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An Innovative Retrofitting Technique of an Industrial Prefabricated Building without Evacuation

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Abstract

This paper presents a seismic retrofit of an industrial-type precast reinforced concrete building structure using friction dampers. The project consisted of retrofitting two precast reinforced concrete buildings, one of which will be presented in this paper. Precast concrete is one of the industrial structures' most preferred construction methods due to its low cost, fast construction, and availability in rural areas. Unfortunately, most structures constructed before "Specification for Buildings to be Built in Seismic Zones (TBEC-2018) [1]" are not sufficiently engineered and are expected to perform poorly when exposed to a significant seismic event. The general characteristic of precast reinforced concrete buildings built before TBEC-2018 is their relatively high concrete quality (compared to cast-in-place reinforced concrete buildings) and better reinforcement workmanship. But they have some characteristic weaknesses as well. Weak beam-column connections, high drift ratios due to heavy beams, instability problems, and lack of frame behavior can be considered primary weaknesses. General classical retrofitting techniques include increasing beam/column sections or adding new reinforced concrete or steel members to the system or FRP (fiber-reinforced polymer) wrapping. Mostly classical retrofitting needs additional foundations. All these techniques require a long construction period, causing evacuation and stopping building usage. Because stopping the production cost can be tolerated by the owners, a retrofitting approach that can be assembled during the normal function of the industrial structures is required. As a result of this requirement, friction dampers are selected as the supplemental energy dissipation device. ASCE 41-17 [2] and ASCE 7-16 [3] are employed for the damper design and damping calculations. Before the damper study, some instability and weak connection problems are solved by traditional strengthening measures. The most effective damper configuration and capacities are selected after an iterative trial-and-error linear study using the simplified methods developed by PROMER. Nonlinear push-over analyses are conducted after linear pre-studies. Finally, a nonlinear time-history analysis is performed for confirmation of the results. It is shown that the proposed retrofit scheme satisfies the desired performance goals for both DD-1 [1] and DD-2 [1] events. Overall, it is considered that the proposed retrofit scheme with dampers provides the optimal solution for the project stakeholders from a performance, design, constructability, and economic point of view. Some application visuals will be presented, and methods will be explained in detail in the article. The friction damper behavior is based on the rotational friction hinge concept. The dampers increase building damping, causing a decrease in earthquake demand. As a result, dampers provide passive energy dissipation and protect buildings from structural and nonstructural damage during moderate and severe earthquakes.

Key words: Retrofitting, Friction dampers, Precast structures, Earthquake protection, Passive control

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1. Introduction

Earthquakes have posed the most significant threat to life and caused the highest economic losses in Turkey. Since 1900, 76 earthquakes have resulted in approximately 90.000 fatalities, impacting a total population of 7 million and causing direct losses exceeding US$25 billion. Notably, about half of these fatalities occurred due to two major earthquakes on the North Anatolian Fault in 1939 and 1999. The 1999 Marmara earthquakes, affecting ten cities (including six in the Marmara Region), resulted in a death toll exceeding 18,000 and a direct economic impact estimated at US$5 billion (2.5% of GNP).

Prefabricated buildings, particularly those constructed before the implementation of the 2018 Turkish Earthquake Code (TBEC-2018), exhibit significant vulnerabilities in earthquake resistance. Numerous studies have identified and documented these weaknesses [4, 5].

Prefabricated structures are often the preferred choice for industrial buildings in Turkey. Research indicates that 75-85% of industrial buildings in Turkey's industrial regions are prefabricated [6, 7]. Moreover, research has shown that 80% of prefabricated buildings in the Adapazarı Organized Industrial Zone suffered partial or total damage during the 1999 Marmara earthquake [4, 5].

One of the main challenges in retrofitting industrial buildings is the need to cease operations during construction, which can be costly and disruptive. This study presents a novel approach to retrofitting a reinforced concrete prefabricated building using friction-type energy dissipating devices (FDD) without interrupting operations.

The retrofitting design process involves three main stages:

1- Data Collection: This stage includes material and soil testing, assessment analysis, and building design. Geometrical data is collected from the building, architectural and structural drawings are prepared, and material tests are conducted to determine concrete quality. The existing concrete compressive strength is determined as 26,3 MPa as per TBEC-2018. Reinforcement is determined as StIII using destructive and non-destructive methods. The target performance level for the building is established as "limited damage" under DD-2 earthquake conditions.
2- Assessment Analysis: This stage utilizes the fixed single-mode pushover nonlinear static analysis method via Etabs ver. 18 [8], as described in TBEC-2018 sections 5 and 15.
3- Retrofitting Analysis and Design: Since FDD is preferred for retrofitting and TBEC-2018 does not provide design procedures for structures containing FDD, ASCE 41-17 and ASCE 7-16 are used as reference standards. The fixed single nonlinear static pushover method is employed for retrofitting analysis of the building with dampers. Additionally, nonlinear dynamic time history analyses are performed for two selected earthquakes, fulfilling the reviewer's request even though not explicitly required by codes. Furthermore, the design addresses stability issues identified in the building beyond solely addressing performance concerns.

This study demonstrates the successful implementation of FDD for retrofitting a prefabricated industrial building without interrupting operations. This approach offers a promising solution for enhancing the earthquake resistance of existing prefabricated structures while minimizing downtime and associated costs.

2. Building Information

The building consists of two adjacent blocks: B1 (3-storey precast reinforced concrete) and B2 (4-storey cast-in-place monolithic). Although both blocks were retrofitted using friction dampers, this paper focuses solely on B1. Table 1 summarizes some of their key properties.

Table 1. General building properties

Storey #	**B1 block 3 storey** **B2 block 4 storey**	**B1 BLOCK**			
Concrete Compressive Strength:	B1 block 26,3 Mpa B2 block 15,4 Mpa	Story Name	Story Number	Height (m)	Area (m2)
Area	B1 block 9.481,40 m2 B2 block 1.040,20 m2	Ground Floor	1	5,6	6320,75
Rebar type	ST420	1. Floor	2	2,6-3,6	1580,31
Foundation Type	Single footing	2. Floor	3	3,9-4,2	1580,31

The prefabricated reinforced concrete structure, designated as B1, serves as a warehouse for a food production factory. While the main building consists of a single story, one section of the building incorporates two administrative mezzanine floors (Figure 2). The total building dimensions are 90 meters x 77 meters with 4 x 22,5 meter and 10 x 7,7 meter axes in the long and short directions, respectively (Figure 1). The overall building height is 12,0 meters, with two mezzanine administrative floors positioned at levels of 5,5 meters and 8,2 meters within the designated axis (Figure 2).

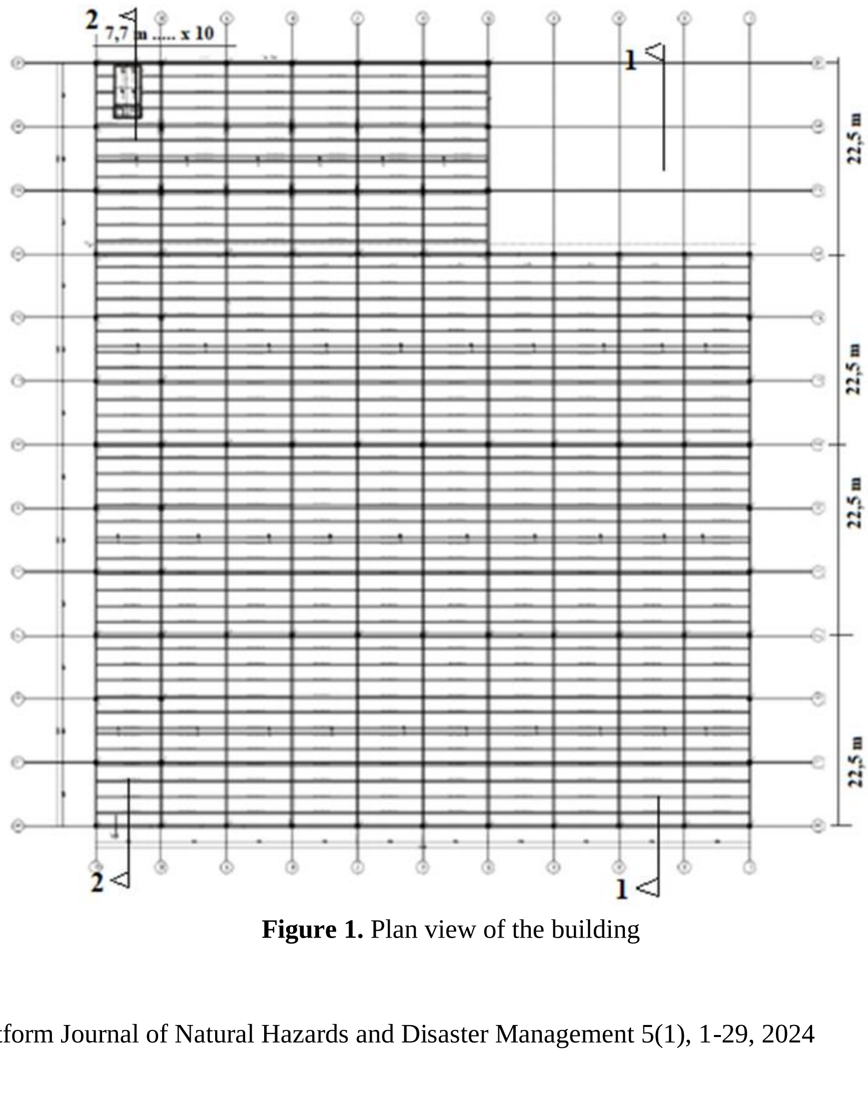


Figure 1. Plan view of the building

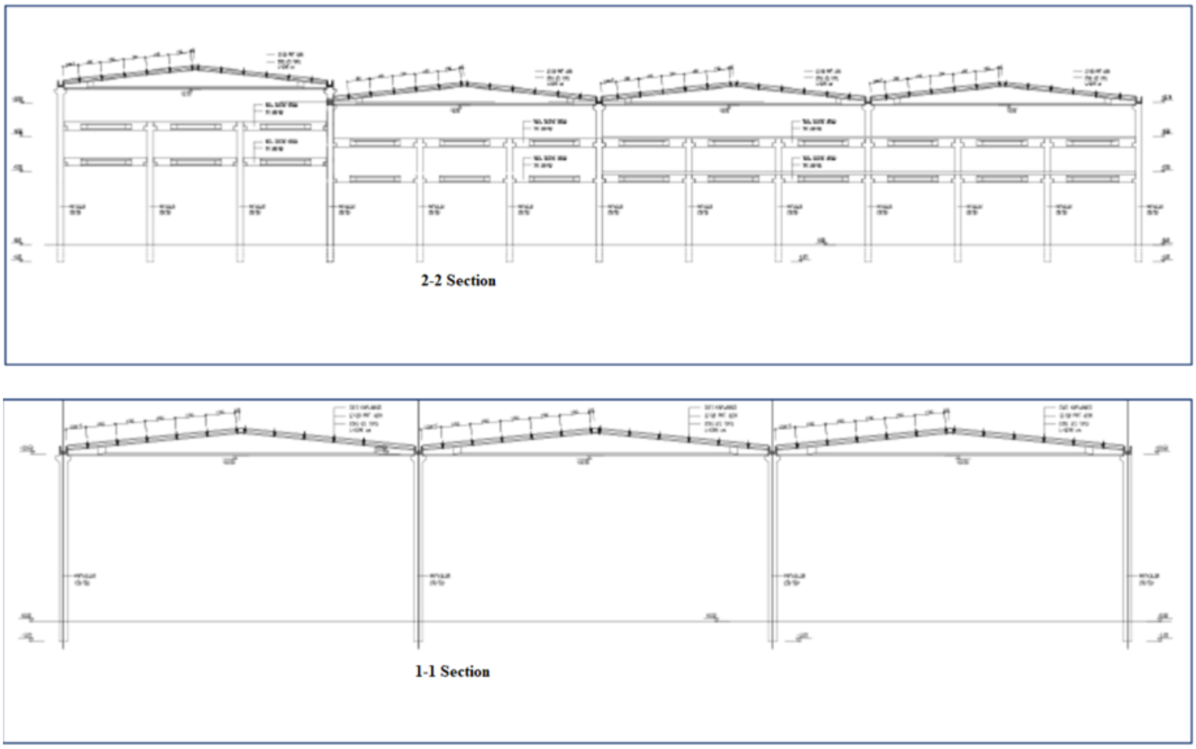


Figure 2. Section views of the building

Foundation System determination is based on construction project drawings provided by the client and confirmed by the excavation of two foundation test pits (Figure 3), the building utilizes a single-footing foundation system. The individual footings measure 360 centimeters by 360 centimeters with a height of 70 centimeters.

Figure 3. Foundation test pit excavation

2.1 Material Tests

While the client provided construction drawings for the prefabricated reinforced concrete (R/C) building, the lack of mechanical strength tests for the rebars necessitated a limited knowledge level approach for data collection. Therefore, a 0,75 capacity decrease was applied to member capacities according to TBEC-2018.

To assess concrete quality, 17 core samples were extracted from selected columns (Figure 4). The compressive strength was calculated based on TBEC-2018 and determined to be 26,3 MPa for the B1 block. Additionally, reinforcement type, orientation, diameter, and corrosion levels were determined for 11 columns using a stripping-destructive method (Figure 5).

a) b)

Figure 4. a) b) Concrete core sampling

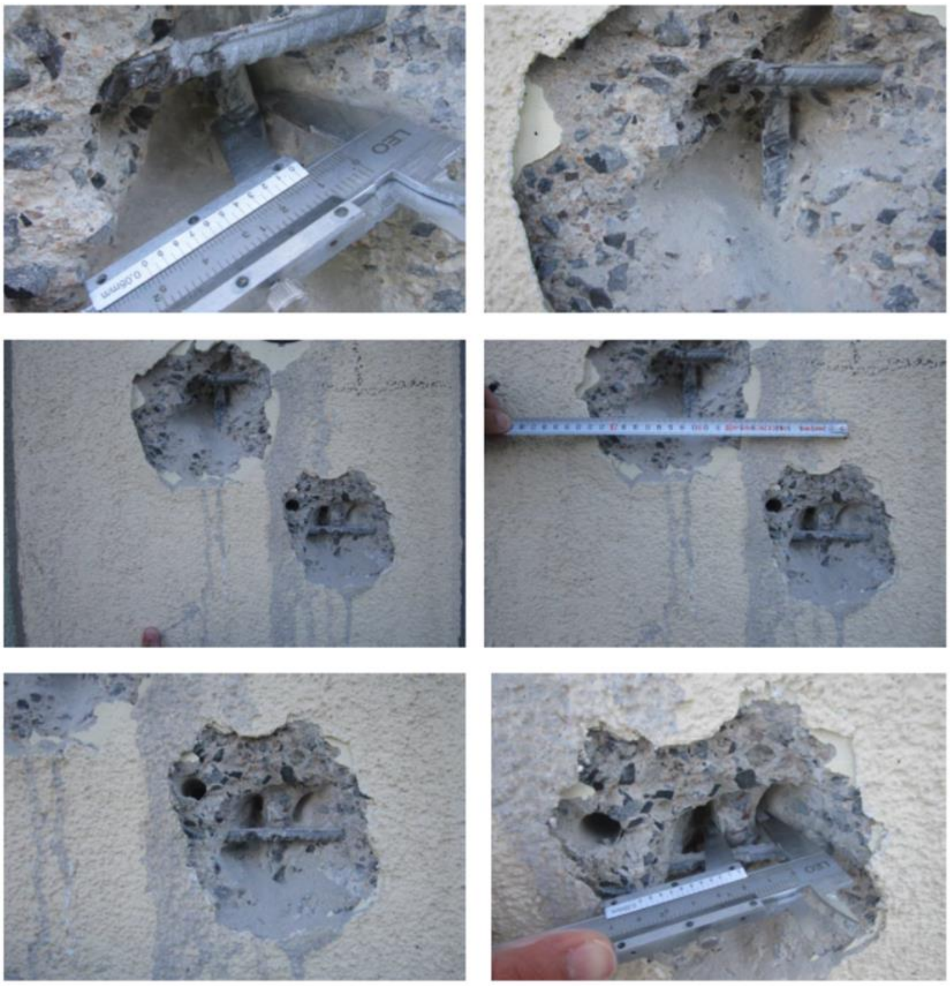
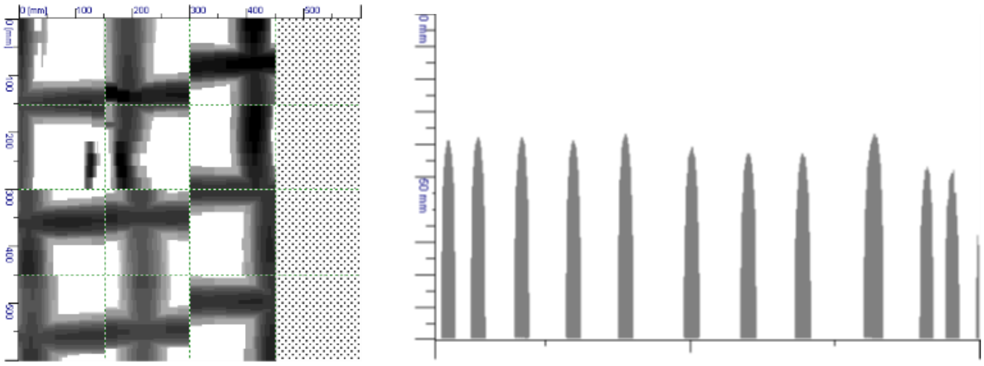


Figure 5. Reinforcement detection by stripping

Furthermore, Ferro-scan technology was employed on 19 vertical members to accurately determine the number and orientation of the reinforcing bars (Figure 6).



a) b)

Figure 6. a) 60cmx60cm scan view from a column b) Vertical scan view of a column starting from the bottom. (1,6m-1,8m length)

The determined reinforcement layout and details are presented in Figure 7 and Table 2. No corrosion and foundation settlement are detected in the structure.

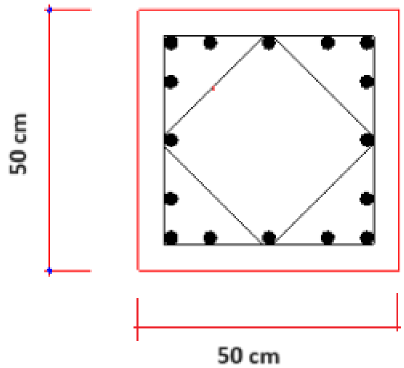


Figure 7. Determined reinforcement layout in a 50cmx50cm column

Table 2. Determined reinforcement details for column, shear wall, and beams

Member Type	Dimension (cmxcm)	Reinforcement Type	Vertical Reinforcement Diameter (pcs/mm)	Lateral Reinforcement Type	Lateral Reinforcement Span	Confinement
Column	50x50	StIII	16 pcs 20mm dia.	StIII	8 mm dia/ 20cm - 10cm inverval	No
Shear Wall	25	StIII	14mm dia / 18cm interval	StIII	12mm dia/ 20cm interval	No
Beam		StIII		StIII	8mm dia/ 20cm interval	No

3. Modeling and Assessment Analysis

3.1. Assessment Conditions

3.1.1 Structural Performance Criteria

The Turkish Earthquake Code (TBEC-2018) does not include specific analysis criteria for structures incorporating energy dissipation devices. However, it allows the use of recognized international codes for subjects not addressed within its regulations. Therefore, while earthquake and performance damage levels are based on TBEC-2018, damping analysis will be conducted according to ASCE 41-17 and ASCE 7-16.

TBEC-2018 defines three primary structural performance levels:

- Limited Damage Level (SH): Minimal damage to structural elements.
- Controlled Damage Level (KH): Repairable but significant damage to structural elements.
- Collapse Prevention (GÖ): Avoidance of total collapse, but potentially severe damage.

These performance levels correspond to varying degrees of damage in lateral force-resisting systems. Additionally, the design process considers four earthquake ground motion levels that are DD-1, DD-2, DD-3[1], DD-4 [1].

As a production facility, the minimum acceptable performance level for the building is Controlled Damage Level (KH) under the DD-2 earthquake level, according to TBEC-2018. However, this implies substantial, repairable damage. The client, therefore, requested the Limited Damage Level (SH) as the target performance objective.

TBEC-2018 provides descriptions of expected damage under each performance level:

- KH (Operational Usage): No or negligible damage to both structural and non-structural systems.
- SH (Limited Damage): Acceptable probability of limited structural damage. Basic vertical and lateral-force resisting systems retain almost all pre-earthquake strength and stiffness.

ASCE 7-16 further clarifies the state of non-structural components at the Immediate Occupancy (IO) performance level:

- Minor cracking of facades, partitions, and ceilings.
- Equipment and contents generally secure but may not operate due to mechanical failure or lack of utilities.

3.1.2 Building Assessment Parameters

Table 3 summarizes the parameters utilized in the assessment. Earthquake spectrum parameters were obtained from the interactive earthquake map provided by AFAD (Turkish Disaster and Emergency Management Presidency) using local soil properties and coordinates.

Table 3. Building assessment and loading parameters

Building Importance Factor (I)	1	Roof LL: 1,5 kN/m2	Corridor and Office rooms LL: 3,5 kN/m2
EQ Load Reduction Factor (R)	1	Mezzanine LL : 2,0 kN/m2	Snow Load: 0,75 kN/m2
Soil Type	ZB	Brick Wall weight : 3,2 kN/m2	
Cracked Section Rigidity Factor	0,4	Wind Load is as per TS EN 1991-I-3 (Eurocode 1)	
Soil Bearing Capacity (kPa)	400		

DD-1 Spectrum Parameters (AFAD)	DD-2 Spectrum Parameters (AFAD)	DD-3 Spectrum Parameters (AFAD)
S_S= 1,326 S_1= 0,367 PGA=0,540 PGV=33.247	S_S= 0,737 S_1= 0,213 PGA=0,308 PGV=19.186	S_S= 0,286 S_1= 0,087 PGA=0,122 PGV=8.087
S_{DS}= S_SxFS= 1,326x0,900= 1,193	S_{DS}= S_SxFS= 0,737x0,900= 0,663	S_{DS}= S_SxFS= 0,286x0,900= 0,257
S_{D1}= S_1xF1= 0,367x0,800= 0,294	S_{D1}= S_1xF1= 0,213x0,800= 0,170	S_{D1}= S_1xF1= 0,087x0,800= 0,070

LL: Live Load
S_S : Short period spectral acceleration multiplier (unitless)
S_1 : Spectral acceleration multiplier for 1 sec. Period (unitless)
S_{DS} : Short period design spetral acceleration multiplier (unitless)
S_{D1} : Spectral design acceleration multiplier for 1 sec. Period (unitless)
PGA: Peak Ground Acceleration
PGV: Peak Ground Velocity

3.2 Modeling

A three-dimensional (3D) model of the building was constructed using ETABS software (Figure 11). Live and dead loads acting on the roof were calculated and distributed appropriately to the supporting beams. On the mezzanine floors, live and dead loads were directly assigned to the respective slabs.

For the concrete behavior, the Mander model was utilized to capture both confined and unconfined conditions (Figure 10). For the steel behavior, the Kinematic model was chosen to accurately represent its nonlinear response.

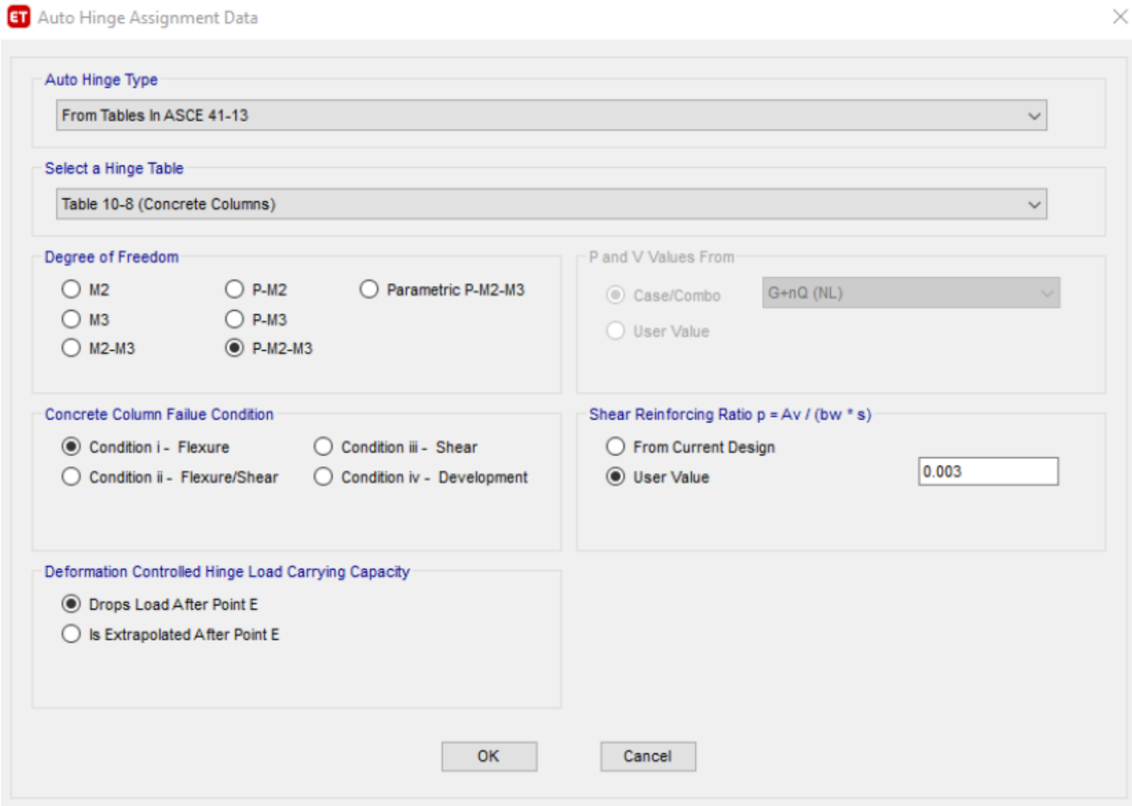


Figure 8. P-M2-M3 auto hinge modeling for columns

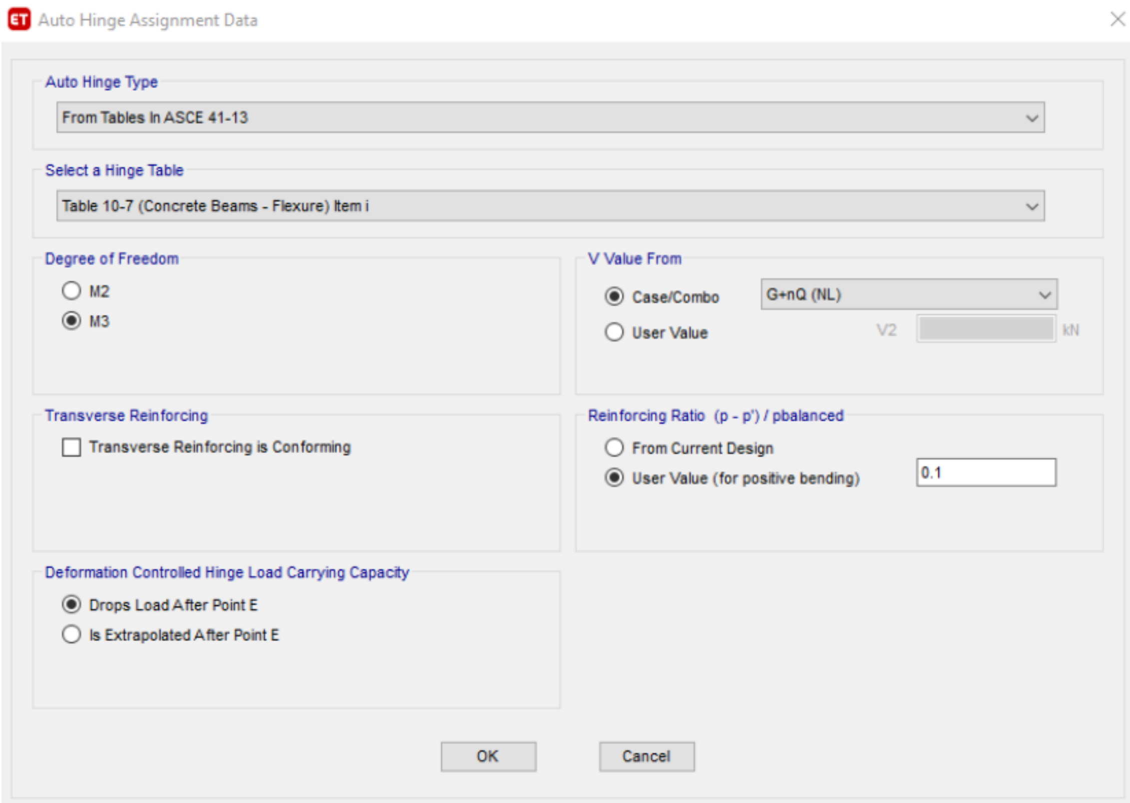
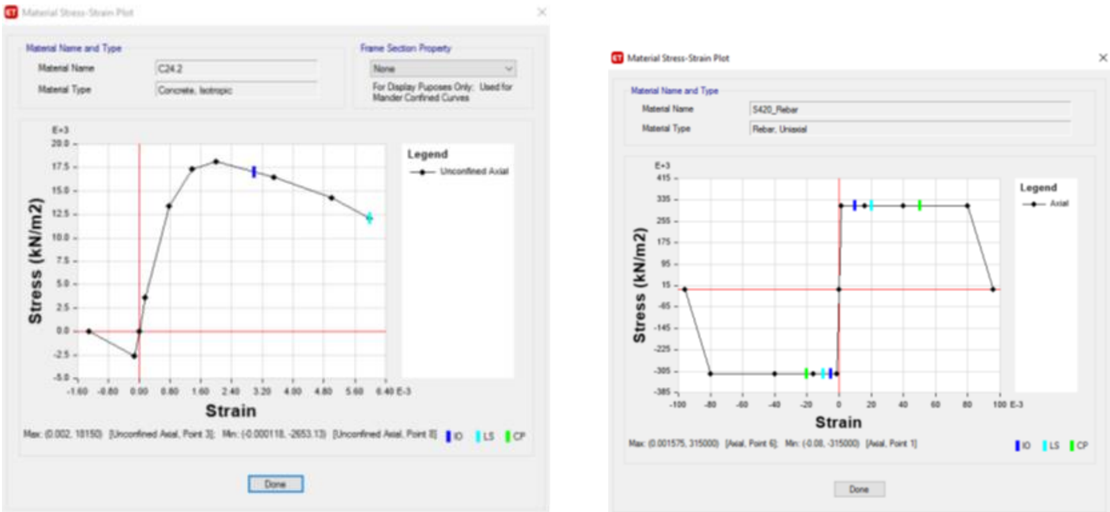


Figure 9. M3 auto hinge modeling for beams



a) b)

Figure 10. Nonlinear material model of a) concrete b) steel

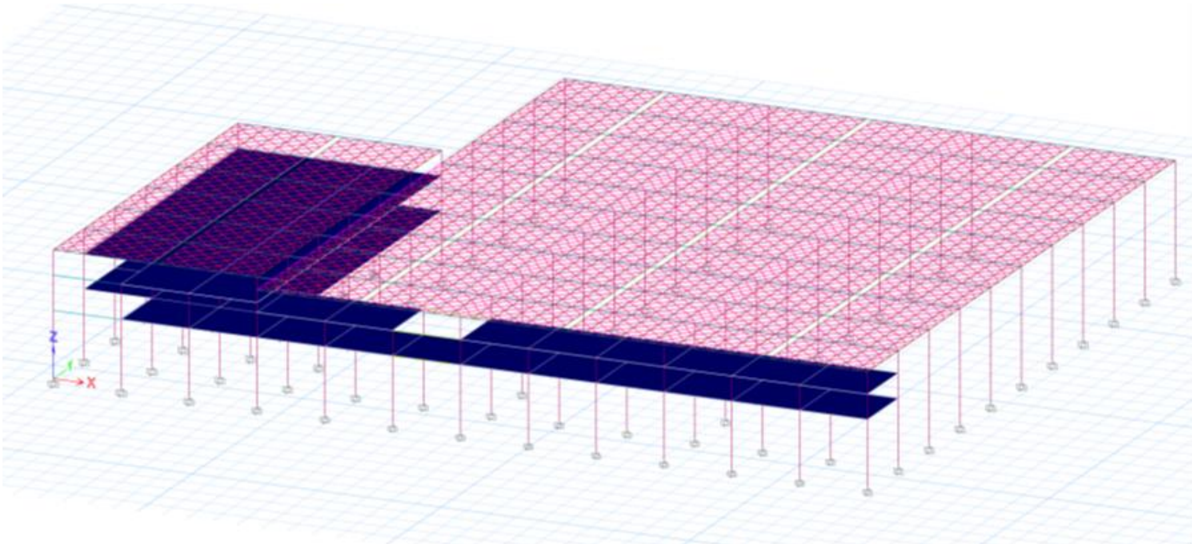
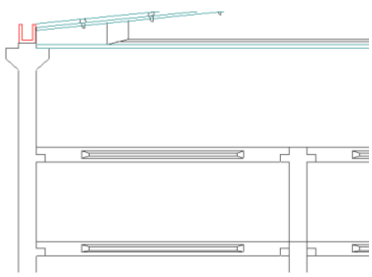


Figure 11. 3-D Etabs model perspective view

Both columns and beams were modeled as frame members within the Etabs software. Non-moment transferring beam column connections were represented as M3 releases and Nonlinear behavior was incorporated using designated hinge elements. Prefabricated beams with non-moment transferring connections accepted to be linear where as, columns utilized P-M2-M3 nonlinear hinges for comprehensive representation of axial and flexural behavior (Figures 8, 9, and 12).



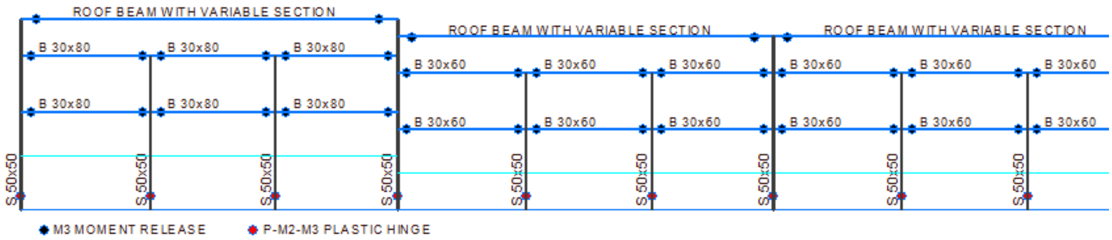


Figure 12. Nonlinear hinge definition for columns and beams

Modeling Roof Panel Rigidity:

The roof panel rigidity is modeled as per the definition outlined in TBEC-2018 Annex 8B (Figure 13). Rigidity is defined by the cross members connecting roof girder members at the panel level.
As specified in TBEC-2018, the rigidity of these cross members is calculated as (EA)e ≈ 400.000 kN.

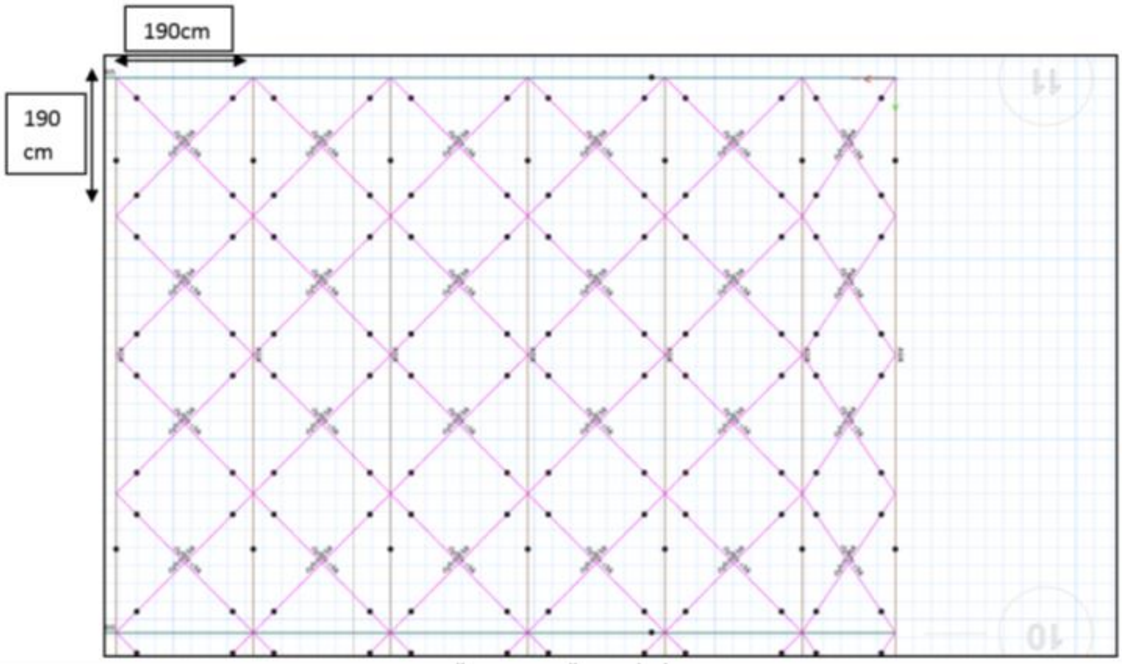


Figure 13. Roof rigidity modeling as per TBEC-2018 attachment 8B

3.3 Assessment Analysis of Existing Building
3.3.1 Linear Dynamic Analysis

A linear dynamic analysis (modal analysis) was conducted for the existing building conditions. The resulting modal participating mass ratios are presented in Table 4. The first mode corresponds to motion in the X-direction with a period of 3,686 seconds and a modal participation ratio of 0,891. The second mode represents Y-direction motion with a period of 3,33 seconds and a modal participation ratio of 0,7789.

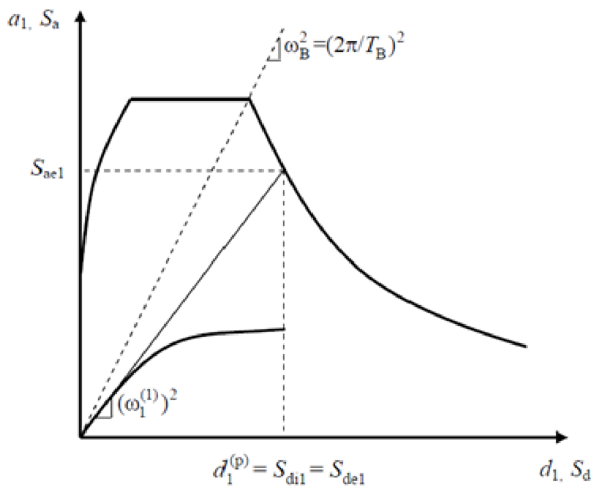
Since the effective modal mass participations exceed 0,70 and the story torsion remains within the acceptable limit (1,4), a nonlinear fixed first-mode pushover analysis will be employed as per TBEC 2018.

Table 4. Modal participating mass ratios

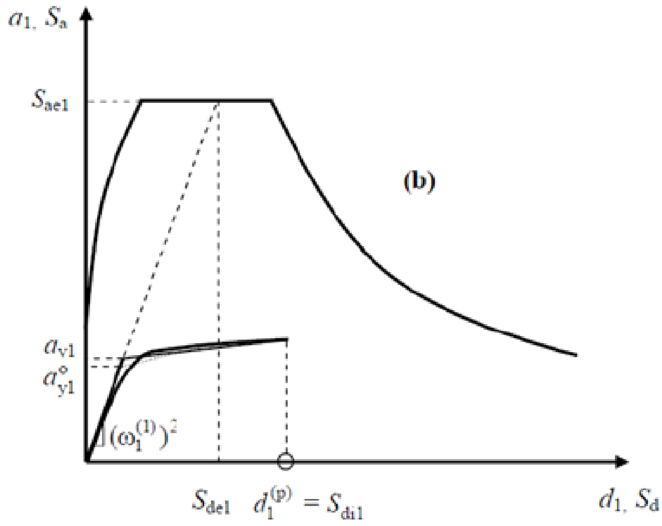
Case	Mode	Period (sec)	UX	UY
Modal	1	**3,686**	**0,891**	0,0003
Modal	2	**3,33**	0,0005	**0,7789**
Modal	3	3,031	3,81E-06	0,1275
Modal	4	2,49	0,0083	0,0002
Modal	5	1,544	0,0016	0,0006
Modal	6	1,247	0,0032	0,0019
Modal	7	1,116	0,0007	0,0072
Modal	8	1,077	0,0005	0,0035
Modal	9	0,99	0,0006	0,0001
Modal	10	0,94	0,0001	0,02
Modal	11	0,931	6,18E-07	0,0021
Modal	12	0,822	0,0051	0,0036

3.3.2 Performance Analysis Using Nonlinear Fixed First Mode Pushover

The procedure for fixed first-mode pushover analysis outlined in TBEC-2018 was followed (Figure 14). Pushover analyses were performed for both the X and Y directions separately. Each analysis was continued until the top floor displacement reached 4% of the building height, generating corresponding capacity curves (Figures 15 and 16).



a)



b)

Figure 14. Target displacement calculation as per TBEC-2018 a) T1 > TB   b) T1 < TB

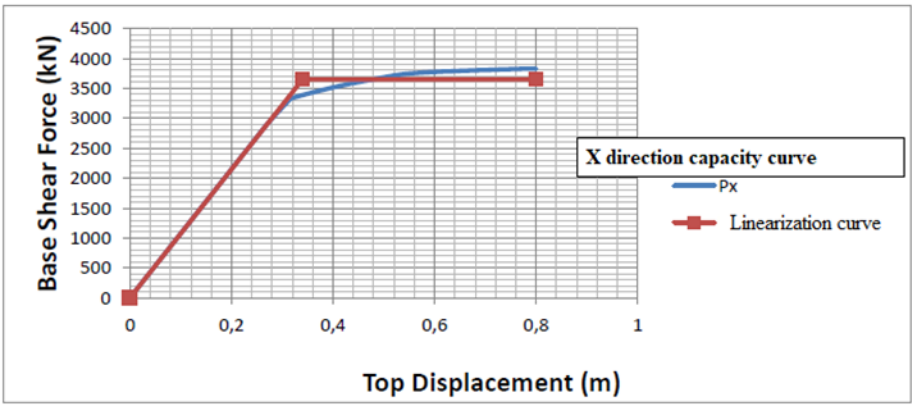


Figure 15. X direction capacity curve

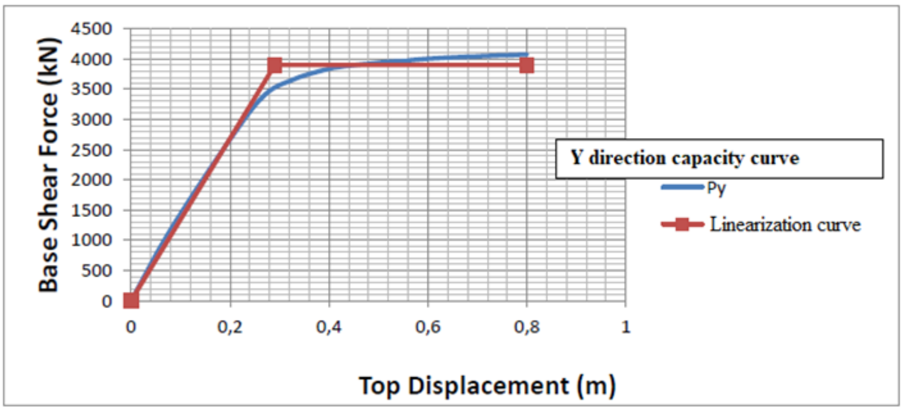


Figure 16. Y direction capacity curve

The target displacement was calculated following the procedure described in TBEC-2018 (Figure 14). The calculated target displacement and related parameters are presented in Table 5. Detailed calculations are not included in this paper as they conform to a standard procedure.

Table 5. Target displacement calculation for x and y direction

Tx (s)	Sae(T)/g	Sae(T)	Sde(m)	ay1	Ry1	CR1	Sdi1	Target Disp. (m) UxN1
3,686	0,04612	0,45244	0,15571	0,80325	0,56326	1	0,15571	0,155708931

Ty (s)	Sae(T)/g	Sae(T)	Sde(m)	ay1	Ry1	CR1	Sdi1	Target Disp. (m) UyN1
3,333	0,05101	0,50036	0,1408	0,85827	0,58299	1	0,1408	0,140797034

Tx (s): Natural vibration period
Sae(T): Lateral elastic design spectral acceleration
Sde(m): Lateral elastic design spectral displacement
ay1: So-called yield acceleration for the first mode.
Ry1: Correlation dependent on elastic spectral acceleration (Sae(T) and so-called yield acceleration (ay1).
CR1: Spectral displacement ratio.
Sdi1: Nonlinear spectral displacement.

Following the target displacement calculation, a new nonlinear fixed first-mode pushover analysis was performed up to the calculated target displacement to evaluate the building's performance.

3.3.3. Performance Analysis Results for Existing Building

The analysis results obtained at the target displacement of the pushover analysis are presented in Tables 6 and 7. While column rotations remained within the limits of the desired Limited Damage Level (SH) for the DD-2 earthquake level, beam rotations at the mezzanine level exceeded those limits. Notably, these beams were also identified as brittle.

Table 6. Column rotation and shear assessment results at target displacement for DD2 earthquake level

Column Damage Levels					
Storey	Limited Damage	Controlled Damage	Collapse Prevention	Collapse	Total
2	46	0	0	0	46
1	46	0	0	0	46

Column Shear Check			
Storey	Ductile	Brittle	Total
2	46	0	46
1	46	0	46

Table 7. Beam rotation and shear assessment results at target displacement for DD2 earthquake level

Beam Damage Levels					
Storey	Limited Damage	Controlled Damage	Collapse Prevention	Collapse	Total
2	0	6	0	0	6

Beam Shear Check			
Storey	Ductile	Brittle	Total
2	0	6	6

The soil bearing capacity was calculated as 400 kPa. All soil stresses under the foundations were evaluated under both static vertical loads and dynamic lateral earthquake loads, ensuring they remain below the bearing capacity. This analysis confirms that no foundation issues exist for the building.

Furthermore, linear modal analysis under the DD-2 earthquake level established that the building complies with TBEC-2018 requirements regarding storey torsion, soft story, and weak story irregularities. However, the performed story drift check, presented in Table 8, revealed that story drifts exceeding the limits for the desired Limited Damage Level were present.

Table 8. Storey drift check under DD-2 earthquake level

		Drift	Limited Damage Limit	Damage Level
X Direction Displacement	186 mm	0,014	0,01	Controlled Damage
Y Direction displacement	210 mm	0,016	0,01	Controlled Damage
Storey height	13.200 mm			

The analysis has identified potential stability issues within the building structure. While one direction of the building features a pin-connected frame system (Figure 2), the other direction lacks a complete framing system (Figure 1). This deficiency relies solely on girders to connect the individual frames, rendering it susceptible to instability under earthquake loads in this direction.

The assessment analysis revealed that while building columns meet the rotation and shear criteria for the desired Limited Damage Level (SH) under the DD-2 earthquake level, beams do not. Additionally, the building exhibits excessive story drift exceeding the limits for the SH level. This significant displacement, particularly concerning for a building height of 12,0 meters (Figure 2), poses a potential risk of damage to non-structural walls and elements.

Although the observed story drifts comply with the Controlled Damage Level (KH) performance level, the top floor displacement reaches 19 centimeters, potentially leading to substantial damage to non-structural components like walls and equipment. Furthermore, the analysis identified a risk of girder drop-down and damage to roof panels.

Considering these findings and associated risks, the client requested to enhance the structural performance to the Limited Damage Level (SH) as much as feasible. This emphasis on "as much as feasible" acknowledges that achieving the desired performance level surpasses the minimum code requirements (TBEC-2018), but significant improvement is desired despite not being mandatory. Story drifts associated with the Controlled Damage Level (KH) could result in extended repair times, financial losses, and even potential safety hazards. Additionally, the identified stability issues within the building require resolution.

Due to the client's desire to minimize production downtime during the retrofitting process, traditional methods like adding reinforced concrete shear walls, column jacketing, and techniques involving wet concreting, molding, and excavation are unsuitable. Therefore, a retrofit strategy using dampers emerged as the preferred solution. Friction-type dampers were chosen for this purpose, and their application is described in the following section.

4. Retrofitting of the Building
4.1. Analysis Methods

The proposed retrofitting plan, including the application of friction-type dampers, necessitates the use of additional codes alongside the existing TBEC-2018 standard. Since TBEC-2018 does not explicitly address the design and analysis of structures incorporating damping devices, ASCE 7-16 and ASCE 41-17 will be employed as the primary reference codes in tandem with TBEC-2018. This multi-code approach ensures comprehensive guidance for the design and analysis of the damped structure.

The main objectives were;

Linearity of Steel Frame and Connections: Ensure that the steel frame system and connections associated with the damping system remain within the elastic range under the design earthquake forces corresponding to the DD-2 level specified in ASCE 7-16 Section 18.2.1.2.

Overstrength of Steel Frame and Connections: Design the capacity of the steel frame system and connections involved in the damping system to be at least 20% greater than the member demands calculated under the DD-1 earthquake level, as per ASCE 7-16 Section 18.2.1.2. This ensures adequate reserve strength to accommodate potential uncertainties and variations in loading.

Displacement Capacity of Dampers: Design the selected friction dampers to possess a displacement capacity that is at least 30% higher than the maximum displacement experienced by the dampers under the DD-1 earthquake level, adhering to ASCE 7-16 Section 18.2.4.6. This additional capacity provides a safety margin against potential overload scenarios and ensures the long-term functionality of the dampers.

Two earthquake levels were considered for the retrofitted building: DD-2 and DD-1, with each employing a specific analysis method:

DD-2 Earthquake Level: Modeling, analysis, and assessments: These processes were conducted according to TBEC-2018 sections 15 and 5 under the DD-2 earthquake level.

Fixed first-mode nonlinear pushover analysis: This method was chosen due to the structure's behavior satisfying the relevant conditions outlined in TBEC-2018 Section 5.6.2.2. It provides a computationally efficient approach for evaluating the building's global seismic performance.

DD-1 Earthquake Level: Nonlinear time history analysis: This method was selected as the primary analysis technique under the DD-1 earthquake level due to the potential for nonlinear behavior in the structure, including the nonlinearity introduced by the friction dampers.

Linear modal analysis: This method was considered as a permissible alternative if the structure exhibited linear or almost linear behavior. However, in this study, despite the retrofitted structure displaying almost linear behavior, a time history analysis was still performed.

Two earthquake records: To capture the potential variations in seismic loading, two earthquake records were utilized for the time history analysis: Düzce and Erzincan. This approach aligns with recommendations presented in 5ICEES congress papers [9-10], ensuring a comprehensive evaluation of the structure's response under realistic earthquake scenarios.

4.2. Final Retrofitting Studies
4.2.1. Modeling

The proposed retrofitting design for the industrial facility incorporates several key considerations to ensure minimal disruption to operations:

Avoiding Interference with Critical Equipment and Lines: The location of dampers and associated bracing systems was carefully determined to avoid any interference with essential equipment, pipelines, or other important lines within the facility. This involved extensive study and consultation with the client to identify critical areas and ensure that the retrofitting work would not obstruct vital operations.

Minimizing Workflow Disruption: The retrofitting plan was designed to minimize the impact on the facility's workflow. This involved, for example, scheduling construction activities during off-peak hours or utilizing temporary bypasses for critical systems to maintain uninterrupted operation.

Optimized Damper Locations: Following numerous trial studies and discussions with the client, the final locations of the dampers were fixed, as shown in Figures 17, 18, and 19. This optimized placement ensures efficient energy dissipation and improved seismic performance while minimizing the overall impact on the facility's layout and operations.

Damper Selection and Modeling: For the X-direction, 44 pieces of 60 kN dampers were chosen, while for the Y-direction, 52 pieces of 120 kN dampers were selected. The hysteretic, linear, and nonlinear behavior of the dampers was accurately modeled in the Etabs software using plastic Wen spring modeling. This ensured a realistic representation of the damper behavior during the analysis and design process.

Modeling Approach: A linear model with effective stiffness was used for equivalent viscous damping calculations. For nonlinear pushover and nonlinear time history analysis, a bilinear plastic model was employed. This approach provided accurate results while maintaining computational efficiency.

Consistent Load Modeling: Loads, load combinations, and nonlinear hinge properties were modeled in the same manner as described for the existing performance assessment of the structure (refer to Sections 3.1 and 3.2). This ensured consistency and facilitated comparisons between the pre- and post-retrofitted states of the building.

Bracing System Design: Toggle-type steel braces were used to support the dampers in the Y-direction (Figure 20), while diagonal-type steel braces were chosen for the X-direction (Figure 17).This selection was based on considerations of efficiency, ease of construction, and compatibility with the overall structural system. S275 type structural steel was used for the braces and frames.

By carefully considering these factors, the retrofitting design was optimized to achieve the desired performance improvements while ensuring minimal disruption to the functionality of the industrial facility.

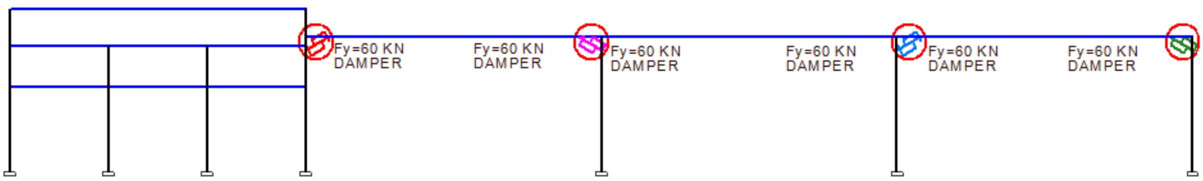


Figure 17. Damper application in X direction (dampers marked with red circles are in a diagonal position close to the top of the columns and between the column and beams)

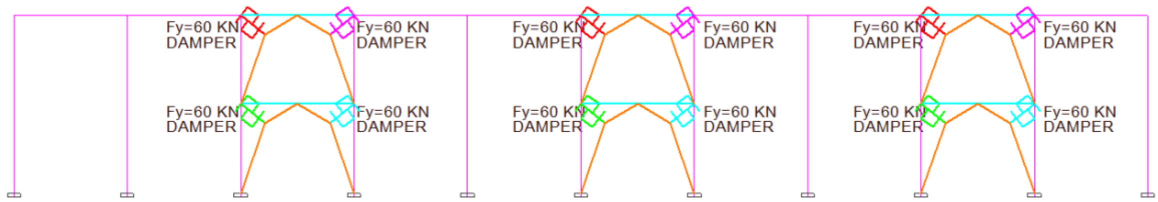


Figure 18. Damper application in Y Direction (dampers are in toggle braces)

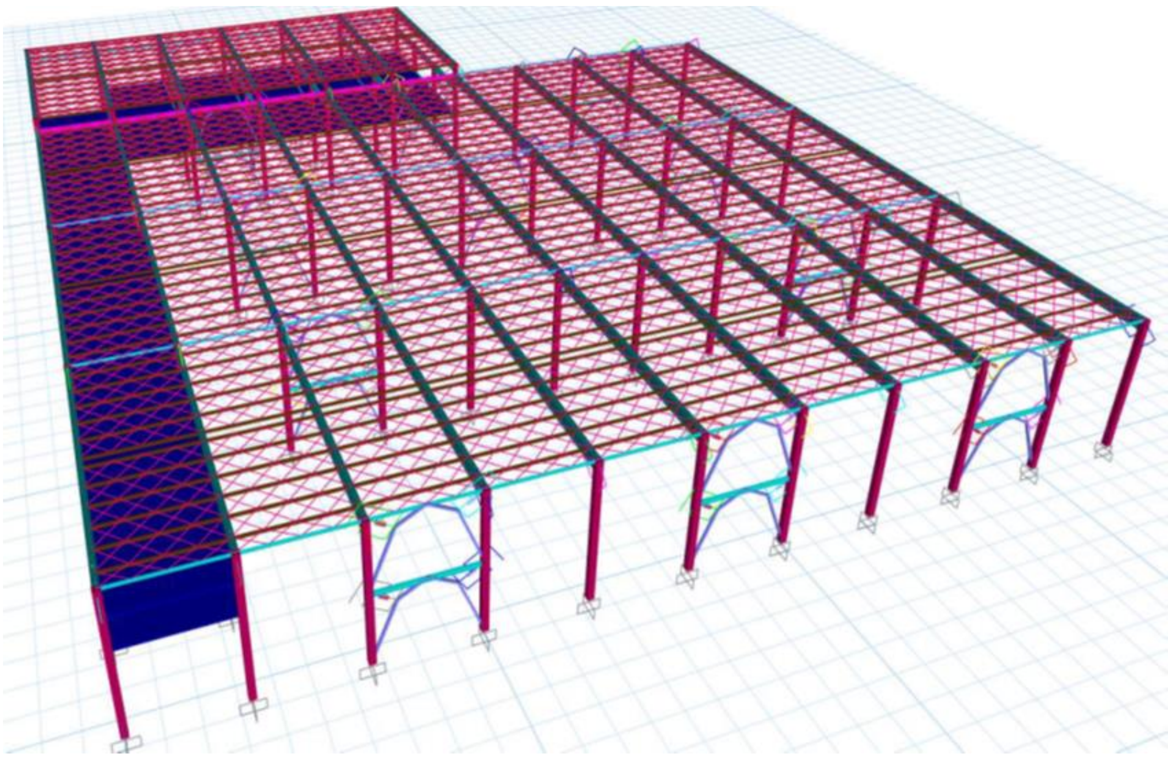


Figure 19. Etabs model view of damper application

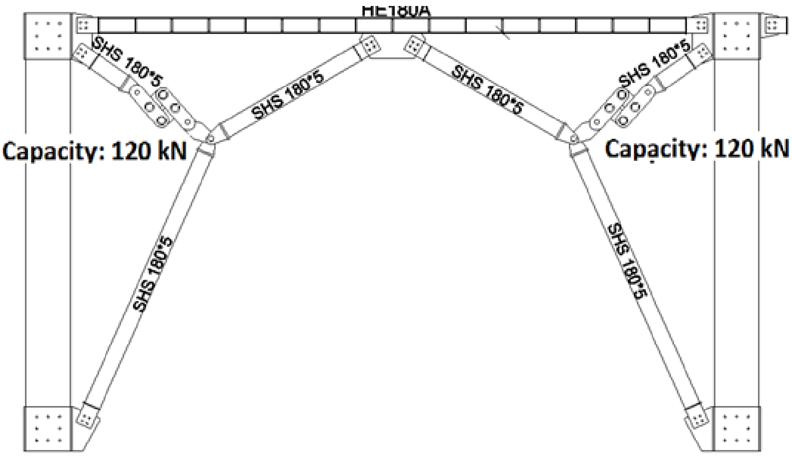


Figure 20. Damper location in toggle type of bracing

4.2.2 Damping Analysis

Equivalent viscous damping calculations were performed for the retrofitted structure using ASCE 7-16 section 18.7.3.2.2 and equations 18.7-47 and 48. The calculated parameters are presented in Table 9.

As intended, the additional damping added to the system through the seismic dampers is higher in the Y-direction (15,8%) compared to the X-direction (6,55%). This is due to the placement of more dampers with higher capacities in the Y-direction.

Consequently, the reduction factors calculated for the X and Y directions are 0,79 and 0,67, respectively. This translates to a 21% decrease in structural demands for the X-direction and a more significant 33% decrease in demands for the Y-direction.

$$\beta_{HD} = q_H (0.64 - \beta_I)\left(1 - \frac{1}{\mu_D}\right)$$: ASCE 7-16, section 18.7.3.2.2 equation 18.7-47

$$\beta_{HM} = q_H (0.64 - \beta_I)\left(1 - \frac{1}{\mu_M}\right)$$: ASCE 7-16, section 18.7.3.2.2 equation 18.7-48

Table 9. Viscous damping calculation of B1 block under DD-1 and DD-2 earthquake level-linear model

	Initial	Effective
Tx Period (Sec.)	1,808	2,05
Ty Period (Sec.)	0,952	1,398
Equivalent Ductility -X	0,777838	
Equivalent Ductility -Y	0,463724	
Additional Damping -X	0,065538	
Additional Damping -Y	0,158201	

Under DD-1 EQ Level		Under DD-2 EQ Level	
Reduction factor - X	0,79	Reduction factor - X	0,77
Reduction factor - Y	0,67	Reduction factor - Y	0,60

4.2.3 Assessment Analysis of Retrofitted Structure

The assessment analysis of the retrofitted building will be performed following Section 5 of TBEC-2018. This adherence to Turkish codes is essential to ensure the building's compliance with safety regulations.

As the first effective modes of the structure have periods exceeding 70% (Table 10), performing a fixed first mode nonlinear pushover analysis is permissible according to TBEC-2018.

For this analysis, the following key parameters are established:

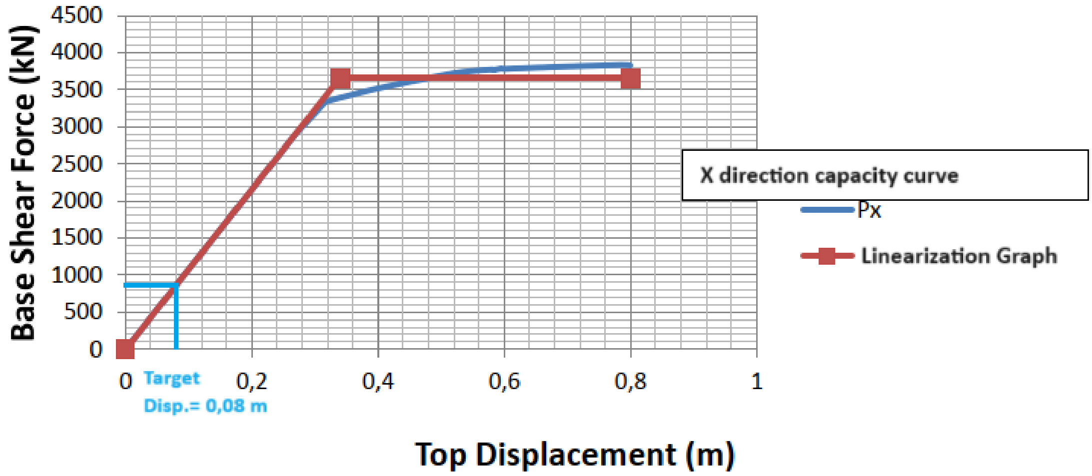
Seismic mass of the structure: 39.680,23 kN
Spectrum reduction factors:
X-direction: 0,79 (Table 9)
Y-direction: 0,67 (Table 9)

The original earthquake spectrum will be reduced by these factors in both the X and Y directions during the pushover analysis to account for the additional damping provided by the friction dampers. This will ensure a more accurate assessment of the building's seismic performance in its retrofitted state.

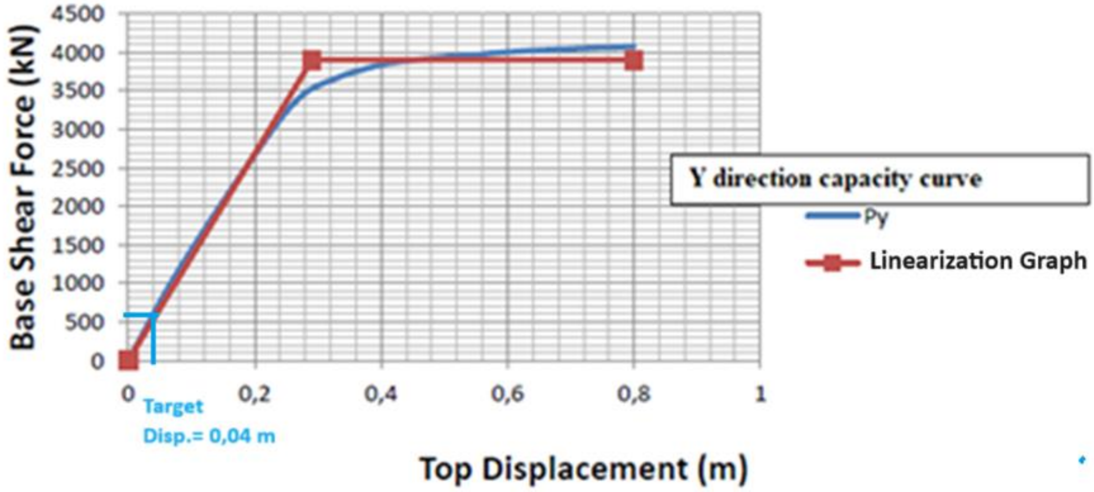
Table 10. Mass participation ratios (non-linear model)

Case	Mode	Period (sec)	UX	UY
Modal	1	**2,407**	**0,8811**	2,01E-05
Modal	2	**1,591**	0,0012	**0,9056**
Modal	3	1,394	0,0156	0,0597
Modal	4	1,063	0,0017	0,0064
Modal	5	0,761	0,0043	0,0085
Modal	6	0,65	0,0003	0,004
Modal	7	0,559	0,0002	0,0002
Modal	8	0,509	0,0462	0,0003
Modal	9	0,491	0,364	3,12E-05
Modal	10	0,409	0,001	0,001
Modal	11	0,401	2,59E-05	2,29E-06
Modal	12	0,388	0,0006	0,0002

A nonlinear fixed first mode pushover analysis was performed for both the X and Y directions of the retrofitted structure to determine the target displacement (Figure 21). This analysis is crucial for evaluating the building's global seismic performance and identifying potential weaknesses. Bilinear plastic springs used to capture the nonlinear behavior of the dampers.



a)



b)

Figure 21. a) X direction push over capacity curve b) Y direction push over capacity curve

Target displacements are determined and presented in Table 11 and inserted in Figure 21. The retrofitted structure is pushed until target displacement using fixed first mode pushover analysis. Demands are obtained at this point, and structure performance is checked.

Table 11. Target displacement calculation of retrofitted structure

Tx (s)	Sae(T)/g	Sae(T)	Sde(m)	ay1	Ry1	CR1	Sdi1	Target Disp. (m) UxN1
2,416	0,055588	0,545316	0,080627	2,817628	0,193537	1	0,080627	0,080627318

Ty (s)	Sae(T)/g	Sae(T)	Sde(m)	ay1	Ry1	CR1	Sdi1	Target Disp. (m) UyN1
1,586	0,0686	0,672968	0,042879	1,347561	0,499397	1	0,042879	0,042878674

In addition to fixed first mode pushover analysis, two time history analyses are performed in order to see the behavior of nonlinear dampers under earthquake (Figure 22).

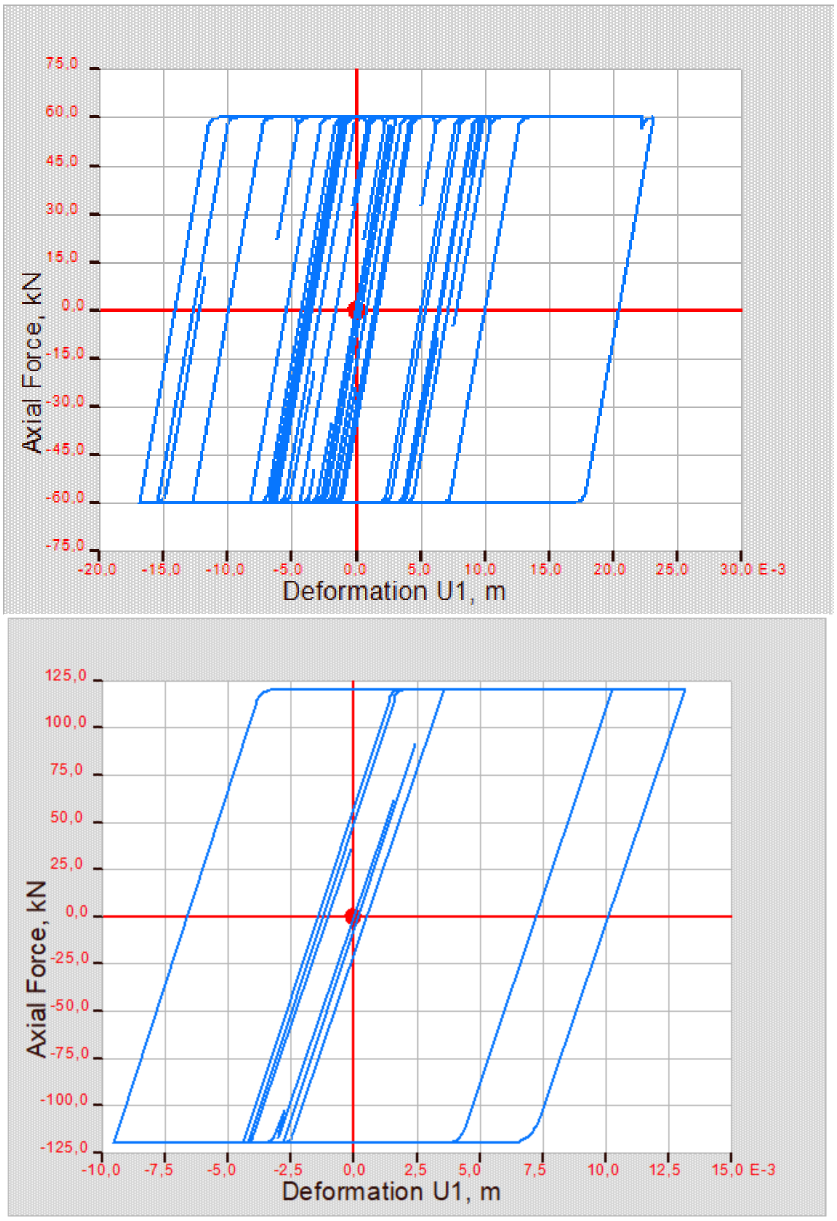


Figure 22. Hysteretic behavior of nonlinear hinges (dampers) 60 and 120 kN under time history analysis

4.2.4 Assessment Analysis Results for Retrofitted Building

The retrofitted structure is re-evaluated using nonlinear fixed first mode pushover analysis up to the target displacement determined in the previous section. re-evaluation considered both DD-1 and DD-2 earthquake levels. The results indicate that all plastic rotations satisfied the SH criteria, confirming sufficient ductility within the primary structural elements for both of the earthquake level (Table 12 and Table 13).

Table 12. Column damage levels and shear check of retrofitted building under DD-1 earthquake level

Column Damage Levels					
Storey	Limited Damage	Controlled Damage	Collapse Prevention	Collapse	Total
2	46	0	0	0	46
1	46	0	0	0	46

Column Shear Check			
Storey	Ductile	Brittle	Total
2	46	0	46
1	46	0	46

Table 13: Beam Damage Levels and Shear Check of Retrofitted Building.

Beam Damage Levels					
Storey	Limited Damage	Controlled Damage	Collapse Prevention	Collapse	Total
2	6	0	0	0	6

Beam Shear Check			
Storey	Ductile	Brittle	Total
2	6	0	6

Story drift checks are carried out for the retrofitted building under DD2 earthquake level;
X direction displacement: 129 mm, storey drift =129/13200 = 0,0098 Limited Damage
Y direction displacement: 72 mm, storey drift =72/13200 = 0,0055 Limited Damage

When drifts of existing and retrofitted structure are compared, it is seen that roof displacement is decreased from 186 mm to 129 mm for X direction (31% decreases) and from 210 mm to 72 mm for Y direction (66% decreases). That means that considerable decrease in non-structural member damages.

Additionally, pin connections of girder-beam and beam-column (Figure 23) are checked for DD-1 earthquake level for overturning and pin failure. They are both determined as safe for retrofitted buildings.

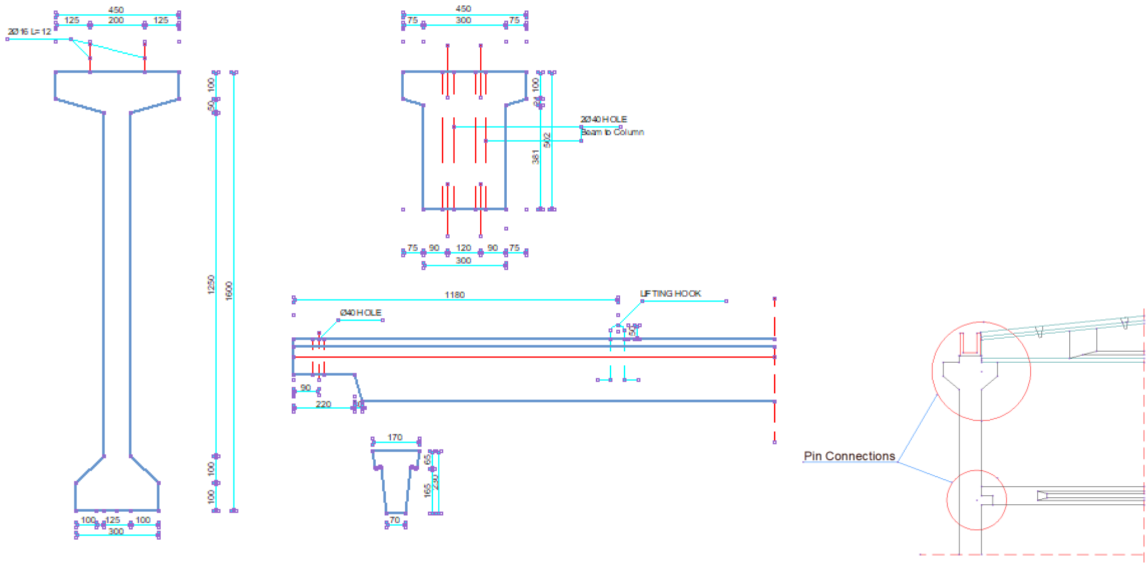


Figure 23. Pin connection of girder to beam and beam to column

Foundation Stress Check: Stresses under the foundations were analyzed and found to be less than 400 kPa, which satisfies the soil-bearing capacity. This confirms that the additional loads introduced by the retrofitting measures and seismic loads do not exceed the soil's capacity, ensuring the stability of the foundation system.

The effectiveness of the dampers is checked using DUZCE earthquake scaled for DD-2 level. Displacements are seen as significantly decreased for X and Y directions (Figure 24, Figure 25).

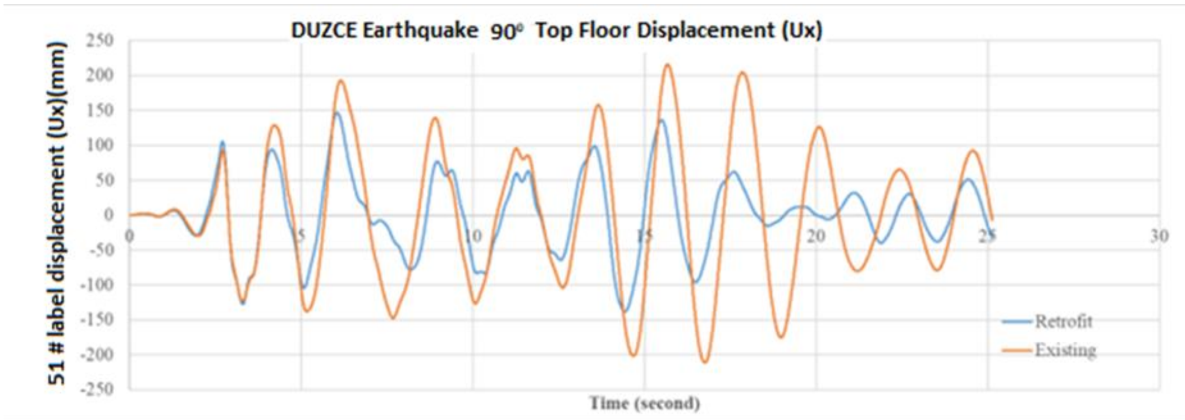


Figure 24. X direction top floor displacement for DD-2 scaled Duzce earthquake

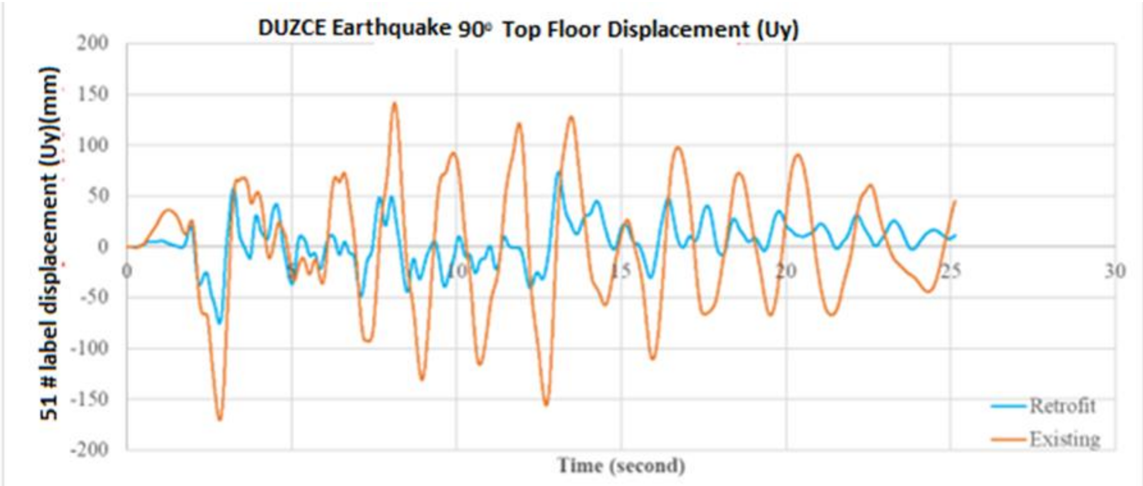


Figure 25. Y Direction top floor displacement for DD-2 scaled Duzce earthquake

Another important mutual problem of B1 and B2 blocks were pounding each other during the earthquake due to insufficient dilatation space between two adjacent blocks (Figure 26).

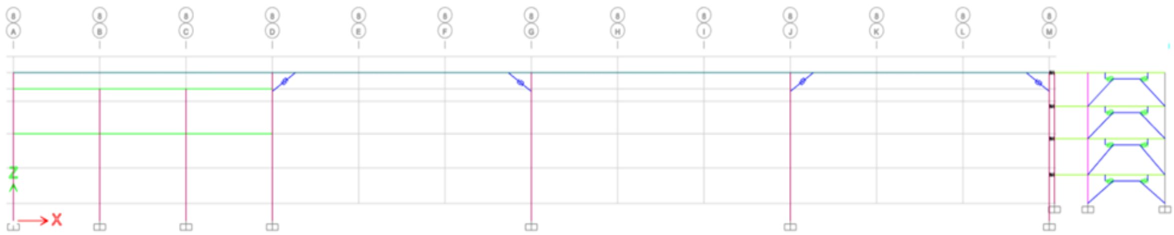


Figure 26. B1 and B2 blocks adjacent to each other

X-direction displacement of Block B1 decreased to 129 mm after retrofitting with dampers, but it still exceeds the existing space of 50 mm between the two blocks. This discrepancy suggests a potential for pounding to occur during seismic events. To further investigate this concern, a time history analysis was performed. For this purpose, inelastic spring HERTZ modeling is preferred and gap members are modeled between two adjacent blocks as seen in Figure 27.

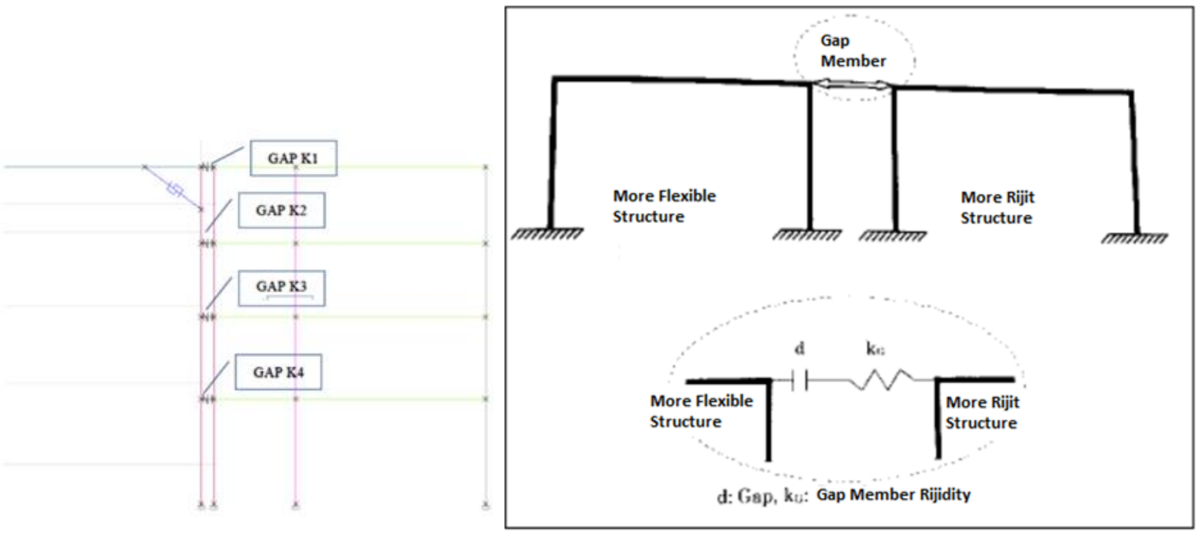


Figure 27. Gap member modeling in Etabs.

Pounding force – Displacement relationship of the Hertz modeling is given as:

$$f = \begin{cases} K(d - open) & if\ d - open < 0 \\ 0 & otherwise \end{cases} \tag{1}$$

After time history analysis, pounding forces obtained as a function of time and force can be seen in Figure 28.

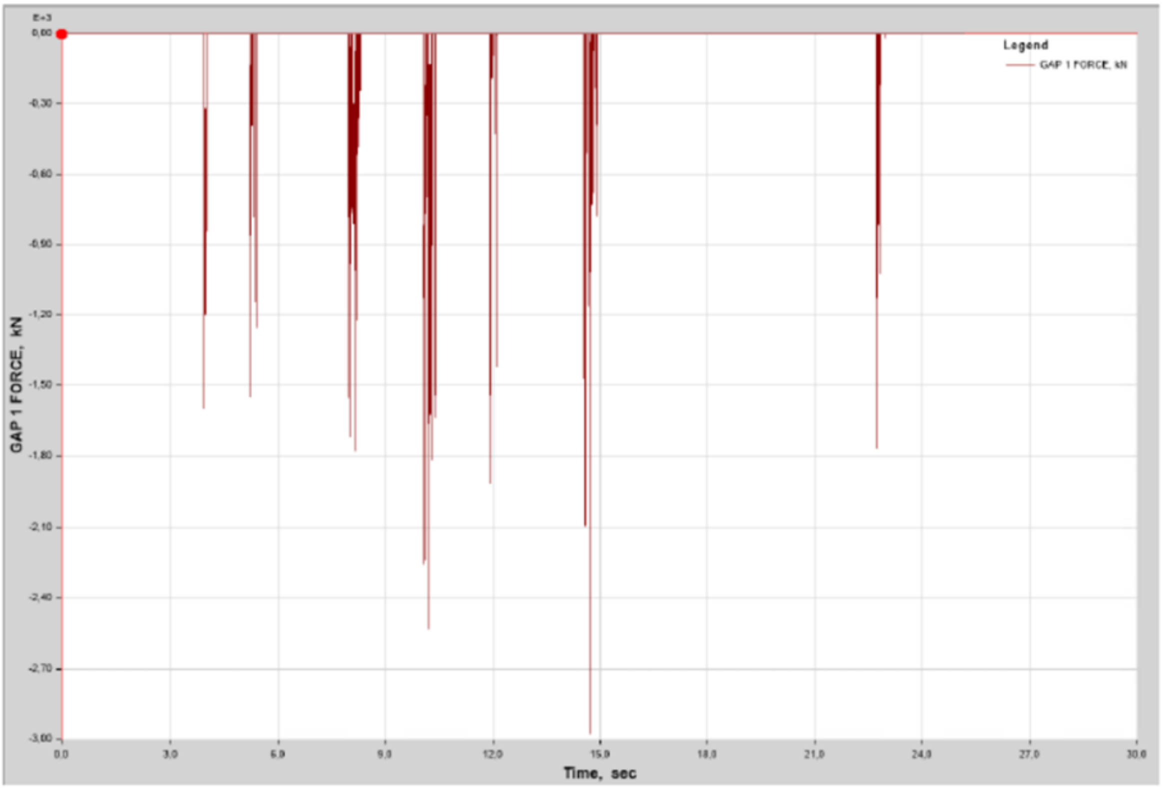


Figure 28. Pounding forces obtained in time history analysis

Demand-capacity ratios for the columns under pounding forces were checked and found to be acceptable, with a maximum value of 1.5 (Figure 29). This indicates that the columns have sufficient capacity to withstand the pounding forces without exceeding their yield strength. This represents a significant improvement compared to the pre-retrofitting state, where the demand-capacity ratios due to pounding forces were significantly higher, reaching 4-5. This confirms that the increased damping provided by the friction dampers effectively addressed the pounding issue, ensuring the safety and stability of both blocks during seismic events.

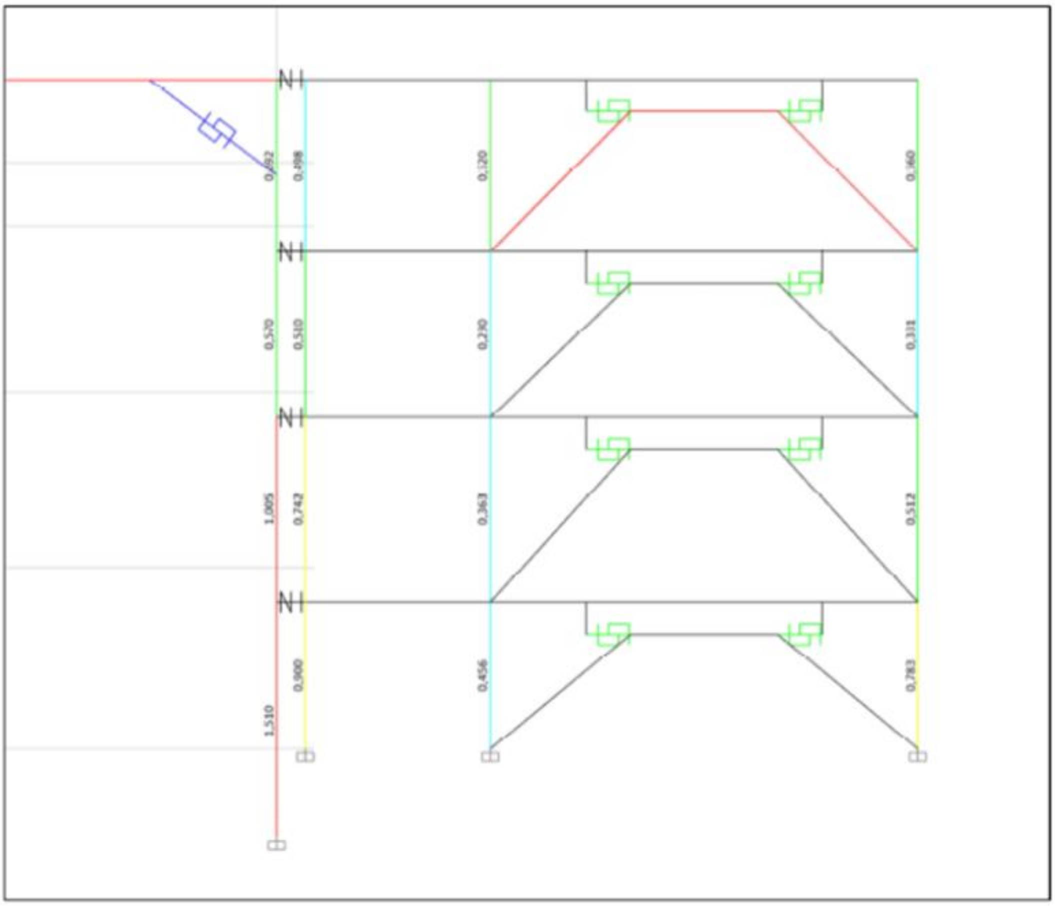


Figure 29. Demand capacity ratios of two blocks under pounding forces

5. Results

Original Performance:
Controlled Damage Level under DD-2 earthquake level (as per TBEC-2018).
Brittle behavior observed in some beams.
High story drifts causing potential damage to non-structural members.
Pounding problem identified between adjacent blocks.

Retrofitted Performance:
Upgraded to Limited Damage Level under DD-2 earthquake level.
All columns and beams exhibit ductile behavior.
Significant reductions in roof displacement:
X-direction: 31% decrease (186 mm to 129 mm).
Y-direction: 66% decrease (210 mm to 72 mm).
Pounding issue between blocks solved by increased damping.
Pin connections for girder-beam and beam-column deemed safe for DD-1 earthquake level.
Retrofit installation completed without disrupting building operations.

Key Benefits of Retrofitting:
Improved seismic performance and safety.
Reduced risk of damage to both structural and non-structural elements.
Enhanced occupant comfort and building functionality.
Mitigation of pounding problem between adjacent blocks.
Minimal disruption to building operations during retrofitting.

6. Conclusions

The implemented retrofitting measures, particularly the friction dampers, demonstrably improved the seismic performance of the precast building. By successfully addressing the identified deficiencies, the retrofitting project ensured the building's compliance with the desired Limited Damage Level and enhanced its resilience against earthquake loads. This case study highlights the effectiveness of friction dampers as a viable solution for improving the seismic performance of existing prefabricated structures.

Retrofitting installation works are performed without stopping the functionality of the Building with some safety precautions and slight separations (Figure 30).

Figure 30. Installation photos

Author Contributions

Suat Yıldırım and Yüksel İlkay Tonguç both contributed to the design and implementation of the research, to the analysis and the evaluation of the results.

Conflict of Interest

Authors declare no conflict of interest.

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