Building's Original State in 1997

The structural 3-D model of the original state of the school building, when it was built in 1997, is created using "ETABS v22.5.0" [23] software. In the model, beams and columns are idealized and modeled using frame elements. The bending stiffness of the cracked section is taken into account in all sections of reinforced concrete elements. Furthermore, the nonlinear behaviors of the structural elements were modeled with Concentrated Inelasticity Fiber Elements model to calculate the plastic deformations in the structural elements.

Initially, the building was modeled in its existing state, and the existing reinforcements were defined for all elements. The models were analyzed according to TBDY-2018, and deficiencies in the structure were identified based on this regulation.

4.3.1. Material Properties

In the assessment report of Ülker Mühendislik [28] the material quality was given as 12.5 MPa. However, the concrete core tests are re-evaluated in the scope of this study

according to Section 15.2.4.3 of TBDY-2018 [2]. By eliminating the very low value from the average, the concrete quality is found as 14.7MPa and this value is used in the analysis.

Table 4.15. Concrete core sample results taken from assessment report of Ülker Mühendislik [28].

Floor	D _{core} (mm)	h _{core} (mm)	F _{l/d}	F _{dia}	F _{mc}	F _d	f _c (MPa)	Corrected f _c (MPa)
Basement	73.0	73.0	0.8816	1.03	1.00	1.06	26.91	25.96
Basement	73.0	73.0	0.8801	1.03	1.00	1.06	23.53	22.66
Basement	73.0	73.0	0.8797	1.03	1.00	1.06	22.49	21.65
Ground	73.0	73.0	0.8794	1.03	1.00	1.06	21.88	21.05
Ground	73.0	73.0	0.8774	1.03	1.00	1.06	17.09	16.41
Ground	73.0	73.0	0.8779	1.03	1.00	1.06	18.34	17.62
1 st	73.0	73.0	0.8763	1.03	1.00	1.06	14.55	13.96
1 st	73.0	73.0	0.8749	1.03	1.00	1.06	11.30	10.82
1 st	73.0	73.0	0.8777	1.03	1.00	1.06	17.95	17.24
2 nd	73.0	73.0	0.8762	1.03	1.00	1.06	14.44	13.85
2 nd	73.0	73.0	0.8772	1.03	1.00	1.06	16.78	16.10
2 nd	73.0	73.0	0.8755	1.03	1.00	1.06	12.90	12.36
3 rd	73.0	73.0	0.8758	1.03	1.00	1.06	13.40	12.84
3 rd	73.0	73.0	0.8769	1.03	1.00	1.06	16.03	15.38
3 rd	73.0	73.0	0.8770	1.03	1.00	1.06	16.19	15.53

Table 4.16. Concrete strength calculated according to TBDY-2018 [2].

Min.	10.82 MPa
0.75*Ave.	13.00 MPa
Ave.	17.33 MPa
Std.Dev.	4.05 MPa
Ave. - Std.Dev.	13.28 MPa
0.85*Ave.	14.73 MPa
Concrete Strength	14.73 MPa

In the same assessment report of Ülker Mühendislik [28], the reinforcement quality is given as S220 ($f_y=220\text{MPa}$), and the hooks of the stirrups are given as 90° .

4.3.2. Section Properties

In the assessment report of Ülker Mühendislik [28], the column and beam sections are given as shown in Figure 4.14. The same sections are used in this study.

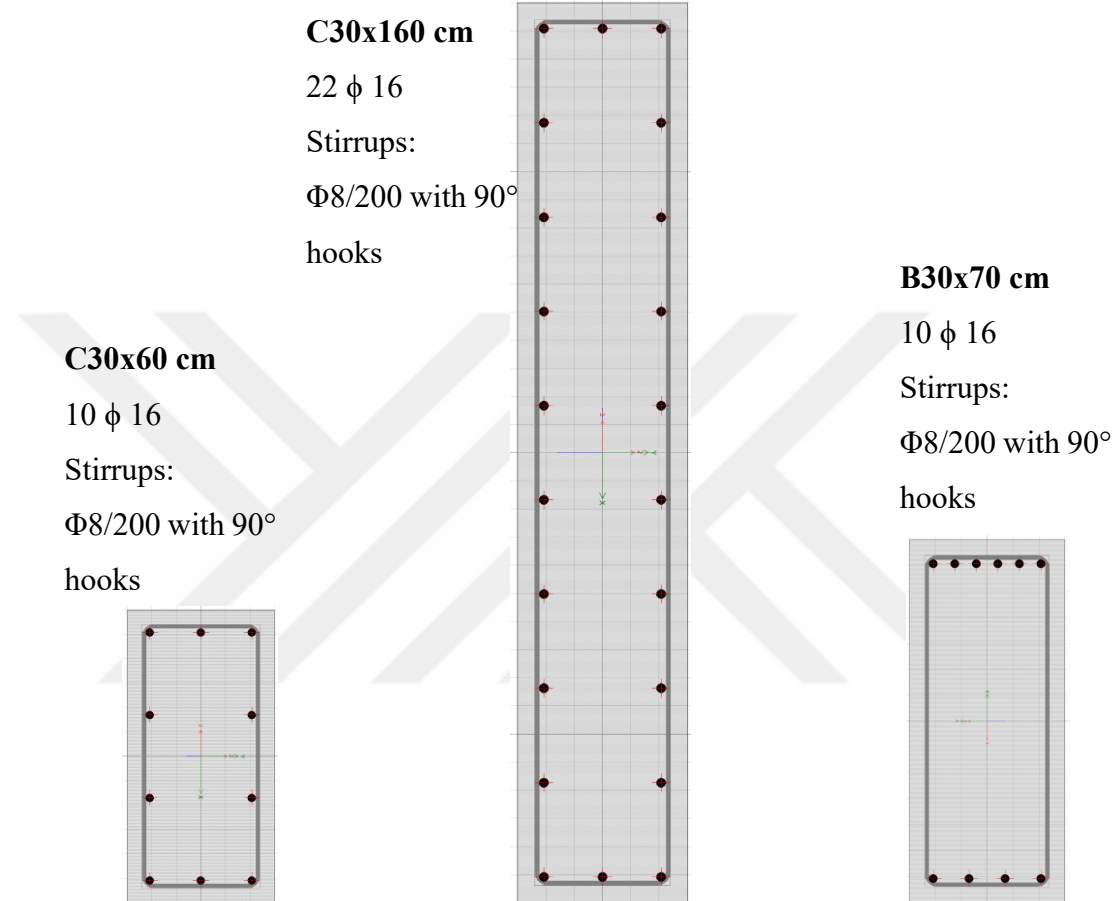


Figure 4.14. Column and beam sections in the assessment report [28].

4.3.3. Summary of Analysis Parameters

Parameters used in the analysis are summarized in the table below:

Table 4.17. Analysis parameters used in the structural model.

Transverse Reinforcement	S220 ($f_y=220\text{MPa}$)
Longitudinal Reinforcement	S220 ($f_y=220\text{MPa}$)
Concrete Strength	14.7 MPa
Live Load Participation Factor (n)	0.3

Knowledge Level Coefficient (Comprehensive)	1
Modulus of Elasticity (Concrete)	26461 MPa
Building Usage Class (BKS)	BKS-1
Building Importance Factor (I)	1.5
Earthquake Design Class (DTS)	1a

The cracked section stiffness in columns has been taken as 0.70 and in beams as 0.35. Under earthquake effects, it is assumed that there will be no significant uplifts or soil failures (reaching soil bearing capacity) at the foundations that could affect the dynamic behavior of the structure. In the analysis, columns and shear walls are modeled as elements fixed to the foundation.

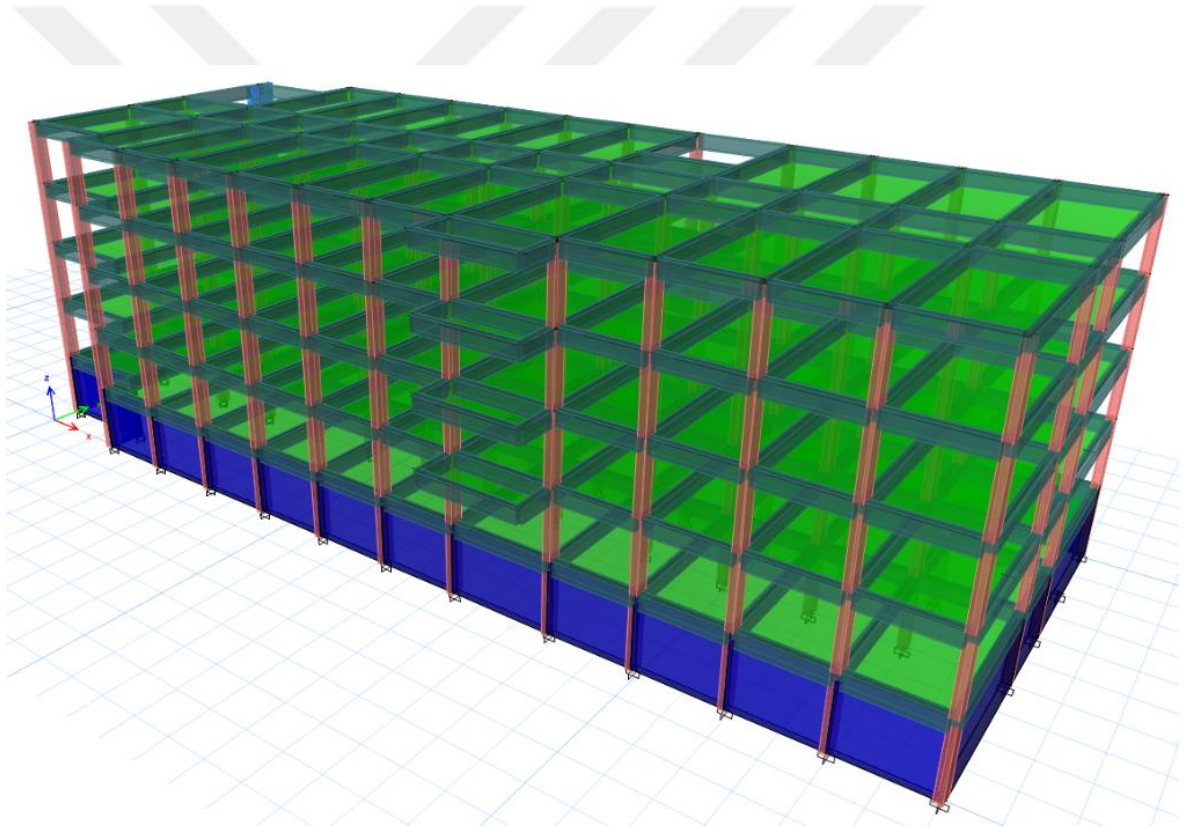


Figure 4.15. 3-D view of the ETABS model of the structure

4.3.4. Modal Analysis Results

The periods and mass participation factors of the building are determined using linear modal analysis as follows:

Table 4.18. Modal participating mass ratios of the model with basement floor.

Mod	Period (sec)	U _X	U _Y	R _Z	ΣU _X	ΣU _Y	ΣR _Z
1	0.97	68%	0%	0%	68%	0%	0%
2	0.77	0%	57%	9%	68%	57%	9%
3	0.66	0%	9%	54%	68%	67%	63%
4	0.33	7%	0%	0%	75%	67%	63%
5	0.24	0%	8%	1%	75%	74%	65%
6	0.21	2%	0%	0%	77%	74%	65%
7	0.20	0%	2%	8%	77%	76%	73%
8	0.16	0%	0%	0%	78%	76%	73%
9	0.13	0%	3%	1%	78%	79%	73%
10	0.10	0%	0%	0%	78%	79%	73%
11	0.10	0%	1%	3%	78%	80%	76%
12	0.10	0%	0%	0%	78%	80%	76%

The basement floor of the building is surrounded by reinforced concrete shearwalls as can be seen in Figure 4.15. Since these shearwalls create a very rigid story compared to the superstructure, the modes of the basement floor are not participated in the superstructure's modes. Therefore, the modal participation factors are affected in negative way. If the basement floor is removed from the model, the modal participation ratios become as follows:

Table 4.19. Modal participating mass ratios of the model without basement floor.

Mod	Period (sec)	U _X	U _Y	R _Z	ΣU _X	ΣU _Y	ΣR _Z
1	0.96	88%	0%	0%	88%	0%	0%
2	0.73	0%	68%	16%	88%	68%	16%
3	0.63	0%	15%	66%	88%	82%	82%
4	0.33	9%	0%	0%	97%	82%	82%
5	0.23	0%	9%	2%	97%	91%	84%
6	0.20	3%	0%	0%	99%	91%	84%

Mod	Period (sec)	U _x	U _y	R _z	ΣU _x	ΣU _y	ΣR _z
7	0.19	0%	3%	10%	99%	94%	94%
8	0.16	1%	0%	0%	100%	94%	94%
9	0.13	0%	3%	1%	100%	97%	95%
10	0.10	0%	0%	1%	100%	97%	95%
11	0.10	0%	1%	3%	100%	98%	98%
12	0.09	0%	0%	0%	100%	98%	98%

4.3.5. Definition of Non-linear Plastic Hinges

In the modeling of frame elements (columns and beams) based on plastic behavior, three-dimensional element yield surfaces and damage limits have been defined according to TBDY-2018 [2].

In the Deformation-Based Assessment (ŞGDT) approach of TBDY-2018 [2], the section properties of plastic hinges are defined using Concentrated Inelasticity Fiber Elements to specify the plastic hinge definitions in the relevant sections. In the “Guidelines for Nonlinear Structural Analysis for Design of Buildings” [38], it is stated that in this modeling technique, each element end utilizes a single fiber cross-section, and the user specifies a plastic hinge length to correlate curvature with rotation. The central part of the element is modeled as elastic. As illustrated in Figure 4.16, the fiber hinge method employs inelastic sections solely at the ends of the element, with a predefined plastic hinge length.

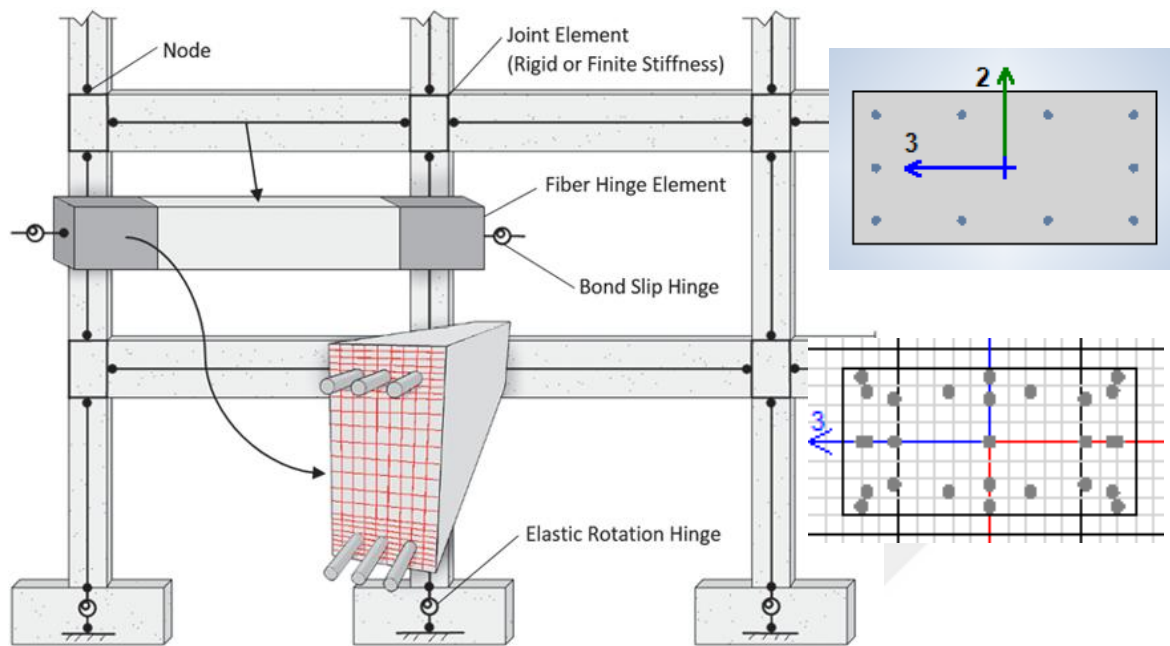


Figure 4.16. Overview of a typical reinforced-concrete moment-resisting frame system, showing the fiber-type model idealization [38].

Figure 4.16 above shows the cross-sectional cell (fiber) model of a reinforced concrete element. Here, each fiber is defined according to the concrete and reinforcement material models described previously. Separate fiber definitions are made for concrete cover, concrete core, and reinforcements.

The fiber element model is an analysis method that provides a more detailed calculation possibility by dividing the element cross-sections into multiple fiber elements, based on the distributed plasticity model. It can accurately account for the spread of nonlinear behavior along the element, hence it delivers very good results in damage estimation, even under effects that cause very high plastic behavior.

Under the effects of bending moments or axial forces at the element sections, the forces that arise are obtained through the integration of stress values in the fiber elements across the section.

The movement of the neutral axis in the fiber sections due the bending behavior is taken into account in the calculations.

4.3.6. Definition of Damage States in the Concrete Sections

All reinforced concrete columns have been modeled using two different types of concrete models: core and shell concrete. Unit shortening limits of the core concrete have been determined in accordance with Section 15.7.1.3 of TBDY-2018 [2], incorporating 30% of the 90-degree closed stirrups.

Additionally, the deformation upper limits related to the cross-sections have been reduced in accordance with the shear stress level in the section as per TBDY-2018 Section 15.7.1.4. Limit states for plastic rotation and unit deformations are calculated according to TBDY-2018 Sections 5.8.1.2 and 5.8.1.3.

Below are the section properties and damage limit state values determined for the reinforced concrete columns.

Table 4.20. Section properties of C30X60 columns used in the structural model.

Section Properties		
Section width (X-direction)	b	300 mm
Section height (Y-direction)	h	600 mm
Cover thickness	d	30 mm
Longitudinal reinforcement diameter	d_b	$\phi 16$
Number of longitudinal reinforcements	n	10
Longitudinal reinforcement area	A_s	2010 mm ²
Transverse reinforcement diameter	d_e	$\phi 8$
Number of stirrup legs in X-direction	n_x	2
Number of stirrup legs in Y-direction	n_y	2
Longitudinal Reinforcement Ratio	ρ_t	1.12%
Spacing of transverse reinforcement	s	200 mm
Average (expected) yield strength of transverse rebar	f_{ywe}	S220
Average (expected) yield strength of longitudinal rebar	f_{ye}	S220
Knowledge Level Factor	KF	1
Unconfined concrete compressive strength	f_{c0}	14.70 MPa
Hook Angle in stirrups	Deg.	90°
Modulus of Elasticity of Concrete	E_c	19,170 MPa

Section Properties		
Shear gap (ratio of moment/shear force in section)	L_s	1450 mm

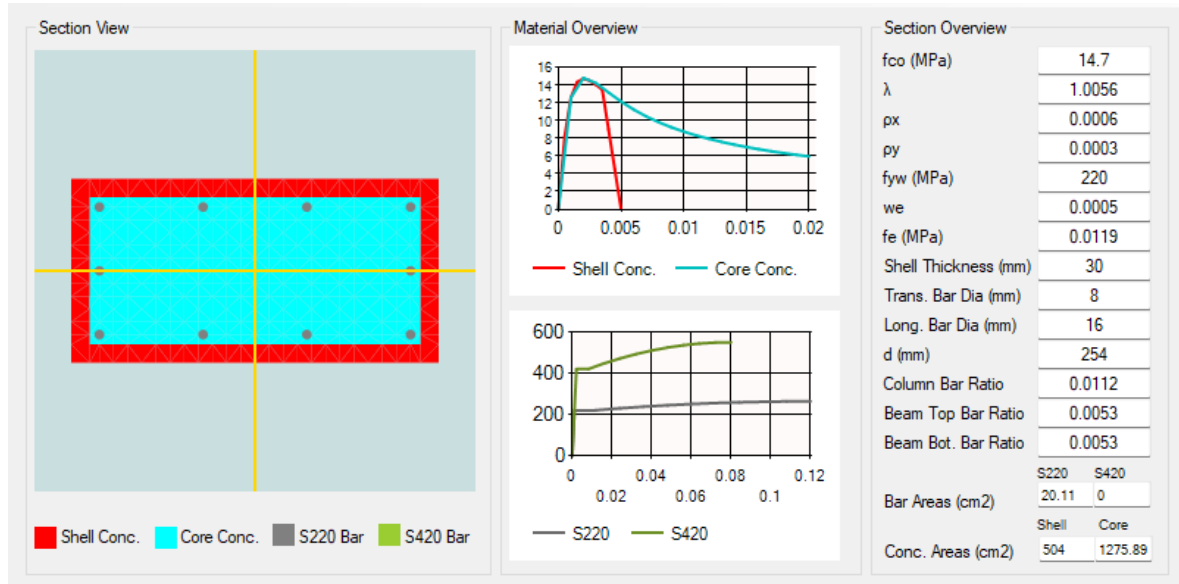


Figure 4.17. Material properties of C30X60 columns used in the structural model.

E Nonlinear Material Data

Material Name and Type

Material Name: C14.73_30X60

Material Type: Concrete, Isotropic

Acceptance Criteria Strains

	Tension	Compression	
IO	0.01	-0.0025	mm/mm
LS	0.02	-0.0037	mm/mm
CP	0.05	-0.0049	mm/mm

☒ Ignore Tension Acceptance Criteria

Miscellaneous Parameters

Hysteresis Type: Concrete

Modify/Show Hysteresis Parameters...

Drucker-Prager Parameters

Friction Angle: 0 deg

Dilatational Angle: 0 deg

Stress Strain Curve Definition Options

☐ Parametric

☒ User Defined

User Stress-Strain Curve Data

Number of Points in Stress-Strain Curve: 15

Point Number	Strain	Stress (N/mm ²)	Point ID
1	-0.0567	-0.01	
2	-0.0125	-0.01	
3	-0.0124	-8.9	
4	-0.0107	-9.53	
5	-0.008997	-10.29	
6	-0.007326	-11.23	
7	-0.005654	-12.39	
8	0.003882	12.77	

Figure 4.18. Material properties of C30X60 columns defined in the structural model.

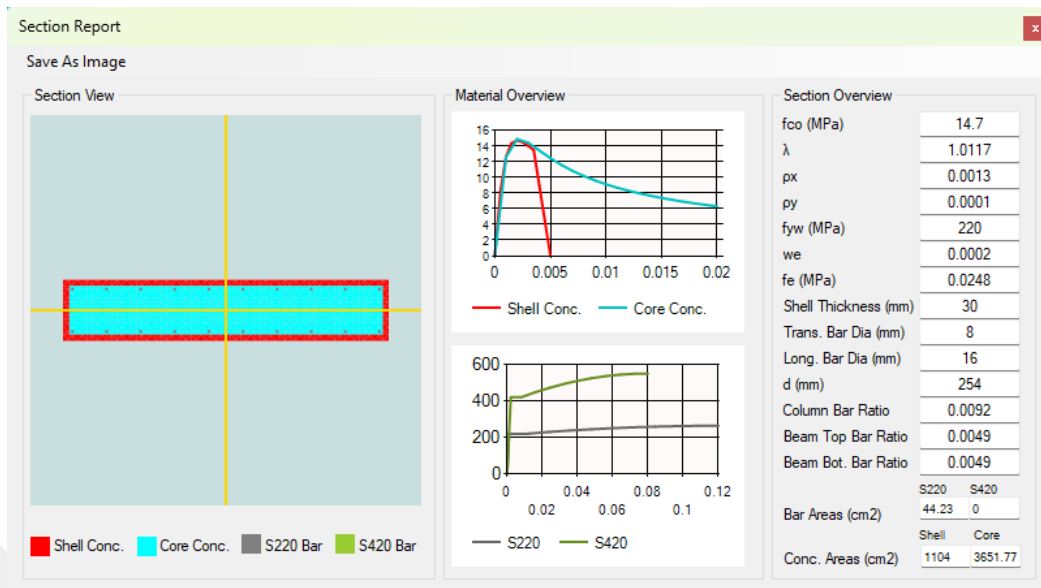


Figure 4.19. Material properties of C30X160 columns used in the structural model.

E Nonlinear Material Data

Material Name and Type

Material Name: C14.7_30X160

Material Type: Concrete, Isotropic

Acceptance Criteria Strains

	Tension	Compression	
IO	0.01	-0.0025	mm/mm
LS	0.02	-0.00334	mm/mm
CP	0.05	-0.00445	mm/mm

☒ Ignore Tension Acceptance Criteria

Miscellaneous Parameters

Hysteresis Type: Concrete

Modify/Show Hysteresis Parameters...

Drucker-Prager Parameters

Friction Angle: 0 deg

Dilatational Angle: 0 deg

Stress Strain Curve Definition Options

☐ Parametric

☒ User Defined

User Stress-Strain Curve Data

Number of Points in Stress-Strain Curve: 15

Point Number	Strain	Stress (N/mm ²)	Point ID
1	-0.05	-0.01	
2	-0.0155	-0.01	A
3	-0.0154	-8.24	
4	-0.0132	-8.87	
5	-0.011	-9.65	
6	-0.00883	-10.64	
7	-0.00665	-11.95	
8	0.00447	12.67	

Figure 4.20. Material properties of C30X160 columns defined in the structural model.

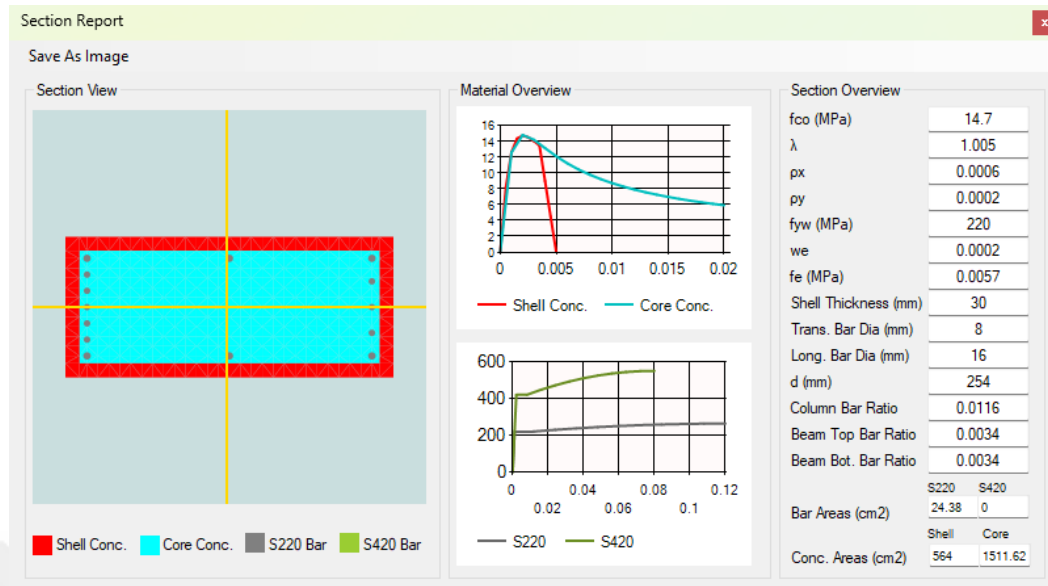


Figure 4.21. Material properties of B30X70 beams used in the structural model.

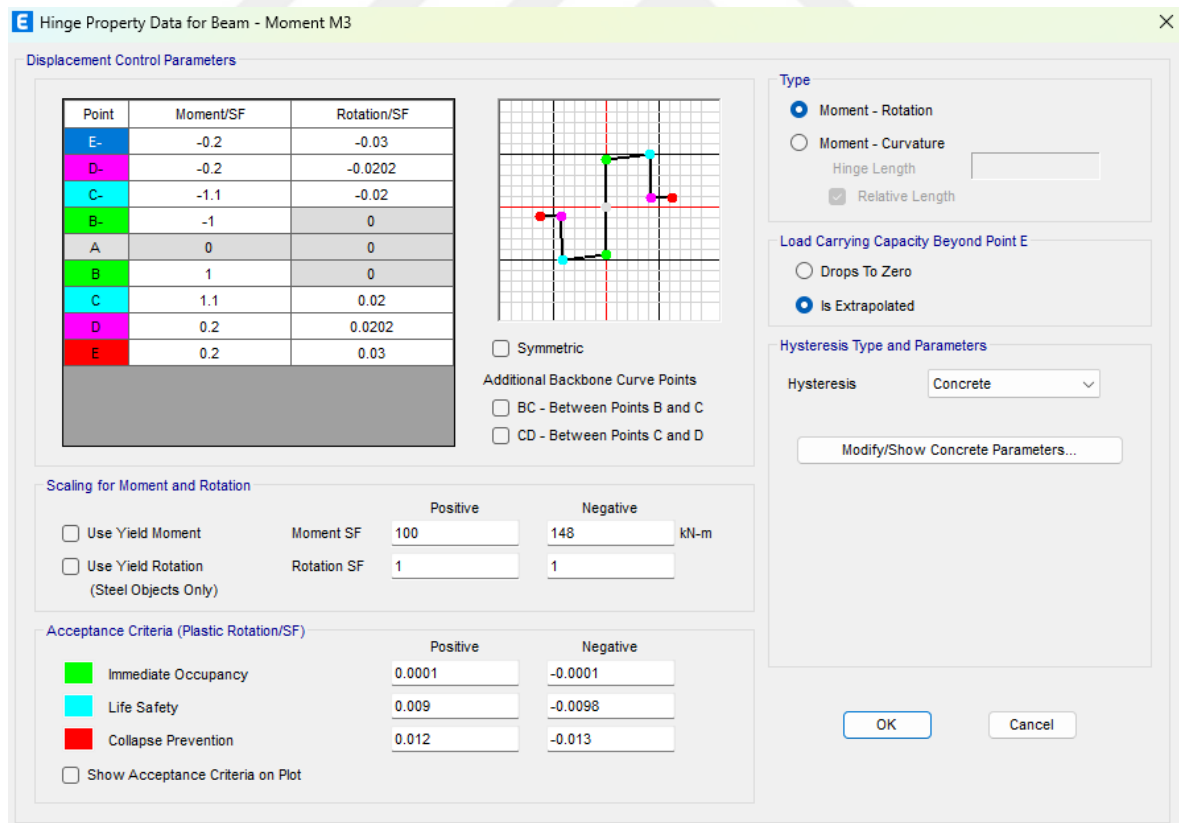


Figure 4.22. Material properties of B30X70 beams defined in the structural model.

4.3.7. Analysis Results of the Original Case of the Building

In order to determine the performance and the deficiencies of the structure's original case, non-linear pushover analysis is performed for two orthogonal directions. As a result of the pushover analyses, the performance point of the building system under the considered earthquake effect has been identified, and the extent of damage to structural elements at this performance point has been determined according to the criteria of TBDY-2018 Chapter 15 [2].

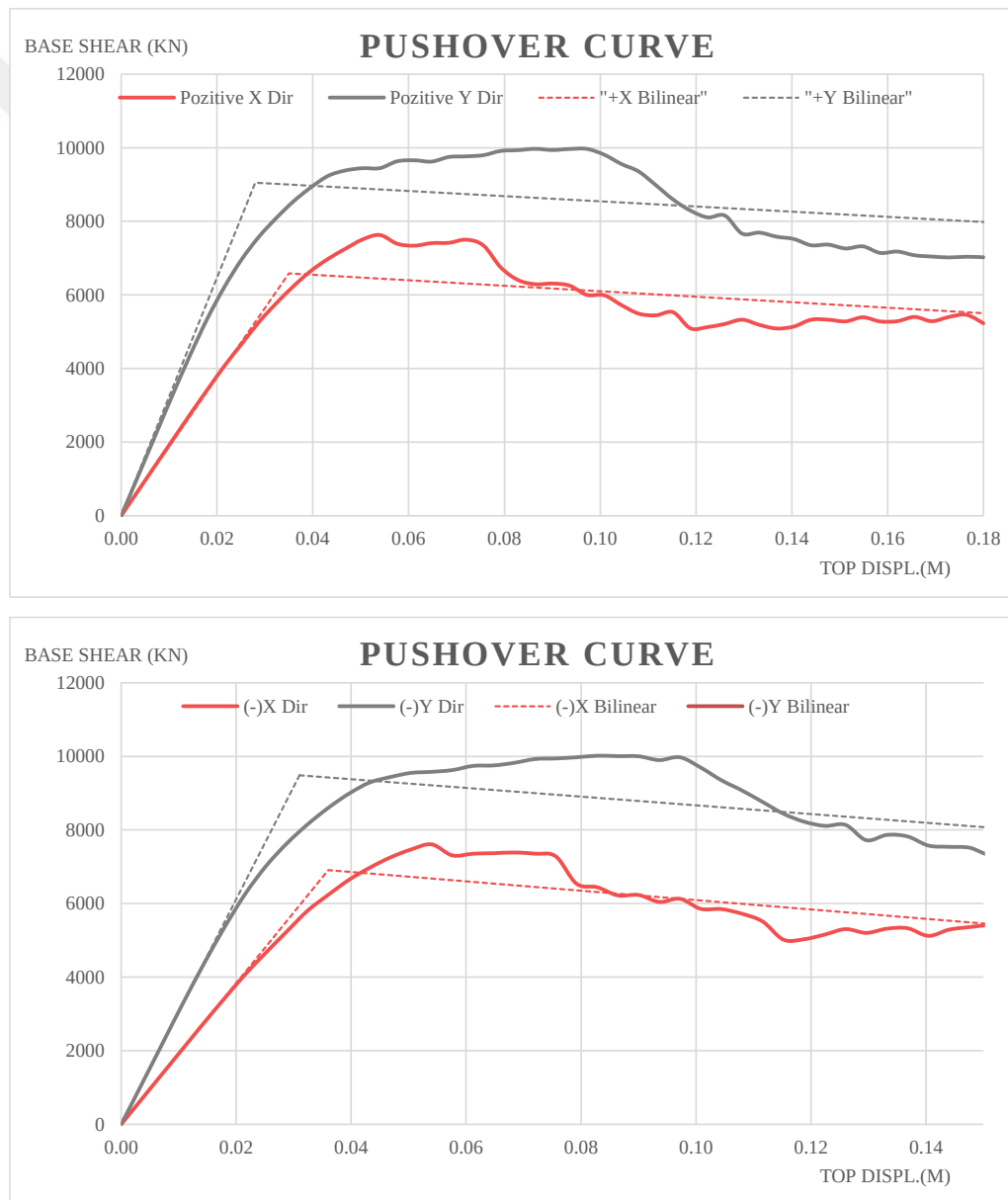


Figure 4.23. Pushover curves of the original case of the school building.

As a result of the pushover analysis, the peak displacement demands at the performance point have been calculated as 26.3 cm for the (+)X direction, 21.4 cm for the (+)Y direction, 26.3 cm for the (-)X direction, and 21.2 cm for the (-)Y direction. The assessment has been performed at these steps of the pushover analysis based on the demands occurring in the system.

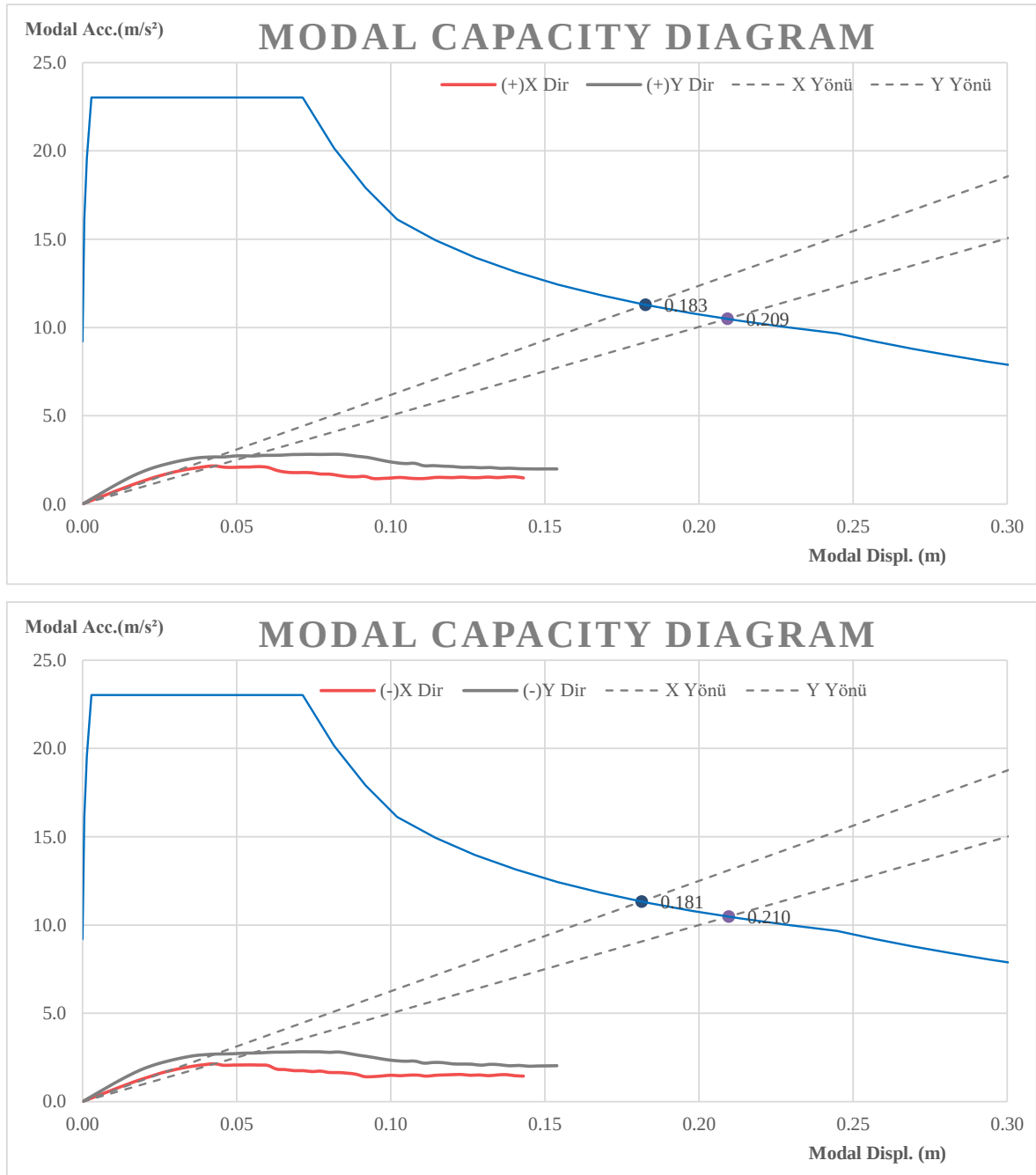


Figure 4.24. Modal capacity diagrams of the original case of the school building.

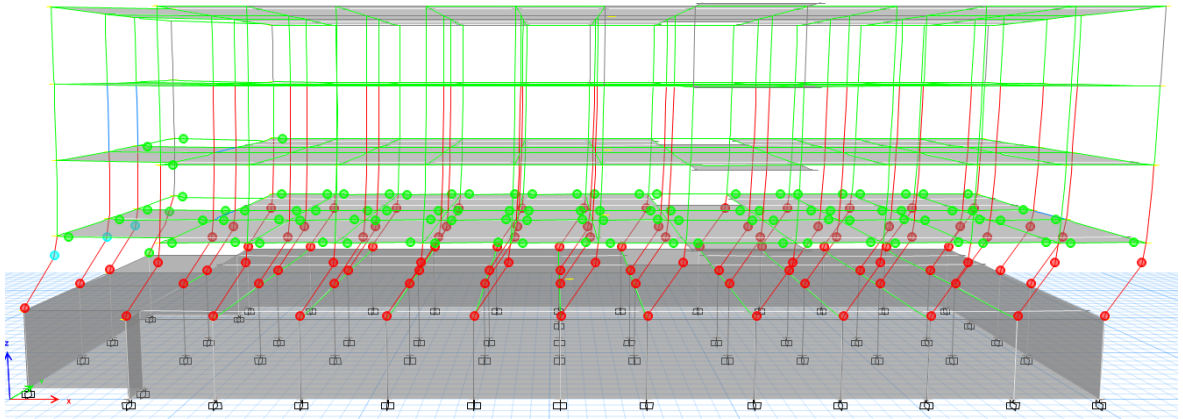


Figure 4.25. Damage states in columns and beams at the performance point in the positive X direction.

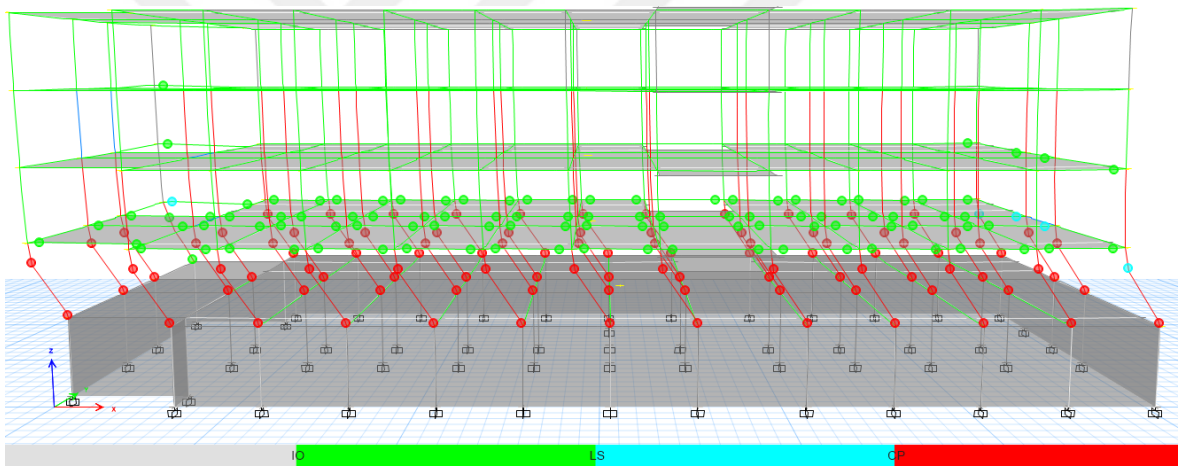


Figure 4.26. Damage states in columns and beams at the performance point in the negative X direction.

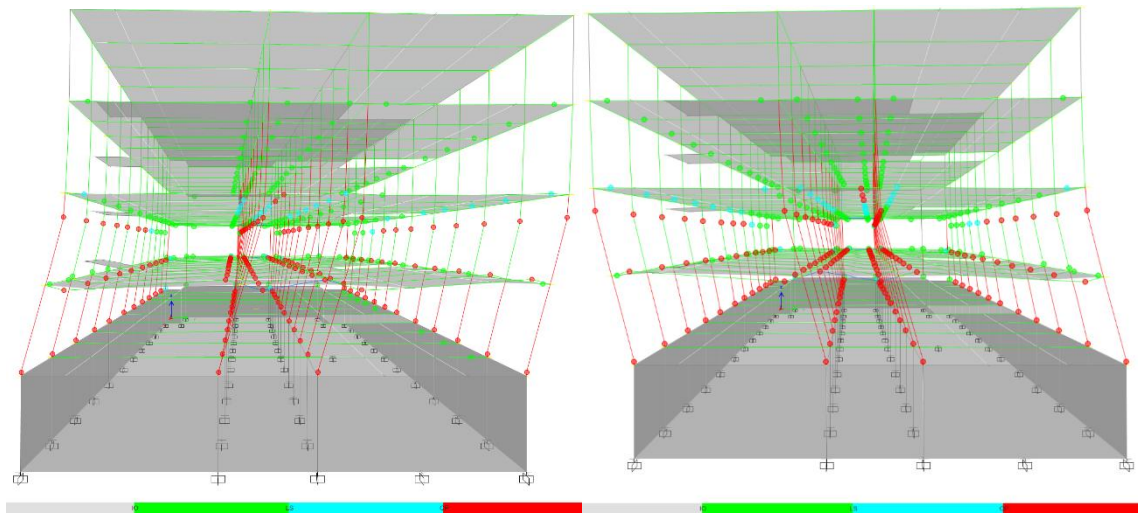


Figure 4.27. Damage states in columns and beams at the performance point in the Y direction (figure on the left is for positive Y direction and figure on the right is for negative Y direction).

Pushover analysis results confirm that, with the given material/structural properties and seismic parameters, the structure is in the collapse state due to high demand on the structure and lack of shear and flexural capacity of the columns. All columns on the ground floor are at collapse state.

Non-linear time history analyses are also performed using the structural model of the original case of the building. Using the selected 11 ground motion records, 22 time-history analysis are performed.

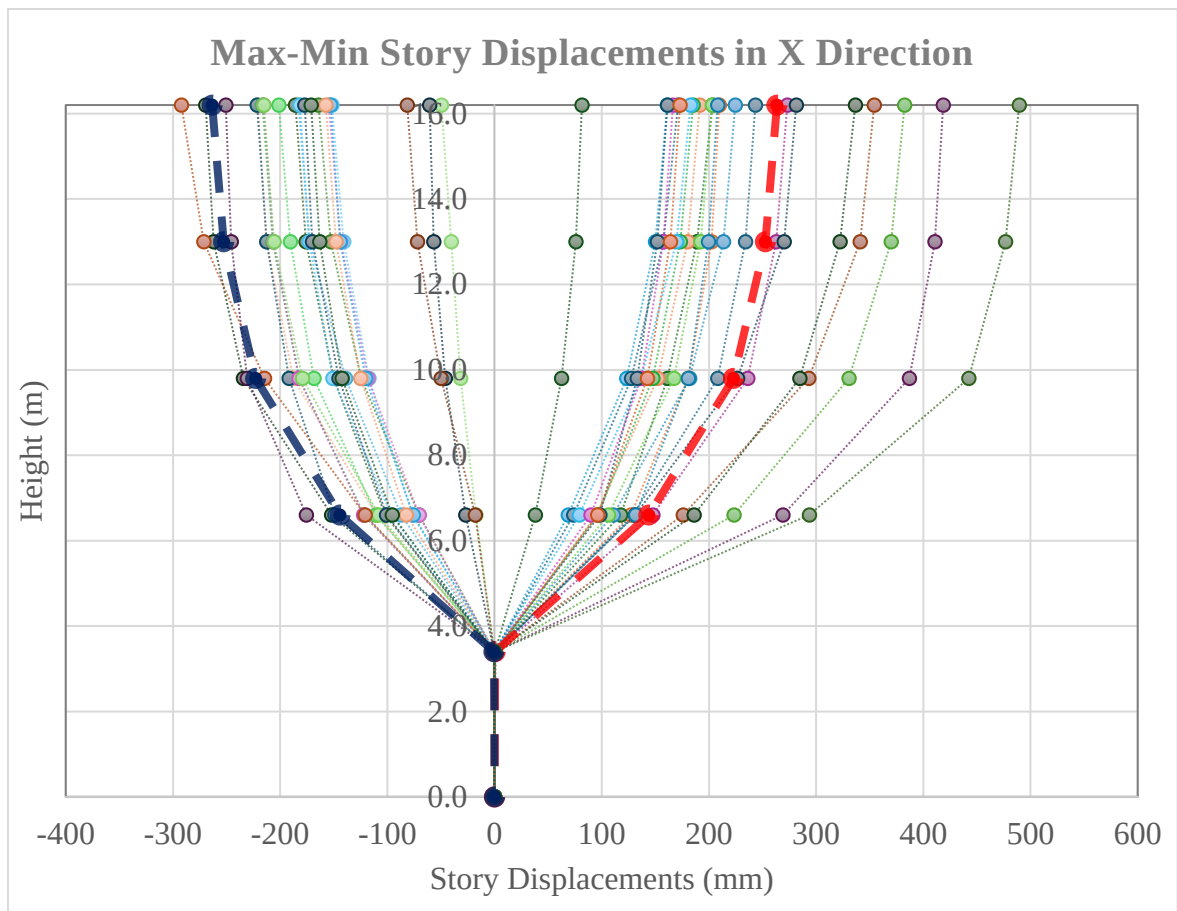


Figure 4.28. Maximum and minimum story displacements in X direction obtained from 22 ground motion analysis for the original case model.

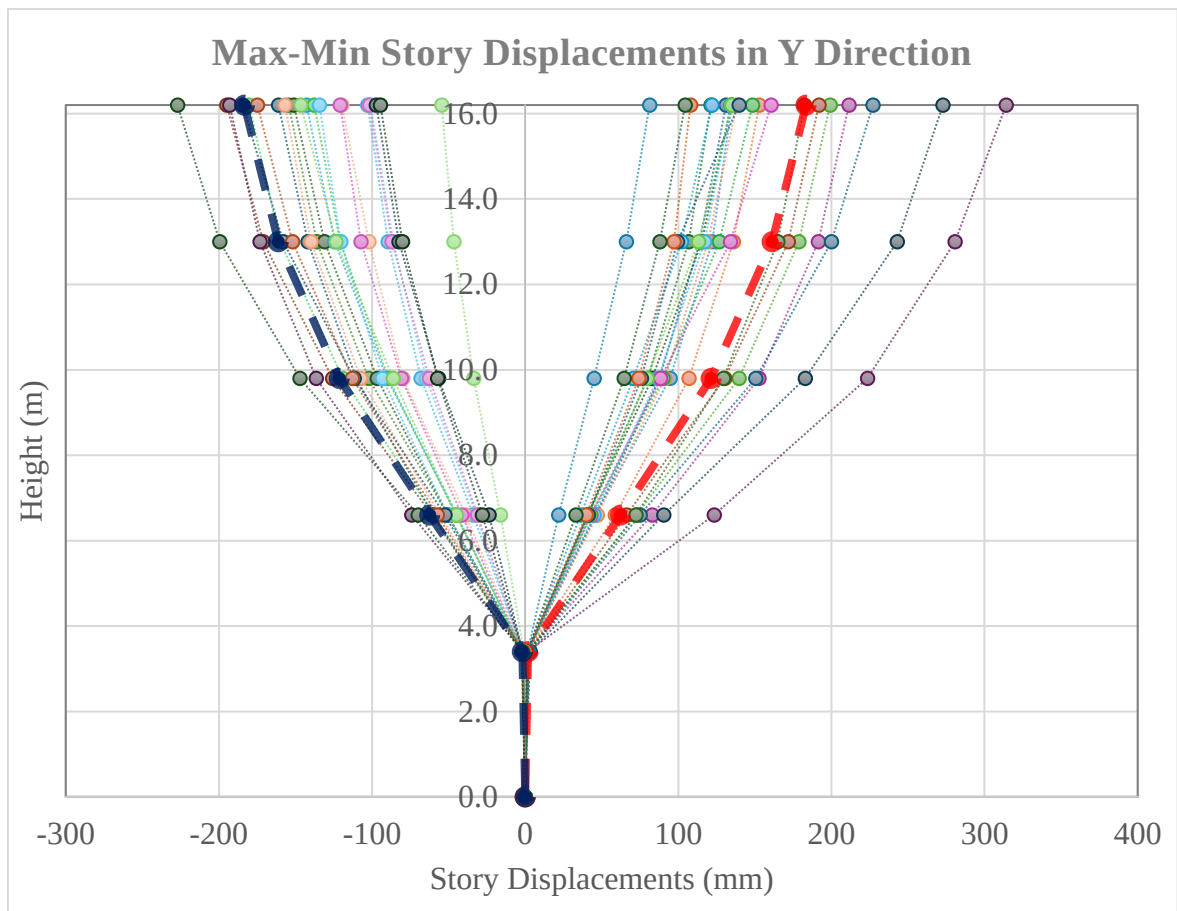


Figure 4.29. Maximum and minimum story displacements in Y direction obtained from 22 ground motion analysis for the original case model.

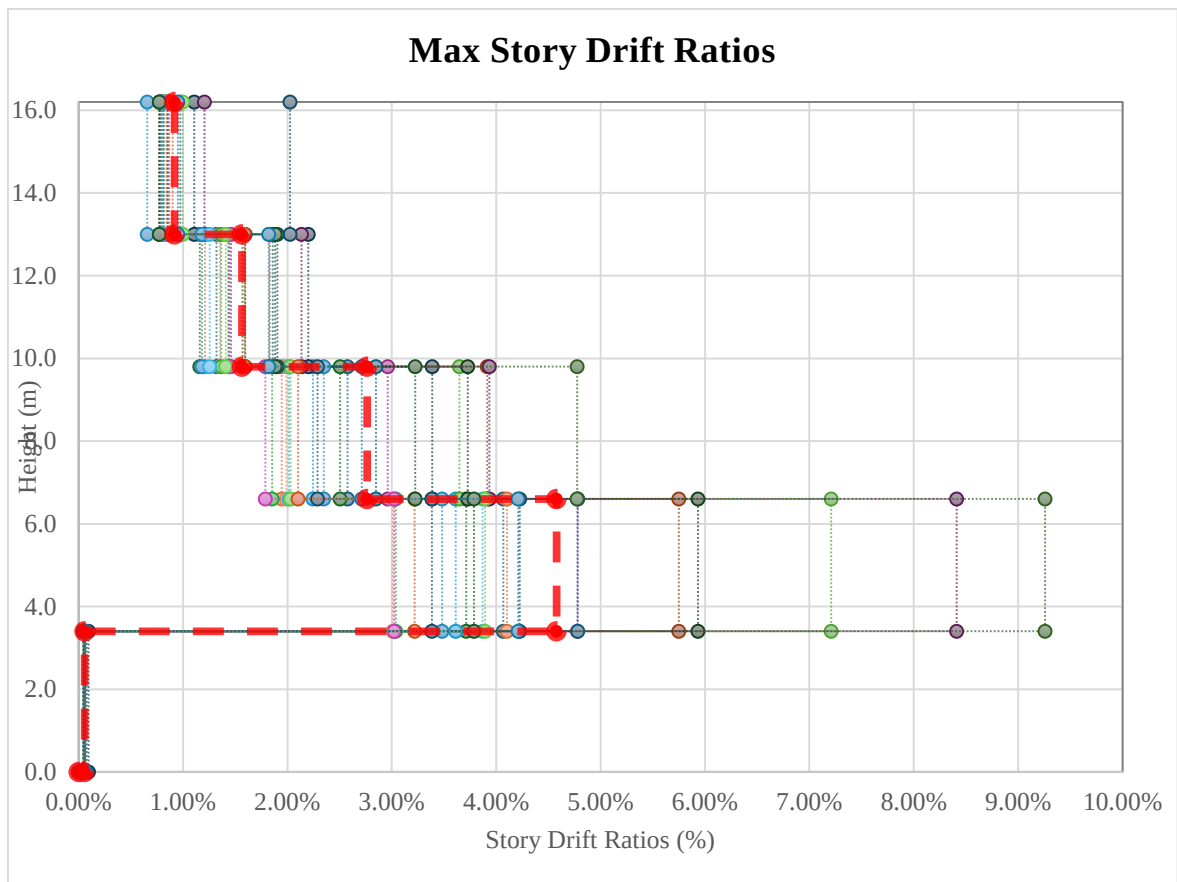


Figure 4.30. Maximum story drift ratios obtained from 22 ground motion analysis for the original case model.

The average of the maximum and minimum top displacements obtained from the 22 time-history analysis results are 26.3 cm for the (+)X direction, 18.4 cm for the (+)Y direction, 26.4 cm for the (-)X direction, and 18.4 cm for the (-)Y direction. These results are exactly same with the pushover analysis results on X direction. On the other hand, on the Y direction, NLTH results are ~15% smaller than the pushover analysis results.

The performance level of the (original case of the) building is the same (Collapse State) for both analysis methods since all the columns on ground floor are in collapse state for both analysis methods. However, damage states for the individual structural elements at upper floors may vary depending on the analysis method. It is considered that, the difference arises from the nature of the applied methods. Since pushover analysis is a displacement-based static analysis method, once plastic hinges form in all ground floor columns and the columns reaches a collapse state, the entire displacement demand becomes concentrated at the ground

level. In contrast, nonlinear time-history (NLTH) analysis is a dynamic analysis method that operates based on applying accelerations to the system. Even if column failures occur at the ground level, upper stories continue to experience accelerations due to their mass, generating forces and motion. Consequently, damage propagation in upper-story elements can still occur.

According to the above presented analysis results, it is concluded and confirmed that the original state of the structure needed to be retrofitted.

4.3.8. An Alternative Retrofit Solution with Viscous Dampers

Instead of the conventional retrofits applied on site, an alternative retrofit solution, adding external fluid viscous dampers to the building, is evaluated in this study. Viscous dampers have been considered a highly suitable retrofitting method for this building due to the following reasons:

- Absence of shear walls – Since the building does not have shear walls, the interstory drifts and velocities are at levels that allow the dampers to function effectively.
- High velocity content in selected pulse-like records – Given that viscous dampers are velocity-dependent devices, the high velocity content in the selected pulse-dominated ground motion records does not pose any adverse effects on their performance.
- Minimal stiffness contribution and reduced foundation demand – Due to the inherently low stiffness of viscous dampers, they generate lower foundation forces compared to Buckling-Restrained Braces (BRBs), thereby reducing or even eliminating the need for additional foundation strengthening.

Using the pushover analysis results, the top displacement demand of the building is already determined. In addition to this, the maximum top displacement at which the building provides Life Safety Performance Level is also determined (~7.5cm in X direction and ~8.5cm in Y direction). Therefore, it has been concluded that if the top displacements can be reduced to approximately 7.5 cm~8.5cm, the building can achieve the targeted performance level. Accordingly, the primary focus in determining the layout and characteristics of the dampers has been to reduce top displacements to this level. However, the target supplemental

viscous damping ratio to be provided for retrofitting (without changing the capacity of the columns and beams) is calculated as more than 60%. This ratio is above the rational and feasible limits. Hence, it is concluded that additional precautions should be taken in order to increase the ductility of the system. Additional precautions are necessary for the structure due to:

- the lack of confinement reinforcement in the building,
- the 90° angle of the stirrup hooks,
- the insufficient concrete quality,
- the very high earthquake demand.

In order to provide a cost-effective solution, while external viscous dampers are added from outside, wrapping of the columns with either Carbon Fiber Reinforced Polymer (CFRP) or steel plates are also considered.

Dampers are located symmetrically around the building. While determining the locations of FVDs, architectural constraints are also considered such as the entrance to the building and the corridor to be used during service of the building are avoided.

After the first few trial-error analyses, it was observed that the demand for the viscous dampers decreases at the upper floors. In order to provide a more economical solution, the number of dampers are reduced on X direction at the upper floors. On the other hand, this reduction was not applied on Y direction due to the limited available space for application of dampers on this direction.

Finalized locations are shown in Figure 4.31, Figure 4.32, Figure 4.33 below.

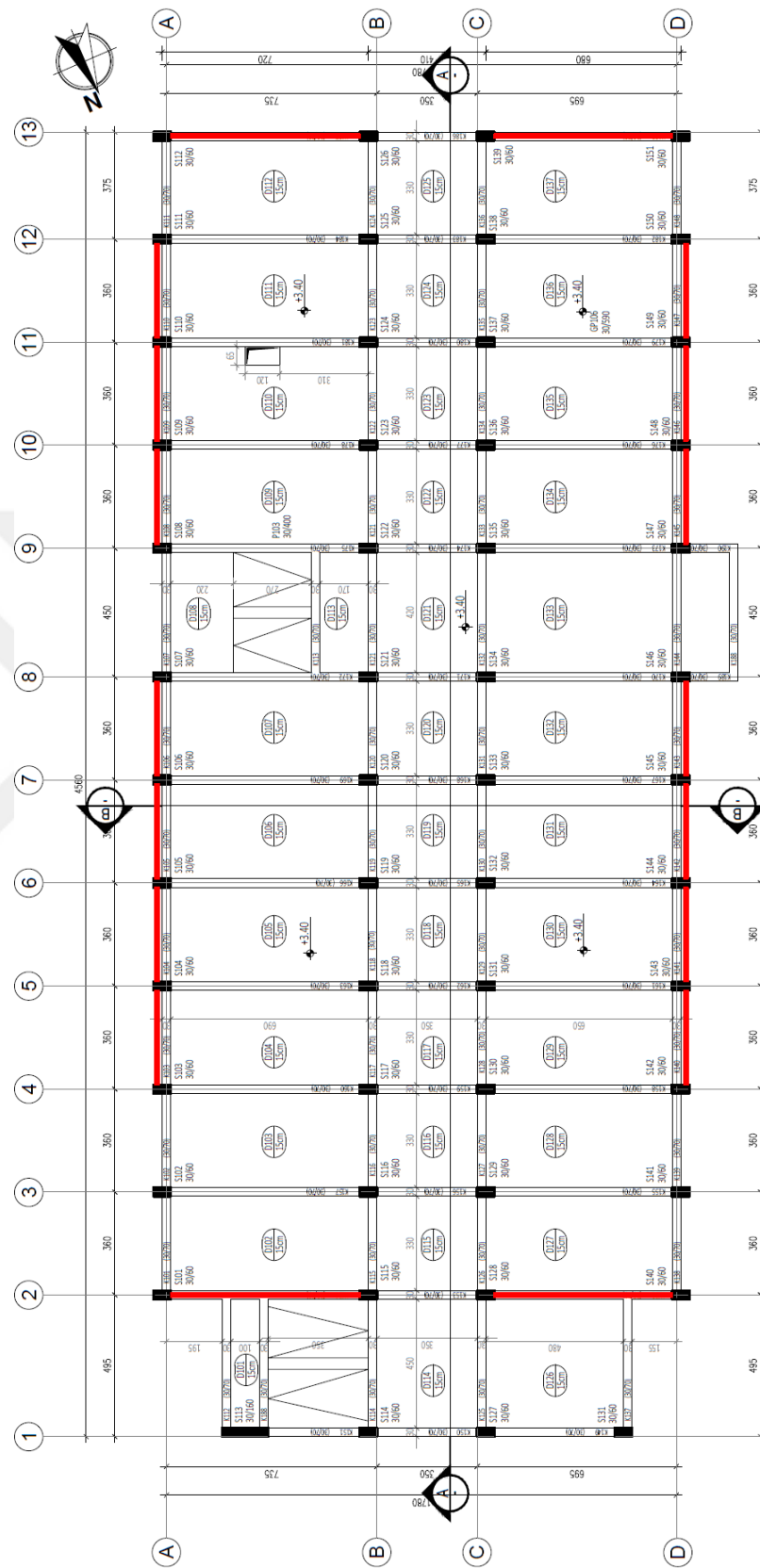


Figure 4.31. Plan view of the ground floor of the structure. The finalized viscous damper locations used in the design are shown in red color.

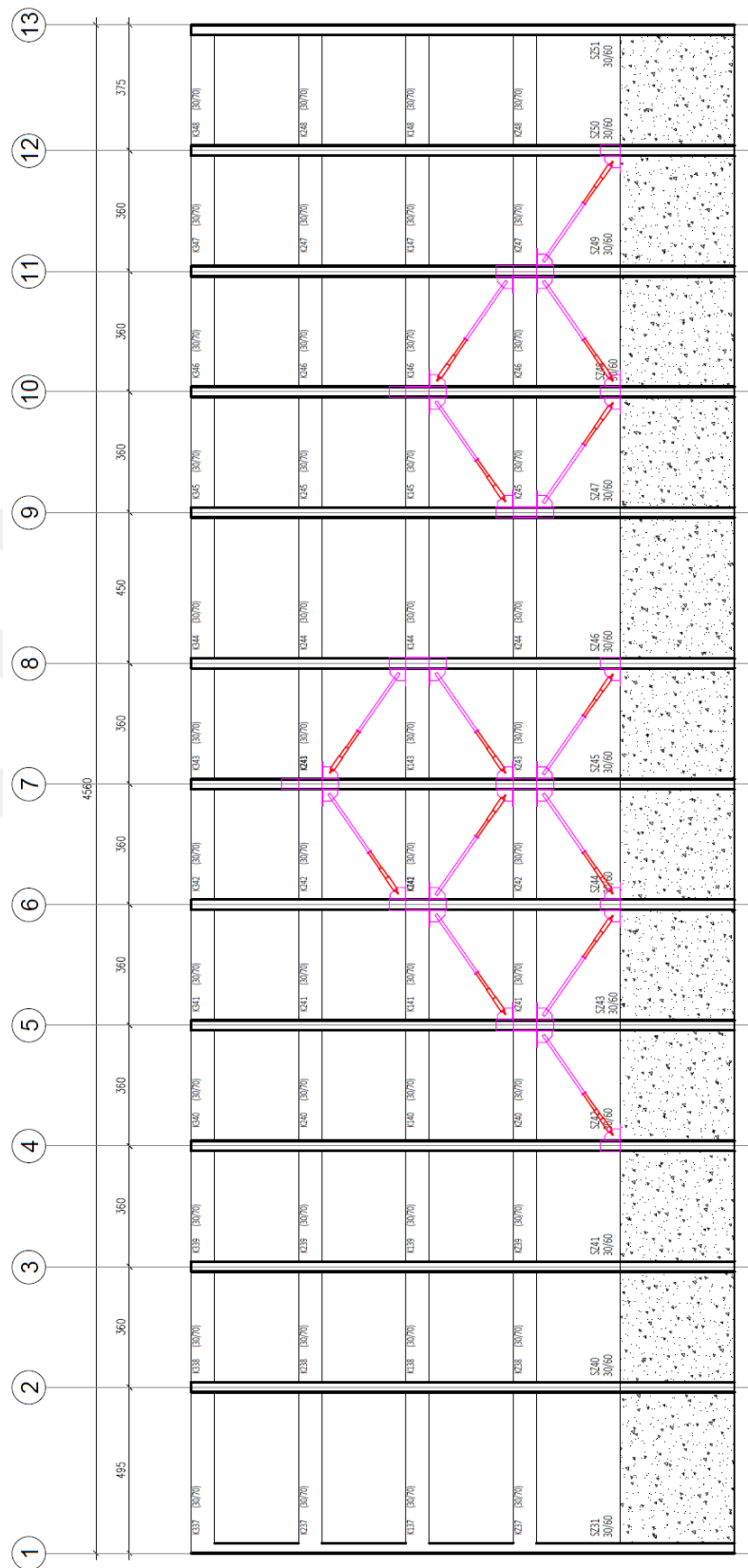


Figure 4.32. Side view of the structure on gridline D. The finalized viscous damper locations used in the design are shown in red and magenta color.

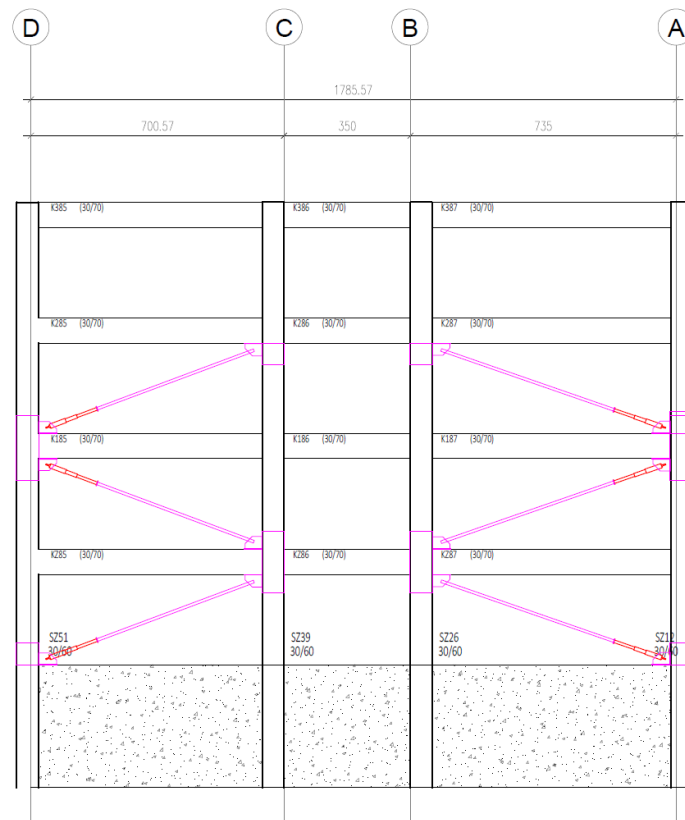


Figure 4.33. Side view of the structure on gridline 13. The finalized viscous damper locations used in the design are shown in red and magenta color.

The target effective damping is selected to be 40% in order to decrease the demand on the structural members around half of the original state according to Table 2.1. Preliminary sizes of viscous dampers (for $\alpha=1$) are determined using Equation (2.8) as follows:

Table 4.21. Preliminary Damper Properties.

Storey	X Direction				Y Direction			
	ζ_{viscousX}	k_x	n_x	C_j	ζ_{viscousY}	k_y	n_y	C_j
3 rd Floor	0%	397559	0	0	0%	470577	0	0
2 nd Floor	9%	356363	4	4432	0%	487885	4	0
1 st Floor	25%	327770	10	4529	25%	499824	4	6853
Ground Fl.	35%	369621	14	5107	35%	709725	4	13624

Viscous dampers are categorized into linear and nonlinear dampers. In linear dampers, the exponential constant α that determines the nonlinear behavior of the dampers is set to 1,

while dampers with $\alpha \neq 1$ are referred to as nonlinear dampers. Viscous dampers can be characterized by Equation (2.6).

In this study, the dampers used have been selected with an α coefficient of 0.1. The reason for choosing $\alpha=0.1$ is that this value is sufficiently small to limit the upper bound of the damper response force at very high velocities since the selected ground motions including pulse content have very high velocities components. Figure 4.34 presents the force-velocity relationship for a damper, considering various values of α .

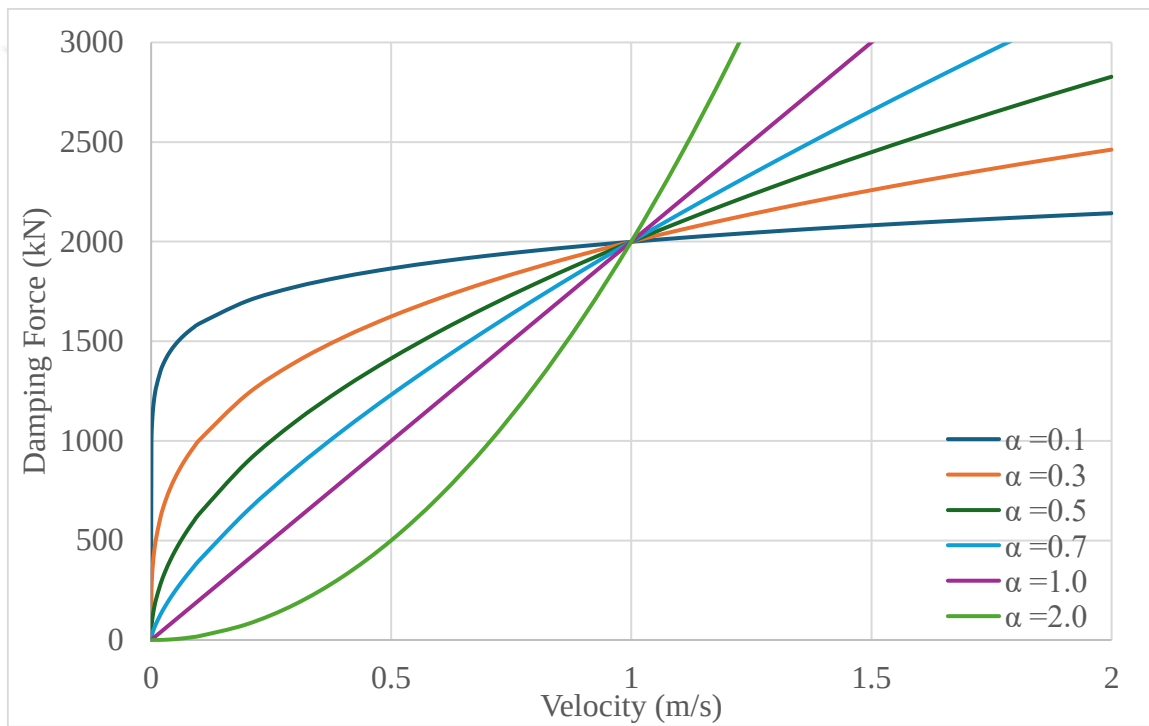


Figure 4.34. Curve of damping force vs. velocity with different damping exponents.

The amount of energy dissipated by the dampers decreases as the value of α decreases for the same damping coefficients. Therefore, higher damping coefficients are required to absorb energy at the level of linear dampers. However, for all nonlinear dampers (compared to linear dampers), the equivalent damping ratio varies depending on displacement amplitude and frequency. Therefore, after selecting the performance target in practice, analyses are conducted on different C values, and optimization is undertaken considering economic factors (such as lower C coefficients, lower piston strokes, lower damper response forces, etc.).

In the context of this study, for the examined structure, it was observed that the performance target for the building was achieved with the chosen α value of 0.1.

In the final design, considering all the columns of the structure to be wrapped with CFRP (with varying thickness), the locations of the viscous dampers are determined as shown in Figure 4.32. A total number of 40 dampers are added (28 dampers in longitudinal (X) direction, and 12 dampers in lateral (Y) direction) to the building as shown in Figure 4.35 below. Damper parameters are selected as $C_X = 1500 \text{ kN}/\sqrt{\text{m/s}}$ ($=54 \text{ kips}/\sqrt{\text{in/s}}$), $C_Y = 2000 \text{ kN}/\sqrt{\text{m/s}}$ ($=72 \text{ kips}/\sqrt{\text{in/s}}$) and $\alpha = 0.1$.

The structural model is analyzed under the selected 11 ground motion records (given in Section 4.2.6). In accordance with TBDY-2018 Section 5.7.2.1 [2], ground motion records in two orthogonal horizontal directions were applied simultaneously along the principal axes of the structural system (X) and (Y). Subsequently, the axes of the acceleration records were rotated by 90° , and the calculations were repeated. Therefore 22 time history analyses are performed representing the DD-1 level earthquake with near fault effects.

As mentioned at the beginning of this section, the analysis results have shown that viscous dampers alone are not sufficient to increase the performance of this building to the 'Controlled Damage' level under the expected earthquake effects. Therefore, it has been decided to additionally reinforce all columns on the ground floor and first floor of the building with steel wrapping. To ensure continuity of the steel wraps between floors, holes will be drilled in the areas of the slabs corresponding to the column corners, and steel rods will be inserted.

After the analyses are completed, strains of the fiber elements (of columns and beams) are exported and processed to determine the average strain for each end of each structural element under 22 analysis case.

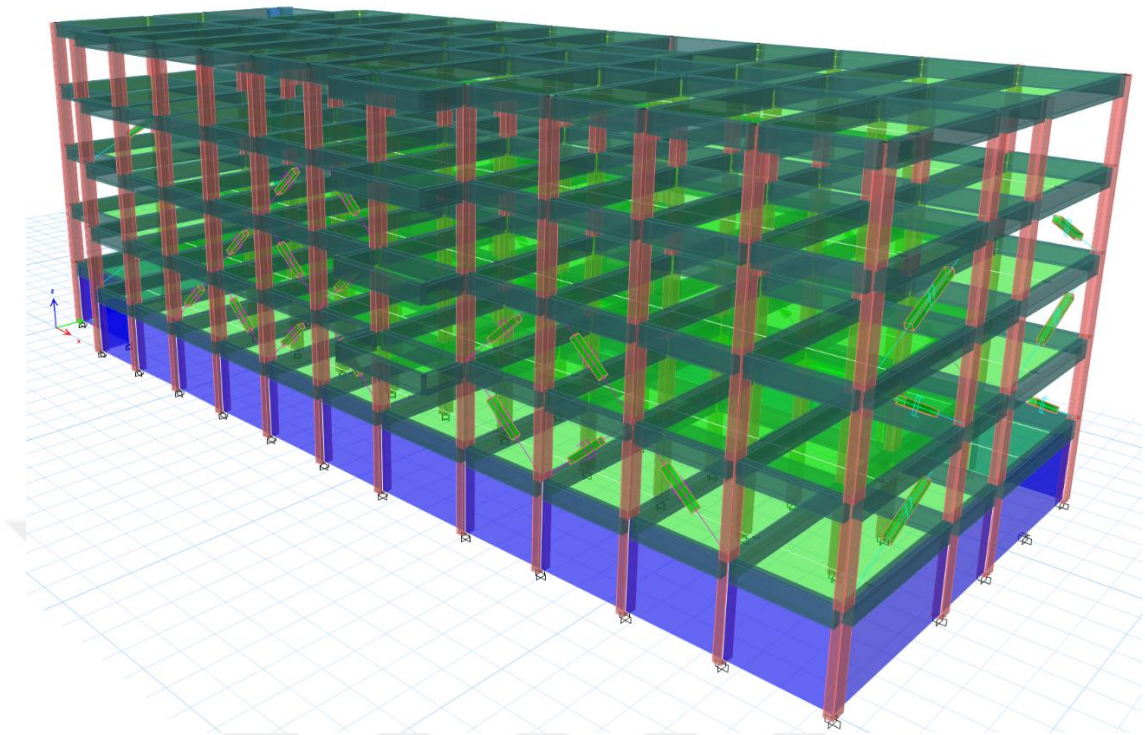


Figure 4.35. 3-D view of the structural ETABS model with FVDs.

Based on the strains obtained from the analyses, the damage states are determined according to TBDY-2018 Section 5.8.1 [2]. Summary of the damage states of columns are shown in below Figure 4.36.

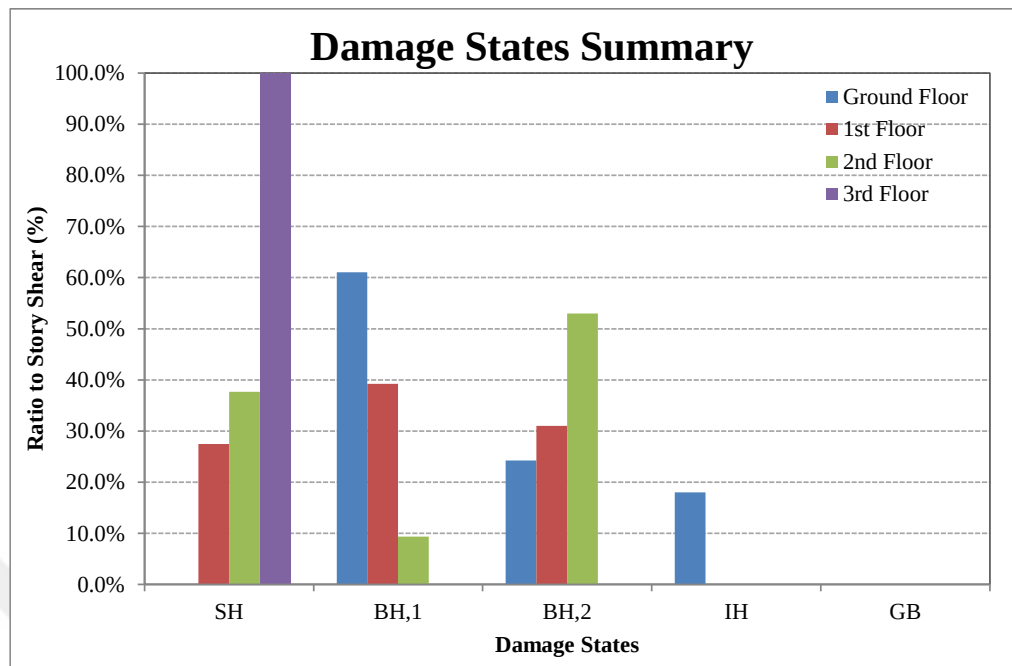


Figure 4.36. Summary of the damage states of columns based on the strains obtained from the analyses.

Damper force-displacement graphs are obtained from the model for the ground motions. Sample graphs for one damper at each story in both X and Y directions under RSN139 earthquake are given in below graphs.

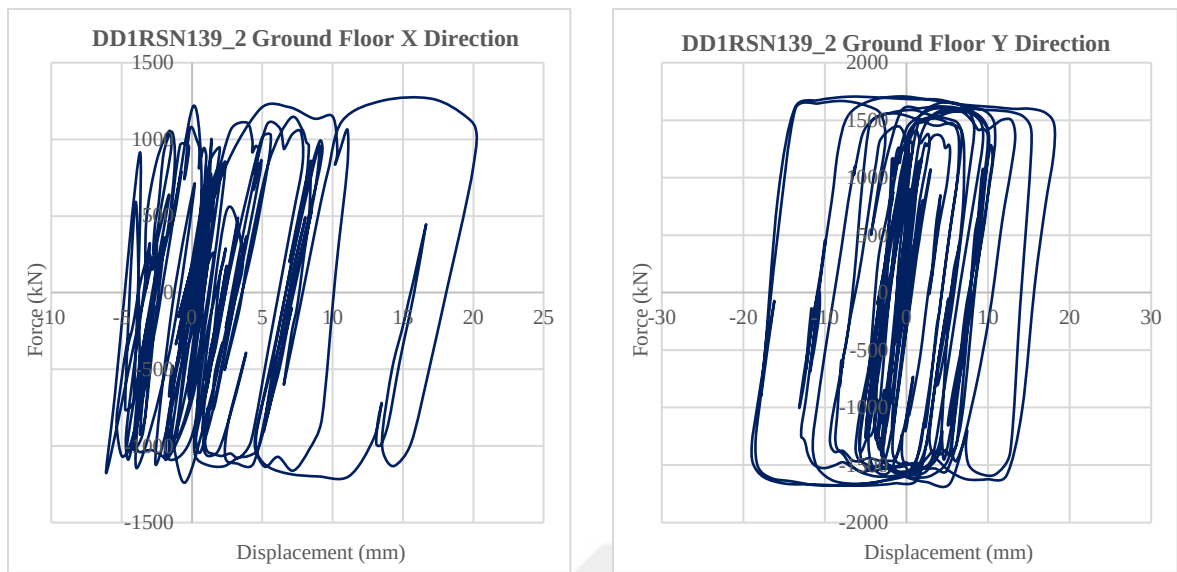


Figure 4.37. Force-displacement graphs of dampers at ground floor under RSN139_2 (left graph is for X direction, right graph is for the Y direction).

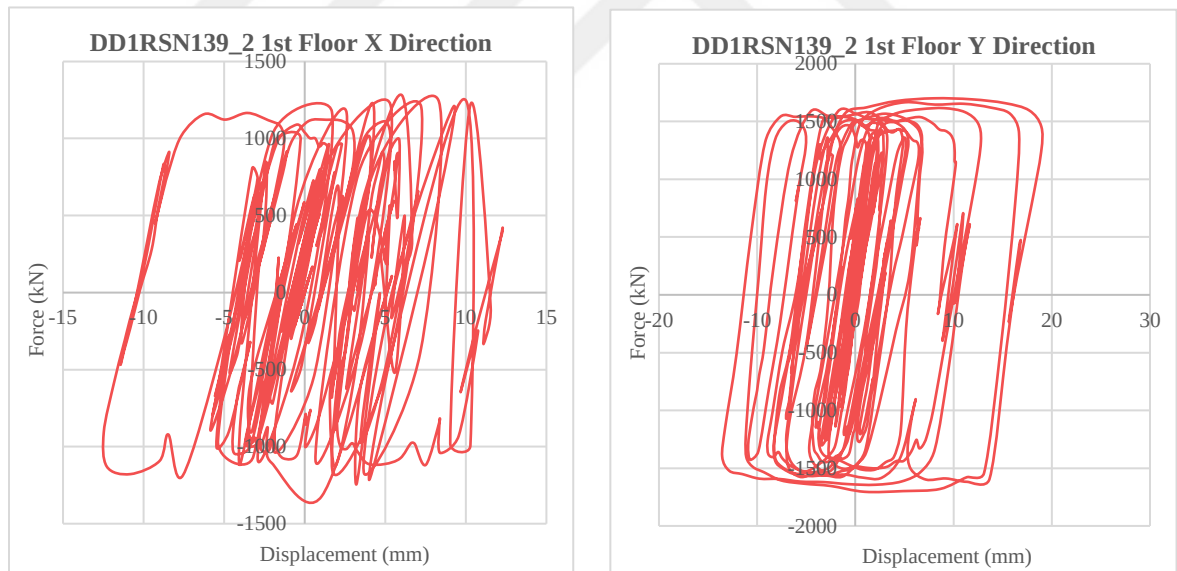


Figure 4.38. Force-displacement graphs of dampers at 1st floor under RSN139_2 (left graph is for X direction, right graph is for the Y direction).

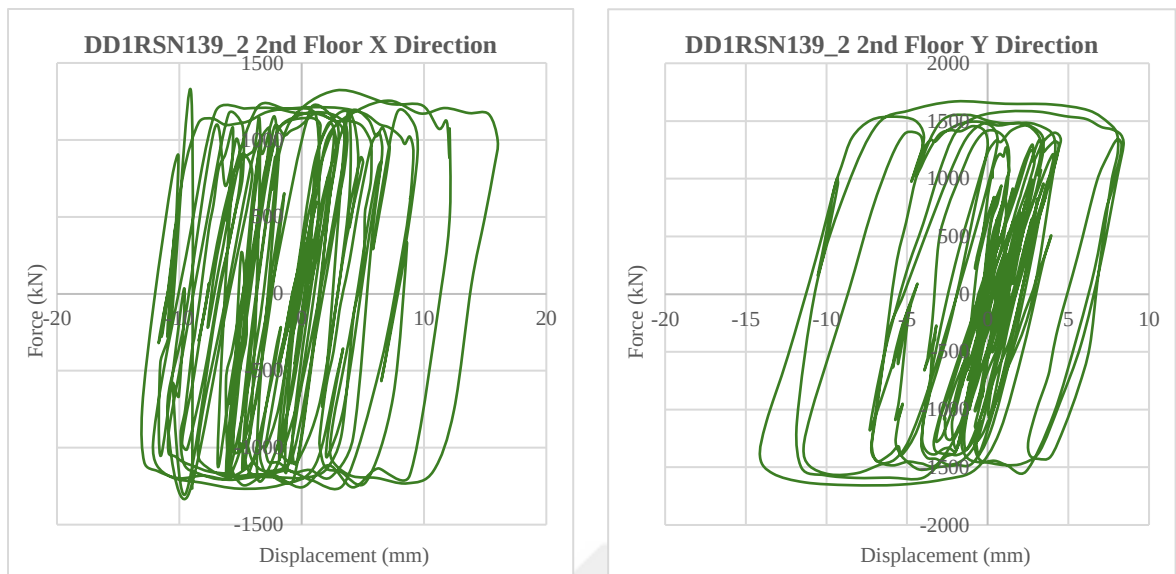


Figure 4.39. Force-displacement graphs of dampers at 2nd floor under RSN139_2 (left graph is for X direction, right graph is for the Y direction).

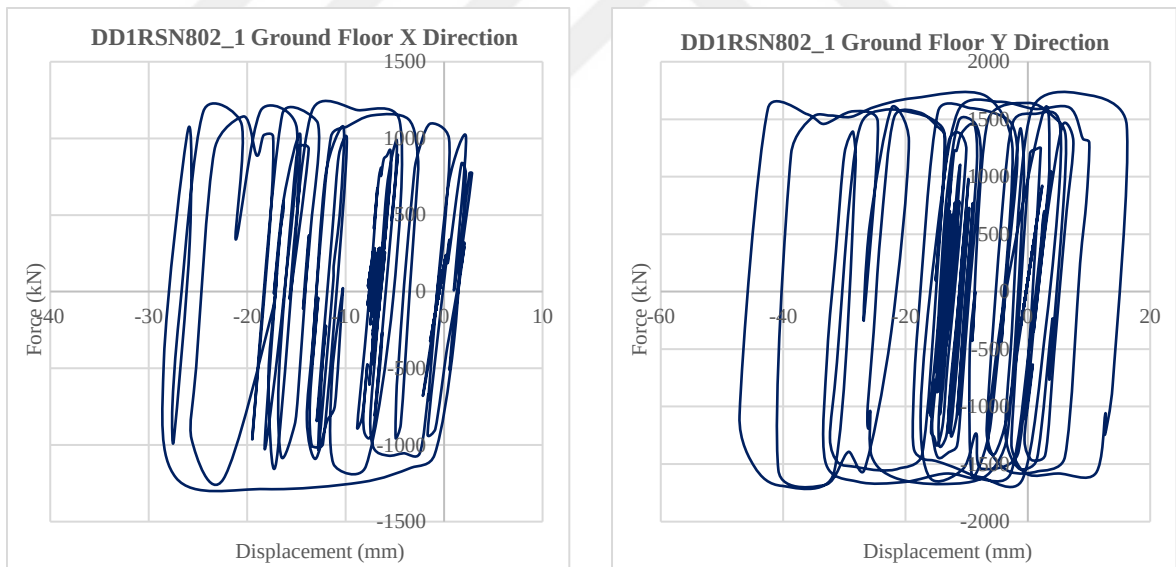


Figure 4.40. Force-displacement graphs of dampers at ground floor under RSN802_1 (left graph is for X direction, right graph is for the Y direction).

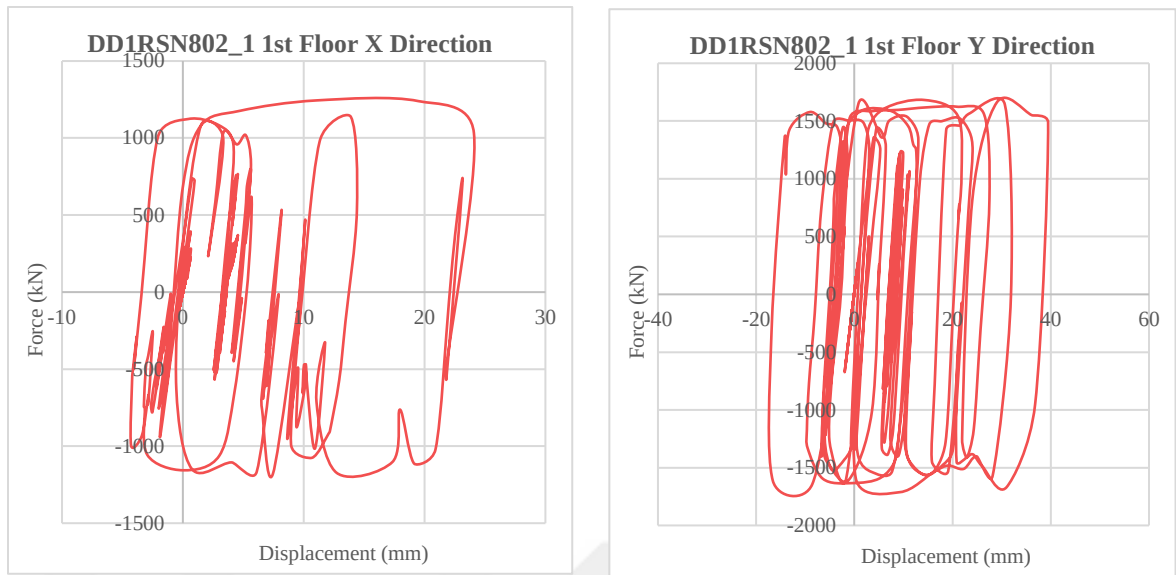


Figure 4.41. Force-displacement graphs of dampers at 1st floor under RSN802_1 (left graph is for X direction, right graph is for the Y direction).

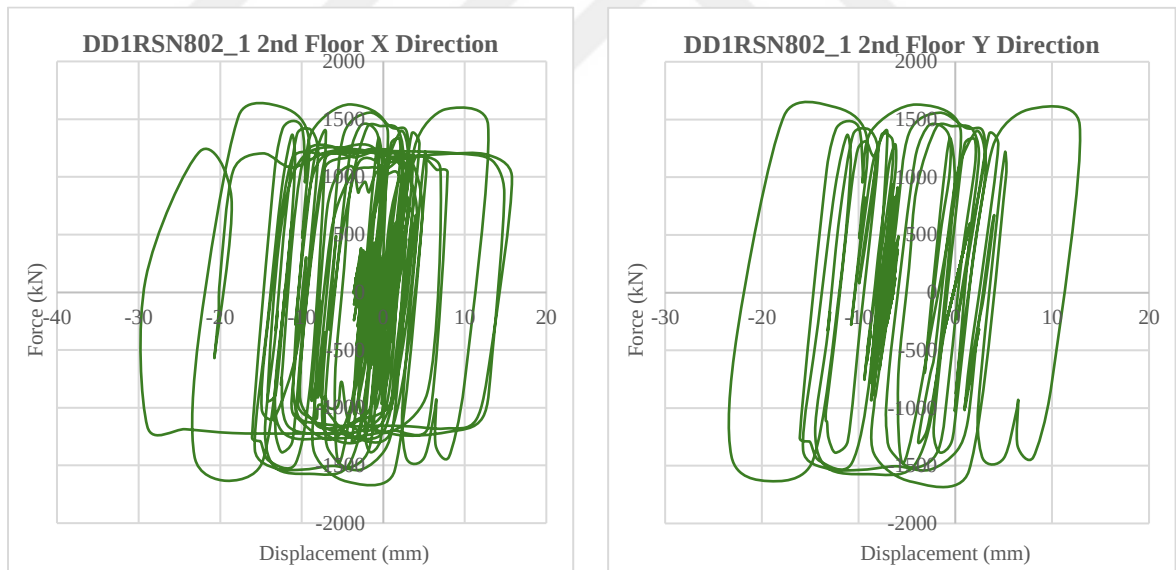


Figure 4.42. Force-displacement graphs of dampers at 2nd floor under RSN802_1 (left graph is for X direction, right graph is for the Y direction).

Story displacements of the retrofitted model are obtained as shown in below Figure 4.43 and Figure 4.44. Average values are represented with thick red and blue dashed lines.

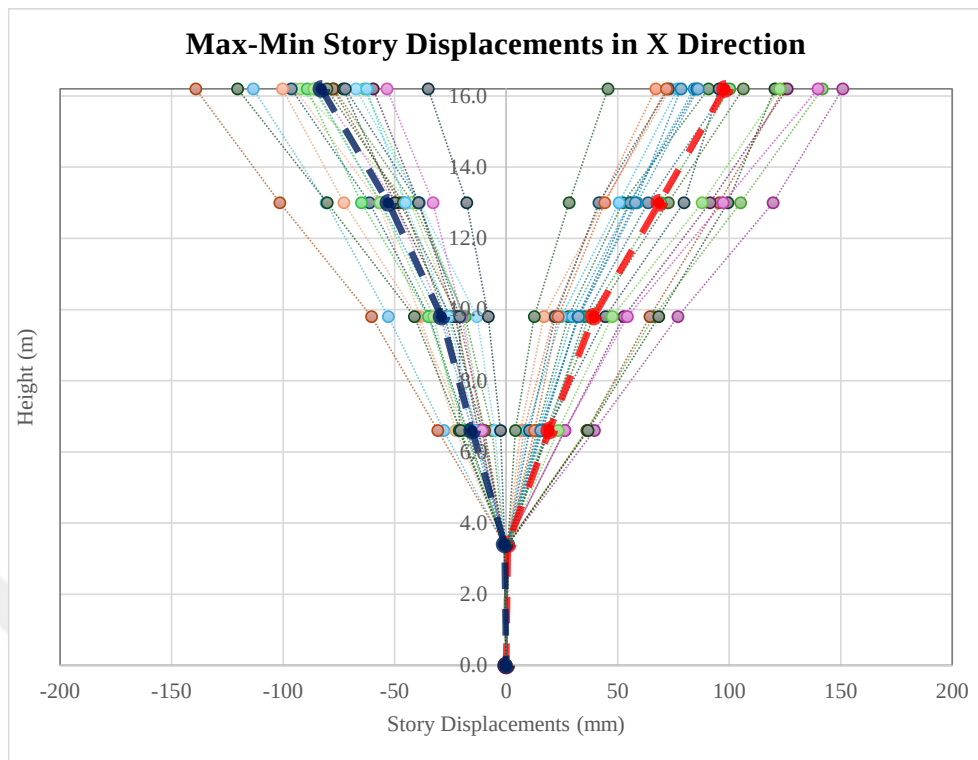


Figure 4.43. Maximum and minimum story displacements in X direction obtained from 22 ground motion analysis for the retrofitted model.

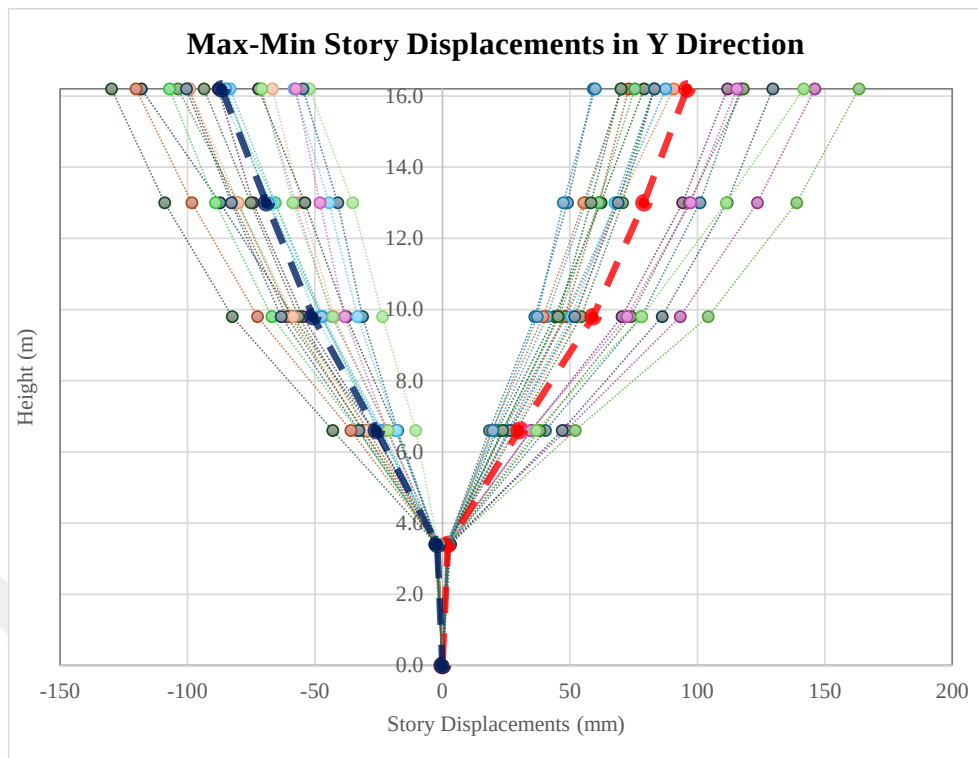


Figure 4.44. Maximum and minimum story displacements in Y direction obtained from 22 ground motion analysis for the retrofitted model.

Story drift ratios of the retrofitted model are obtained as shown in below Figure 4.45. Average values are represented with thick red dashed line.

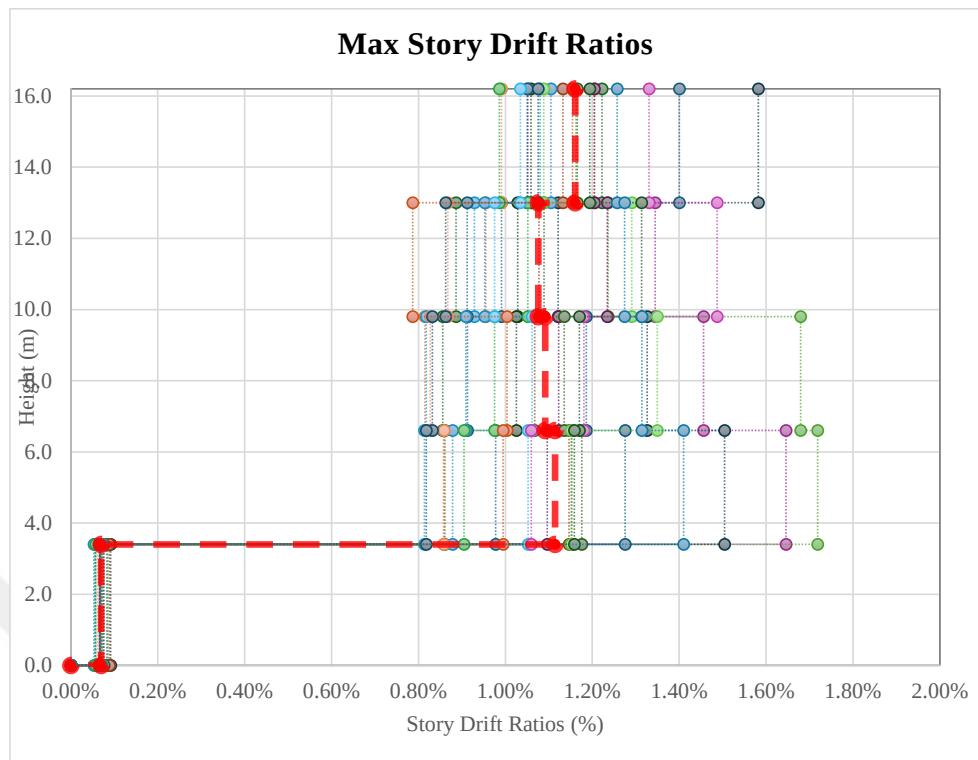


Figure 4.45. Maximum story drift ratios obtained from 22 ground motion analysis for the retrofitted model.

Story accelerations of the retrofitted model are obtained as shown in below Figure 4.46 and Figure 4.47. Average values are represented with thick red and blue dashed lines.

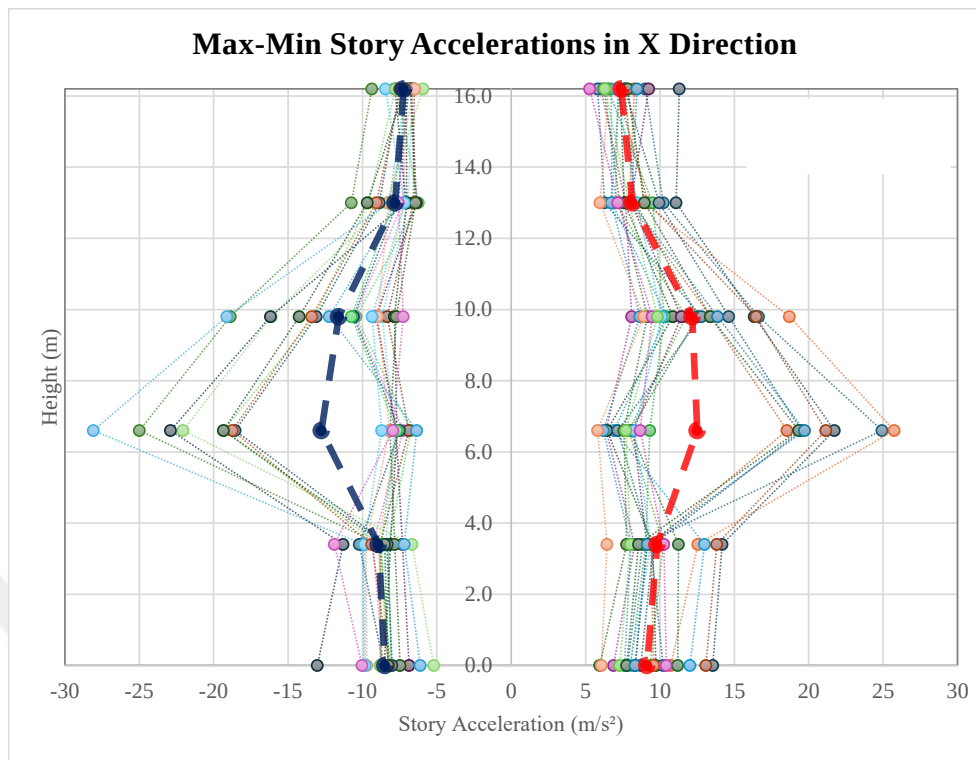


Figure 4.46. Maximum and minimum story accelerations in X direction obtained from 22 ground motion analysis for the retrofitted model.

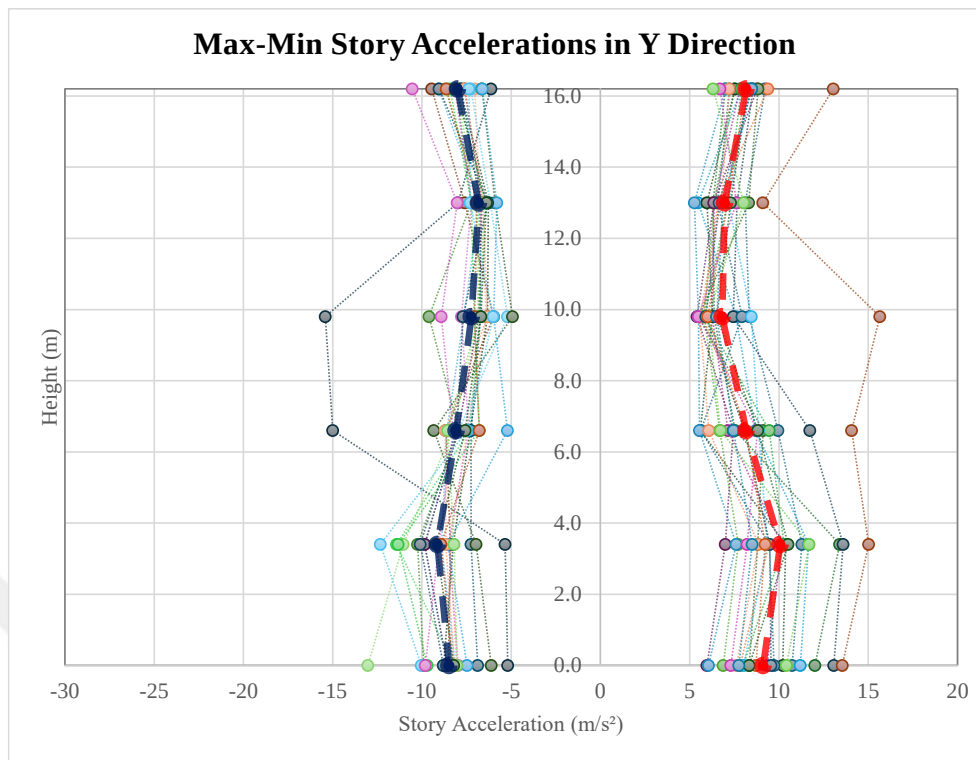


Figure 4.47. Maximum and minimum story accelerations in Y direction obtained from 22 ground motion analysis for the retrofitted model.

From the time history analysis, the additional equivalent damping added to the structure by the viscous dampers is calculated to be around 45-60% depending on the velocity content of the selected ground motions.

The cumulative energy components are shown on the Figure 4.47 below for some of the ground motions. Graphs presented on the left side of Figure 4.47 are for non-pulsive records such as (a) RSN779_1, (b) RSN3750_1, (c) RSN4031_1 and graphs presented on the right side of Figure 4.47 are for pulse-like ground motion records such as (d) RSN802_2, (e) RSN983_2, (f) RSN4228_1.

Upon analyzing the graphs, the most notable observation is the effective performance of the fluid viscous dampers. The majority of the seismic energy has been dissipated by the viscous dampers, while the energy dissipated through hysteretic damping due to nonlinear damage in structural elements remains relatively low. Another important finding is that the initial slopes of pulse-like records, where near-fault effects are prominent, are significantly higher compared to other ground motion records.

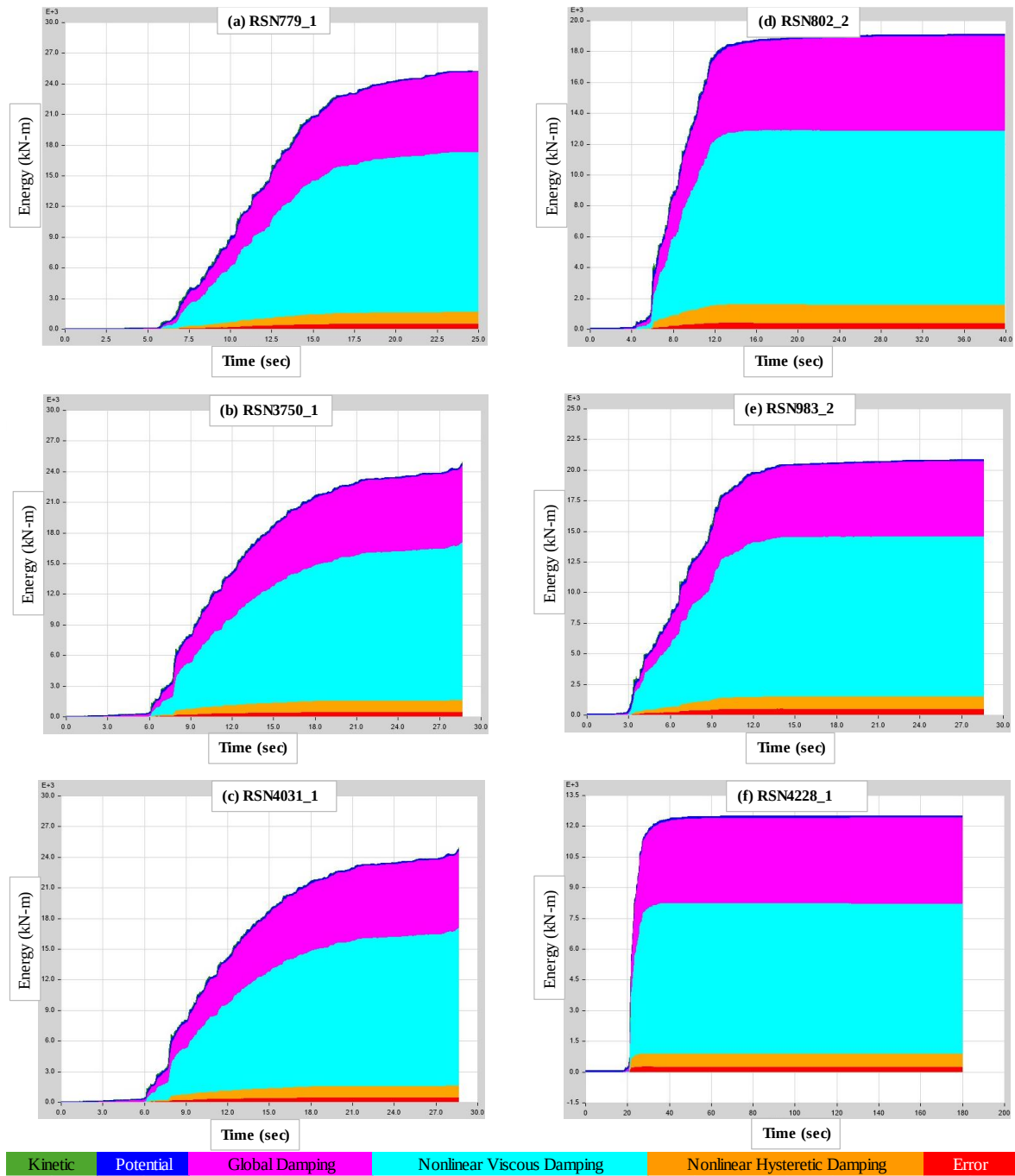


Figure 4.48. Cumulative energy components for selected ground motions. graphs on the left are for non-pulsive records (a) RSN779_1, (b) RSN3750_1, (c) RSN4031_1 and graphs on the right are for pulse-like ground motion records (d) RSN802_2, (e) RSN983_2, (f) RSN4228_1.

Top displacements of the original case of the building are compared with the retrofitted case (with FVDs) in below Figure 4.49. The average of maximum top displacements of the original case (under 22 ground motion records) are calculated as 263mm in X direction and 183mm in Y direction. Pushover results were also similar to these values (264mm and 212mm respectively). For the retrofitted case, the average of maximum top displacements under the same ground motions are calculated as 98mm in X direction and 95mm in Y direction. Therefore, the top displacements are reduced by around 65%. Comparison of top displacements are shown in below Figure 4.49. Thick red and blue dashed lines represent the average of maximum and minimum story displacements of the original respectively. Thick green dashed lines represent the average of maximum and minimum story displacements of the retrofitted model presented in above Figure 4.43 and Figure 4.44.

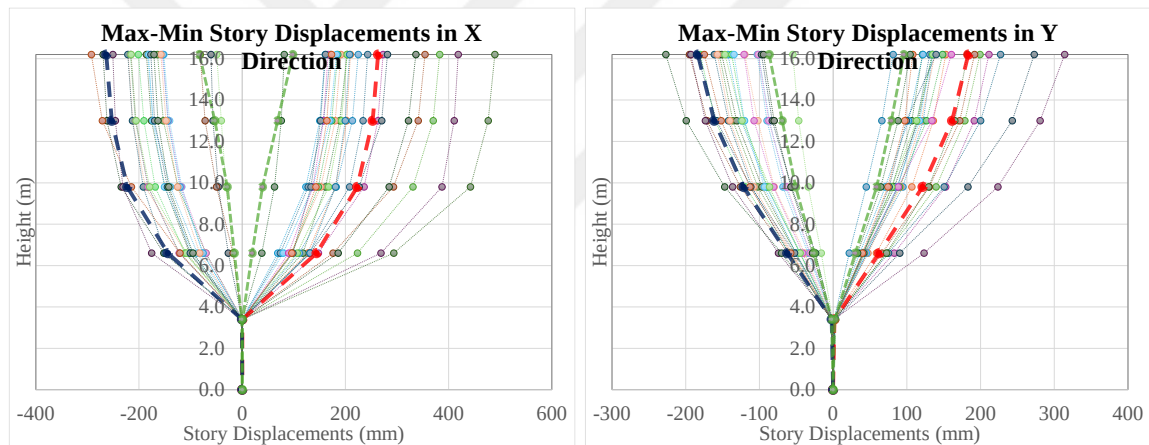


Figure 4.49. Maximum and minimum story displacements obtained from 22 ground motion analysis for the original case model.

Comparison of the top displacement between original case and retrofitted case under ground motion record RSN802 are shown below Figure 4.50. RSN802 is selected for comparison since it has pulsive content.

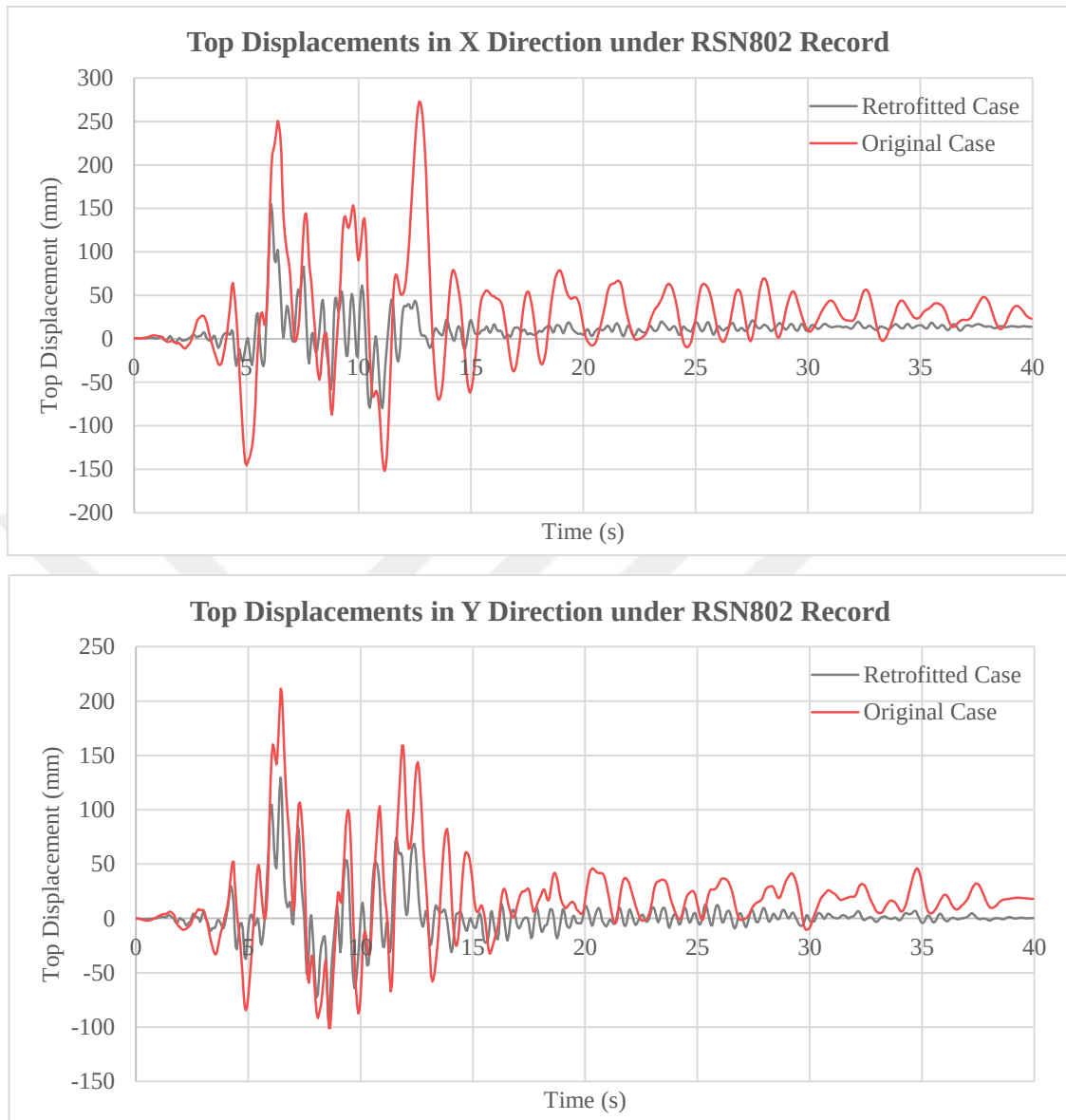


Figure 4.50. Comparison of top displacements (mm) from the analyses of original state of the building vs viscous damper added state under RSN802 ground motion record.

4.4. Comparison of the Applied Conventional Solution vs Proposed Viscous Damper Solution

4.4.1. Comparison of Costs

The studied building is retrofitted by IPCU according to the retrofit designs of Ülker Mühendislik Müşavirlik Ltd. in 2006 and 2019 respectively. The bidding price of the

construction works in 2006 couldn't be obtained. However, since the design drawings are available, the Bill of Quantities (BOQ) and the cost of these reinforcements is calculated based on the drawings.

The cost of the retrofit works applied in 2006, including the renovations that had to be made in architectural elements and installations, is calculated as approximately 26,903,773 TL (727,129 USD) with 2025 construction prices.

The BOQ study of the reinforcements implemented in 2019 is already available. The construction cost of the reinforcement, which was made in April 2019 by adding a total of 4 reinforced concrete shearwalls, 2 inside the building and 2 outside the building, was calculated as 1,031,932 TL (173,142 USD), with the compulsory architectural and electrical/mechanical costs added. With exchange rate of Turkish currency for 2025, the current cost of this application is calculated as 6,406,254 TL.

The total cost of the retrofit works using conventional methods is calculated as approximately 33,310,027 TL (900,271 USD).

The cost of demolition and reconstruction of the building is calculated as 74,321,520.00 TL (2,008,690 USD) from the average construction unit costs table published by the Ministry of Environment and Urbanization in February 2025. It is seen that the ratio of the total strengthening cost of the building (including compulsory plumbing and architectural renovations) to the reconstruction cost is approximately 45%.

In the retrofit method recommended as an alternative solution.

- All columns on the ground floor and first floor of the building are wrapped with steel.
- Columns on the second floor are wrapped with CFRP.
- 40 viscous dampers with 2275 kN axial load and ± 65 mm stroke capacity (1.3 times the average of maximum demands occur in 22 ground motions according to ASCE41-23[21]) were added to the building from the outside.



Figure 4.51. 3-D view of the building (representing the retrofitted case with FVDs) formed in Sketch-up Software.



Figure 4.52. 3-D view of the building (representing the retrofitted case with FVDs) formed in Sketch-up Software (cont.).

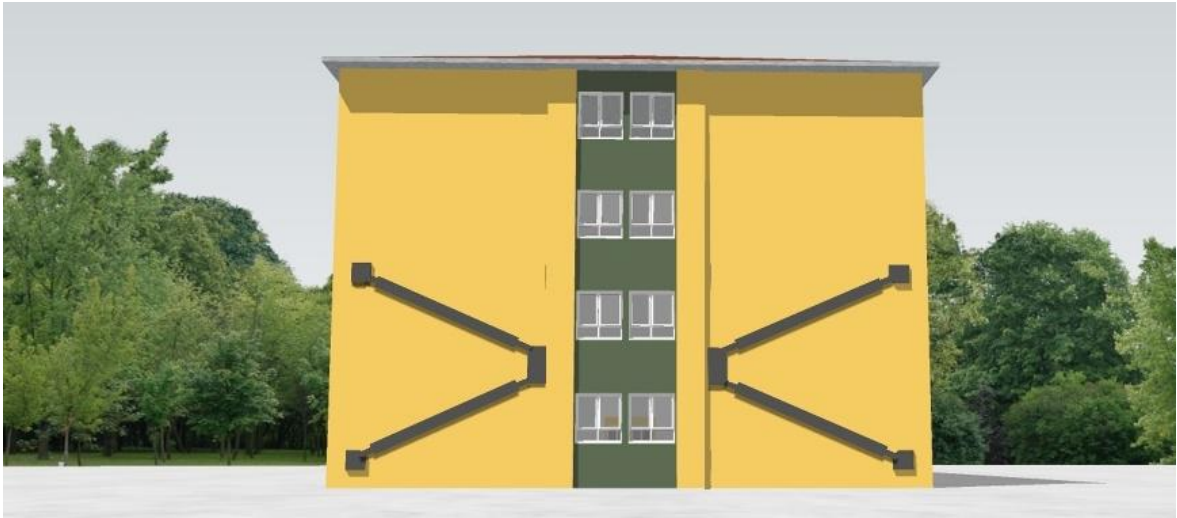


Figure 4.53. 3-D view of the building (representing the retrofitted case with FVDs) formed in Sketch-up Software (cont.).

The approximate cost of the proposed retrofit method was calculated as 29,739,490 TL (863,770 USD).

Table 4.22. Comparison of Costs of Retrofitting Alternatives and Demolishing / Reconstruction.

Method	Estimated Cost (TL)	Estimated Cost (USD)	Ratio
Applied Retrofit Cost	₺33,310,027.00	\$900,271	44.8%
Alternative Solution with Viscous Dampers	₺29,739,490.00	\$803,770	40.0%
Demolishing & Reconstruction	₺74,321,520.00	\$2,008,690	100%

4.4.2. Comparison of Structural Performance

Both conventional and viscous damper solutions comply with the TBDY-2018 performance requirements which is “Controlled Damage” under DD-1 earthquake level and “Limited Damage” under DD-3 earthquake level. Reconstructing the same building would need to comply with the same performance requirement. Therefore, one can say they are all

satisfying TBDY-2018 minimum requirements and they are all safe solutions. However, it is not compared which retrofit solution will perform better in bigger earthquakes.

While making a comparison, it should be kept in mind that viscous damper solution is governing also near fault effects. In other words, it is designed for a higher-level earthquake motion compared to the conventional solution.

4.4.3. Comparison of Business Interruption During Construction Works

There is no information about the construction time of the applied retrofit. Therefore, the comparison in terms of business interruption is based on estimations due to engineering experience.

Considering the conventional method, there are 20 shearwalls added to the building. 14 columns are jacketed with reinforced concrete. Foundations were added. In order to apply these enhancements, beams, slabs and partition walls are also partly demolished. Wet areas had to be demolished and reconstructed. Electrical and mechanical equipment needed to be displaced and/or remounted. As a result, the total construction time of the two conventional retrofits is estimated to be 6-8 months depending on the site conditions and contractor's capabilities. During this period, the building must be totally evacuated, and it will be out of operation.

In the proposed viscous damper alternative, there will be only steel and CFRP wrapping works inside the building and since these works are relatively simpler applications, there will be no need to completely evacuate the building. Partial evacuations can be organized while the building is on service at the same time. The construction time is estimated to be 4-8 weeks depending on the site conditions and contractor's capabilities.

Demolishing and reconstruction of a building with 4,200 m² is estimated to be 18-24 months depending on the site conditions and contractor's capabilities.

5. CONCLUSION

In this study, the imperative need for seismic retrofitting solutions that minimize business interruption and construction complexity is addressed. The Büyük Halkalı Primary School Building in Istanbul served as a case study, having undergone conventional retrofits in 2006 and 2019, necessitating the evacuation of the building each time. Recognizing the limitations of conventional retrofit methods for existing structures and the resultant resistance of building owners, our study proposed an innovative retrofit solution utilizing external viscous dampers, aiming to significantly reduce construction-related disruptions.

The seismic retrofitting methodology employed external viscous dampers strategically placed on the building's exterior, minimizing internal construction works and allowing the structure to remain in service during retrofitting. The study also considered the near fault effects of ground motion, an aspect not covered by the Turkish Seismic Code TBDY-2018. Viscous dampers have been considered a highly suitable retrofitting method for this building due to the following reasons:

- Absence of shear walls: Since the building does not have shear walls, the inter-story drifts and velocities are at levels that allow the dampers to function effectively.
- High velocity content in selected pulse-like records: Given that viscous dampers are velocity-dependent devices, the high velocity content in the selected pulse-like ground motion records does not pose any adverse effects on their performance.
- Minimal stiffness contribution and reduced foundation demand: Due to the inherently low stiffness of viscous dampers, they generate lower foundation forces compared to Buckling-Restrained Braces (BRBs), thereby reducing or even eliminating the need for additional foundation strengthening.

It was shown that viscous dampers work effectively under near fault effects. The top displacements are reduced around 65% compared to the original case of the building while achieving a damping ratio between 45-60% depending on the velocity content of the ground motion record.

Fluid Viscous Dampers have demonstrated their effectiveness as a seismic retrofitting solution. Their ability to dissipate energy efficiently while minimizing structural modifications makes them a viable alternative for improving the seismic resilience of existing structures. By reducing peak displacements and interstory drifts, FVDs significantly enhance the overall stability of buildings subjected to seismic forces.

Selecting an appropriate damping exponent α is crucial in optimizing the performance of fluid viscous dampers. In this study, an α value of 0.1 was chosen to effectively limit damper forces under high-velocity content ground motions. This selection ensures that the dampers operate within their optimal performance range, providing enhanced energy dissipation without inducing excessive forces on the structural components.

Despite their advantages, fluid viscous dampers alone may not be sufficient in structures with insufficient confinement, poor concrete quality and very high seismic demand. Such buildings may require additional reinforcement measures. In such cases, a combination of damping devices with supplemental strengthening techniques such as steel jacketing, CFRP wrapping etc. is recommended to ensure adequate seismic performance like the solution proposed in this case study.

This comprehensive evaluation included a comparison with conventional retrofit methods, considering structural performance, business interruption duration, and construction costs. The results demonstrated that the proposed alternative retrofit solution, featuring viscous dampers, steel jacketing and CFRP wrapping, not only met the required performance levels outlined in TBDY-2018 but also showcased advantages in terms of reduced construction costs and minimized business interruption. The estimated costs of the FVD solution were approximately 10% cheaper than the total cost of conventional retrofits. Additionally, the construction time for the proposed solution was estimated to be significantly shorter, ranging from 4 to 8 weeks, compared to the 6-8 months estimated for conventional retrofits.

In conclusion, our study proposes an alternative retrofit solution that not only meets seismic performance standards but also offers practical benefits in terms of cost-effectiveness and minimal business interruption, making it a promising approach for

retrofitting structures and potentially influencing seismic retrofitting practices on a broader scale.



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