©2022 Published in 6th International Symposium on Natural Hazards and Disaster Management
21-23 October 2022 (ISHAD2022 Bursa - Turkey)
https://doi.org/10.33793/acperpro.XXXX

Retrofitting of a Precast Reinforced Concrete Industrial Building Using Friction Dampers Without Stopping Functionality

*[1]Suat Yıldırım and [2]Yüksel İlkay Tonguç
[1] Str. Eng. M.Sc. Promer Consultancy and Engineering Inc. Ataşehir/İstanbul/Turkiye
syildirim@promerengineering.com.tr
[2] Str. Eng. M.Sc. Promer Consultancy and Engineering Inc. Ümitköy/Ankara/Turkiye ,
ytonguc@promerengineering.com.tr

Abstract:

This paper presents a seismic retrofit of an industrial-type precast reinforced concrete building structure using friction dampers. This project consisted of retrofitting two precast reinforced concrete buildings, of which one of them will be presented here.

Precast concrete is one of the most preferred methods of construction for industrial structures 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(TBDY 2018)" are not sufficiently engineered and are expected to have poor performance when exposed to a major seismic event.

Because stopping the production cost can be tolerated by the owners, a retrofitting technique that can be assembled during 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 [10] is employed for the damper design and performance evaluation of the building. Before the damper study, some instability and weak connection problems are solved by some traditional measures of strengthening. 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 analysis are conducted after linear prestudies. Finally, a nonlinear time-history analyses is performed for confirmation of the results. , It is shown that the proposed retrofit scheme satisfies the desired performance goals for both DBE and MCE events. Overall, it is considered that proposed retrofit scheme with dampers provides the optimal solution for the stakeholders of the project from a performance, design, constructability, and economical point of view. Some application photos will be presented and methods are explained in detail in the paper.

The friction damper behavior is based on the rotational friction hinge concept. The dampers are designed to provide passive energy dissipation and protect buildings, from structural and nonstructural damage during moderate and severe earthquakes.

Intensive Testing of dampers of this type with different slip capacity has been carried out at the technical Univ. of Denmark, USA and in Japan over the last two decades..

*Corresponding author: Address: Promer Consultancy Engineering Inc. Ataşehir/Istanbul. E-mail address: syildirim@promerengineering.com.tr, Phone: +905335921369

Key words: Retrofitting; friction dampers; precast structures; earthquake protection; passive control

1. Introduction

Earthquakes have claimed the highest number of lives and caused the greatest economic loss, with 76 earthquakes since 1900 resulting in approximately 90,000 fatalities, a total affected population of 7 million, and direct losses exceeding US$25 billion5 . About half the casualties were due to two earthquakes on the North Anatolian Fault in 1939 and 1999. In the 1999 Marmara earthquakes, which affected 10 cities 6 in the Marmara Region of Turkey where almost 35 percent of the Turkey's GNP was produced, the death toll was over 18,000 with a direct economic impact estimated at US$5 billion (2.5 percent of GNP).

Prefabricated buildings especially constructed before TBDY 2018 [5] have some majör vulnerable weaknesses about earthquake resistance. These weaknesses are evaluated and published by many investigators [1], [2].

Prefabricated structures are oftenly preferred for industrial buildings in Turkey. In some researches is it determined that 75-85 % of industrial buildings are prefabricated in Industrial regions of Turkey [3-4]. It is also determined in a research that 80 % of prefabricated buildings are damaged partially or totally during Marmara earthquake 1999 in Adapazarı Organized Industrial Zone [1]. Because industrial buildings work non-stop 7/24 usually, one of the main difficulties in retrofitting is stopping function of the building for construction. Cost of stopping the function of the building may be more than retrofitting itself. Retrofitting of a reinforced concrete prefabricated building using friction type of energy dissipating devices (FDD) are presented in this study. Retrofitting of the building is applied without stopping buildings's function.

Retrofitting design has 3 main stages. Data collection, material and soil tests, assesment analysis and retrofitting analysis and design of the building. Firstly all geometrical data is collected from the building and architectural and structural roleve drawings are prepared and material tests in order to determine concrete quality is achieved. Existing concrete compressive strength is determined as 26,3 Mpa reinforced concrete prefabricated building as per TBDY 2018. Reinforcement is determined using some destructive and non-destructive (scanning) methods as StIII. Target performans of the building is determined as limited damage (minimum damage) performans level under DD-2 earthquake ([7]. Fixed single mode pushover nonlinear static analyis method via Etabs ver. 18 is used for assesment analysis of the building.

After exploring that existing performance of the building is not satisfying TBDY 2018 limited damage performance level (client's requested target), retrofitting studies are performed. Because FDD are preferred for retrofitting and TBDY 2018 does not contain design precedure for this type of retrofitting, ASCE 41-17 and ASCE-7 are used as standarts to be followed. Fixed single

*Corresponding author: Address: Faculty of Engineering, Department of Civil Engineering Sakarya University, 54187, Sakarya TURKEY. E-mail address: caglar@sakarya.edu.tr, Phone: +902642955752 Fax: +902642955601

nonlinear static pushover method is used for retrofitting analysis of damper applied building. Addidionally nonlinear dynamic time history analysis are performed for selected two earthquakes as pier reviewer's demand altough codes does not requires. Moreover some stability problems of the building are solved in addition to performance rehabilitation.

2. Building Information

2.1. General Information

Building is consist of two adjacent block that are B1 (precast concrete) and B2(monolitic). Both of them retrofitted using friction dampers but only B1 will be presented in this paper. Some of their properties are tabled in Table 1.

Table 1: General Building Properties

Storey #	B1 block 3 storey ; B2 block 4 storey
Concrete Compressive Strength:	B1 block 26,3 Mpa ; B2 block 15,4 Mpa
Area	B1 block 9481,4 m2 ; B2 block 1040,2 m2
Rebarr type	ST420
Foundation Type	Single footing

Reinforced concrete prefabricated building is used as wairhouse of a food production factory. Building is one storey in general but has administrative two storey mezzanine floors in one axis. Total building dimensions are 90m x 77m and 4 x 22,5 m axis in long direction while 10 x 7,7 m axis in short direction (Figure 1). Building height is 12.0 mt with two mezzanine administrative floors at levels of 5,5m and 8,2m in one axis (Figure 2)

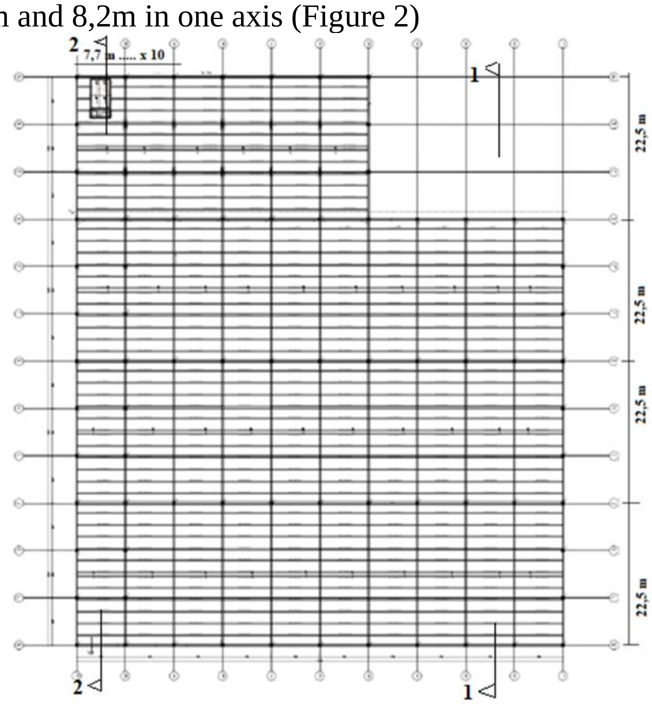


Figure 1: Plan View of The Building.

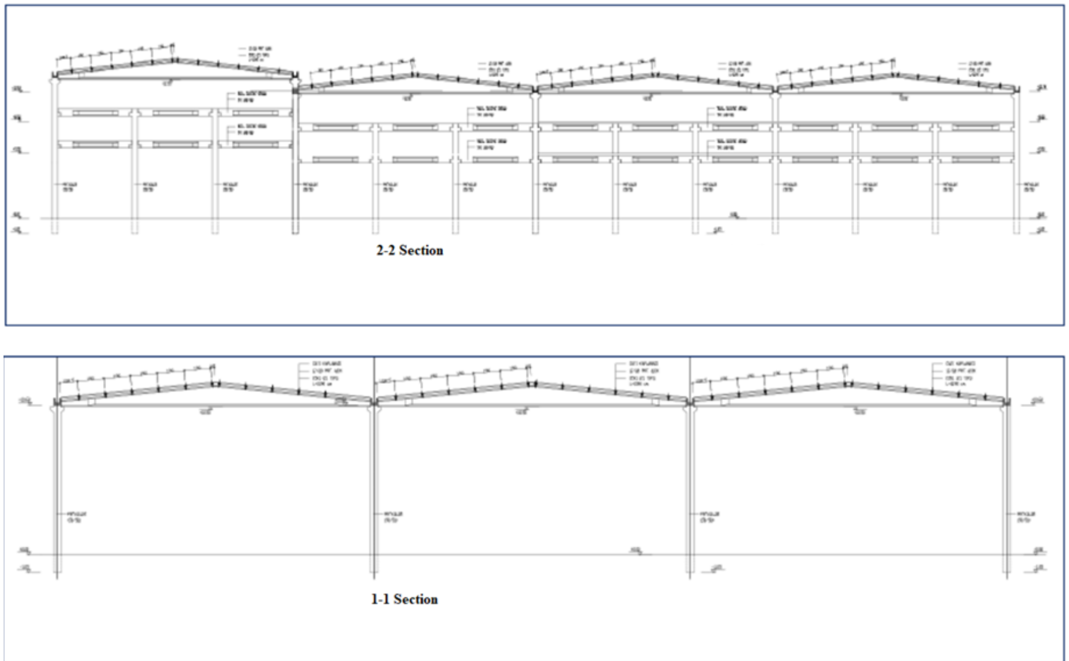
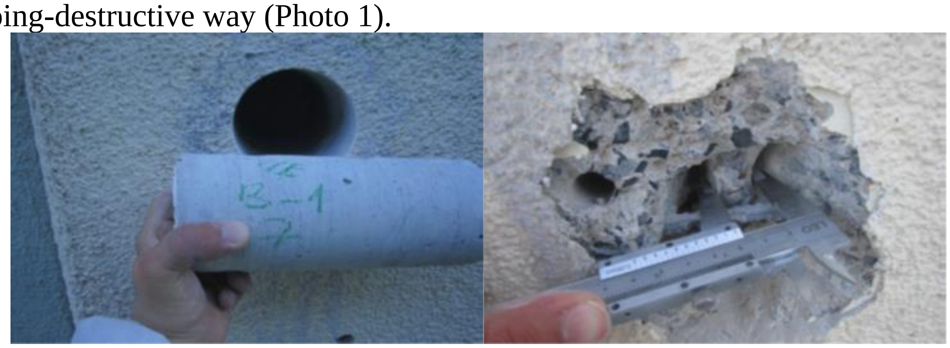


Figure 2: Section Views of The Building

Foundation system of the building is determined as single footing using projects provided by the owner and 2 foundation test pits at site.

2.1. Material Tests

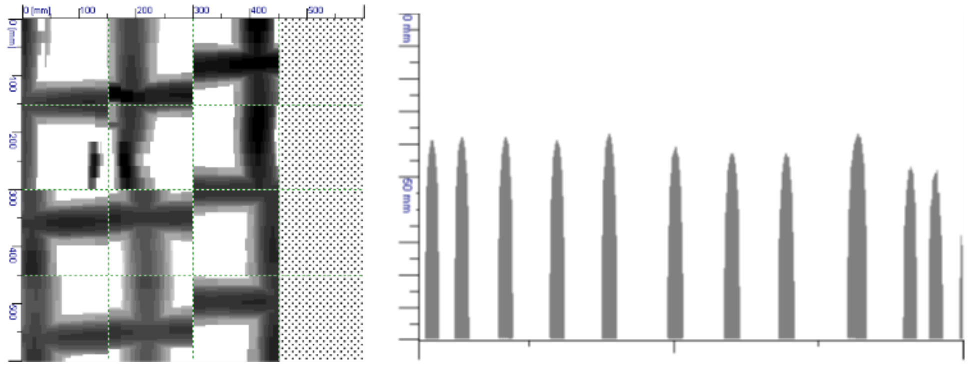
Construction drawings are provided for R/C prefabricated building from the client. But due to mechanical strength test couldn't be performed for rebars, limited knowledge level is used for knowledge level of data collection. That is why 0,75 capacity decrease is used for member capacities as per TBDY 2018. 17 pcs concrete core samples are taken from selected columns. Concrete compressive strength is calculated as per TBDY 2018 and determined as 26,3 Mpa. Reinforcement type, orientation, diameter and corrosion level is determined at 11 column members using stripping-destructive way (Photo 1).



a) b)

Photo 1: a)Concrete Core sampling, b) Destructive reinforcement detection

Additionally ferro-scan is used at 19 vertical members in order to determine reinforcement numbers and orientation.



a) b)

Figure 3: a) 60x60 scan view from a column b) Vertical scan view of a column starting from bottom (1,6m-1,8m length)

Determined reinforcement layout and details are shown in Figure 4 and Table 2.

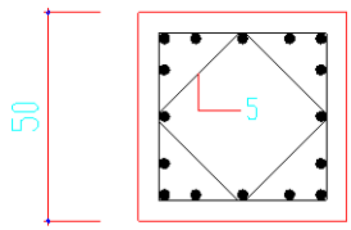


Figure 4: Determined reinforcement layout in a 50x50 column

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 (pcs/cm)	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

No corrossion and foundation settlement is determined in the structure.

Foundations plans are obtained from the client and 2 foundation test pits are opened for confirmation. Foundations are determined as single footings with 360cmx360cm with 70 cm height.

3. Modelling and Assessment Analysis

3.1 Assessment Conditions

3.1.1 Structural performance Criteria

Analysis criterias for the buildings including energy dissipation devices are not included TBDY 2018. That is why earthquake levels, and performance damage levels are provided from TBDY 2018 Whereas, The damping analysis will be applied ASCE 41 and ASCE 7.

In TBDY 2018, three primary structural performance levels are defined:

- Limited Damage Level (SH)
- Controlled Damage Level (KH)

- Collapse Prevention (GÖ)

These performance levels relate to damage states for elements of lateral force resisting systems. In the design process four levels of earthquake ground motion are considered;

- DD1: with 2% probability of exceedance in 50 years
- DD2: with 10% probability of exceedance in 50 years
- DD3: with 50% probability of exceedance in 50 years
- DD4: with 68% probability of exceedance in 50 years

Because the building is a production facility, minimum performance level is controlled damage level (KH) under DD2 earthquake level as per TBDY 2018. But this is minimum for requirement and involves repairable but considerable damage to the building. That is why client request Limited Damage Level (SH) as much as possible.
TBDY 2018 describe the expected level of damage to the building under these performance levels. The KK (Operational Usage) performance is described as no damage or negligible damage to both structural and nonstructural systems of building.
The SH (Limited Damage) performance is described as an acceptable probability of "a damage state in which only limited structural damage has occurred. The basic vertical and lateral-force resisting systems of the building retain nearly all their pre-earthquake strength and stiffness." ASCE 7 further clarifies the state of nonstructural components at IO as "Minor cracking of facades, partitions, and ceilings… Equipment and contents are generally secure but may not operate due to mechanical failure or lack of utilities."

3.1.2 Building Assessment Parameters

Parameters used in assesment are tabled in Table 3. Earthquake spectrum parameters are obtained from interactive earthquake map of AFAD (Turkish disaster and emergency department) using soil properties for local coordinates.

Table 3: Building Asssesment and Loading Parameters

Building Assessment Parameters					
Building Importance Factor (I)	1	Cracked Section Rijidity Factor		0,4	
Earthquake Load Reduction Factor (R)	1	Roof LL	1,5 kN/m2	Coridor LL	3,5 kN/m2
Soil Type	ZB	Mezzanine floor LL	2,0 kN/m2	Snow Load	0,75 kN/m2
Soil Bearing Capacity (kPa)	400	Brick Wall weight	3,2 kN/m2		
Wind load is calculated as per TS EN 1991-1-3 (Eurocode1)					
Earthquake Load Calculation Parameters					
DD1 Spectrum Parameters (AFAD)	$S_S = 1.326$ $S_1 = 0.367$ PGA=0.540 PGV=33.247	$S_{DS} = S_S*F_S =$ $1.326 \times 0.900 = 1.193$		$S_{D1} = S_1 F_1 =$ $0.367 \times 0.800 = 0.294$	
DD2 Spectrum Parameters (AFAD)	$S_S = 0.737$ $S_1 = 0.213$ PGA=0.308 PGV=19.186	$S_{DS} = S_S*F_S =$ $0.737 \times 0.900 = 0.663$		$S_{D1} = S_1 F_1 =$ $0.213 \times 0.800 = 0.170$	
DD3 Spectrum Paramekers (AFAD)	$S_S = 0.286$ $S_1 = 0.087$ PGA=0.122 PGV=8.087	$S_{DS} = S_S*F_S =$ $0.286 \times 0.900 = 0.257$		$S_{D1} = S_1 F_1 =$ $0.087 \times 0.800 = 0.070$	

S_S: Short period spectral acceleration factor (unitless)	S_{DS} : Short period design spectral factor (unitless)
S_1: 1 sec. Period spectral acceleration factor (unitless)	S_{D1} : 1 sec. period design spectral factor (unitless)
PGA: Biggest ground acceleration (g)	F_S : Short period local ground effect factor
PGV: Biggest ground velocity (cm/sn)	F_1 : 1 sec. period local ground effect factor

3.2 Modelling

3-D modelling is done using ETABS software (figure 8). Live and dead loads on the roof are calculated and distributed on the beams. Live and dead loads on slabs are defined on the slabs for mezzanine floors.

Confined and unconfined Mander model is used for nonlinear material model of concrete and Kinematic model is used for nonlinear behaviour model of steel.

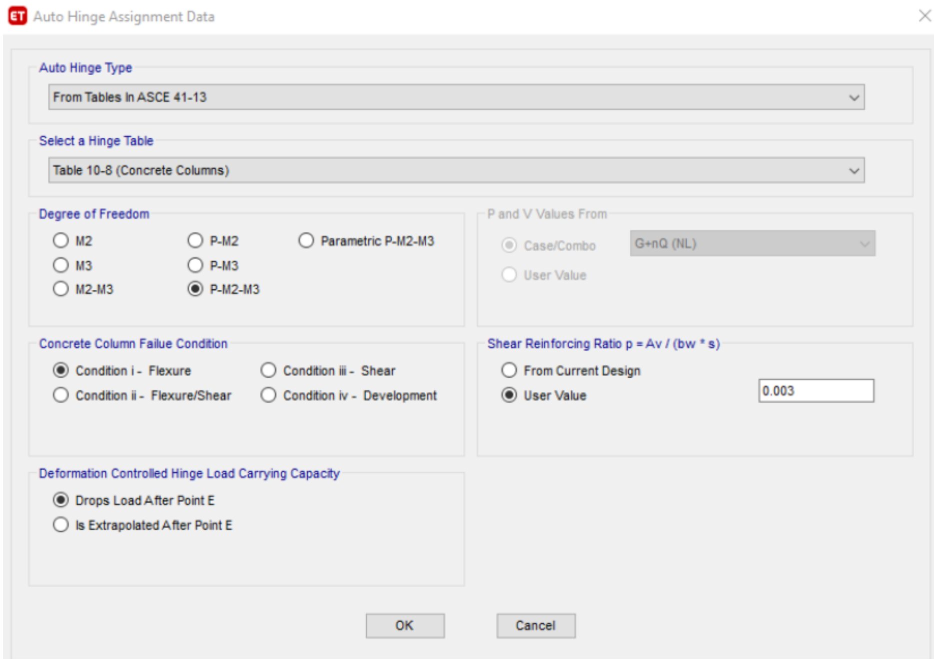


Figure 5: P-M2-M3 Auto Hinge Modelling for Columns

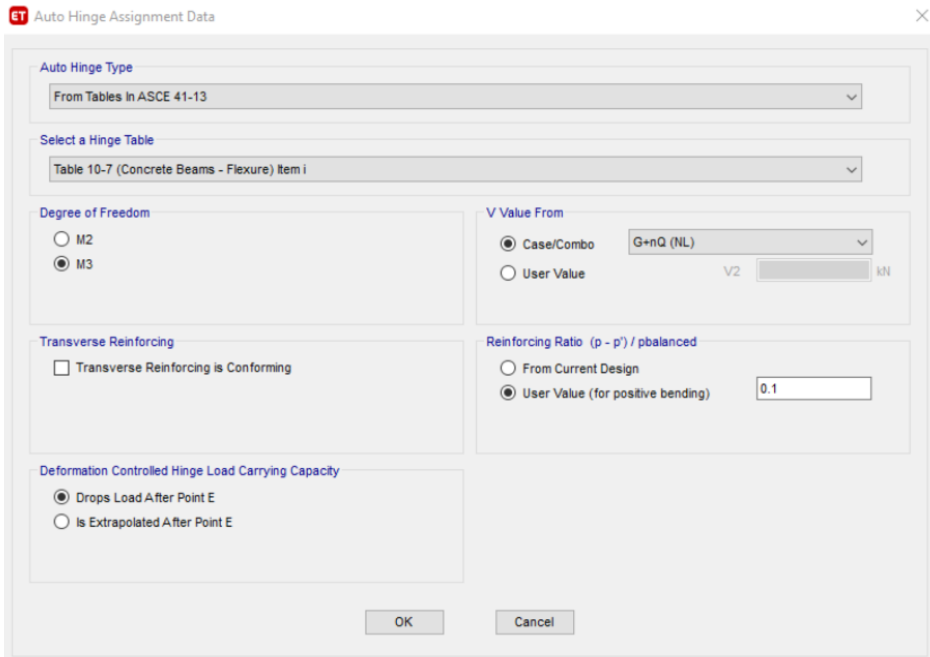
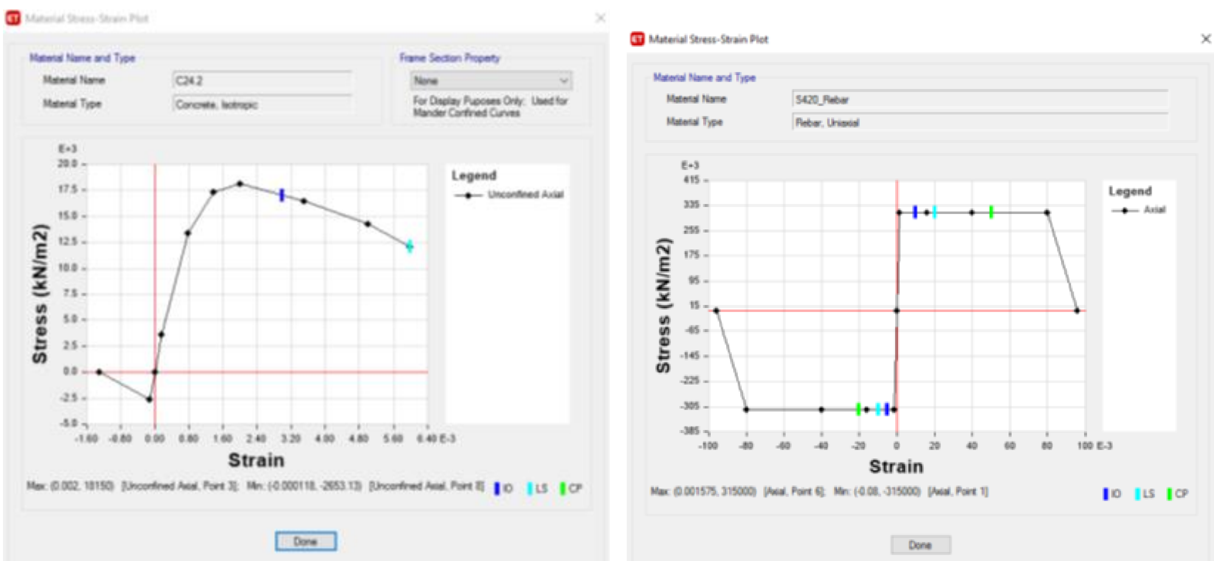


Figure 6: M3 Auto Hinge Modelling for Beams



a) b)

Figure 7: Nonlinear material model of a):Concrete b): Steel

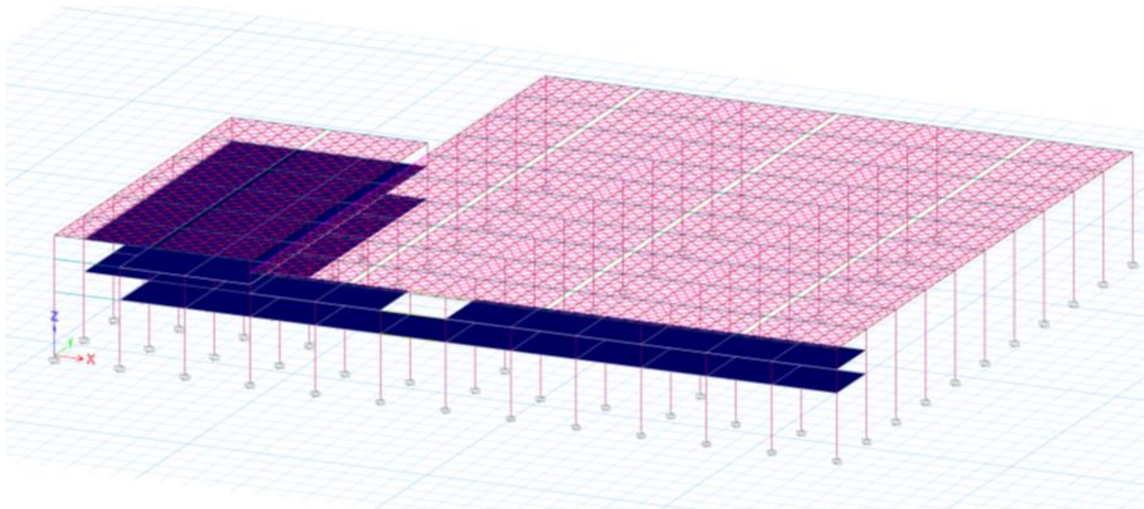


Figure 8: 3-D Etabs model perspective view.

Columns and beams are modelled as frame members in ETABS. Nonlinear behaviour is modelled as M3 nonlinear hinges for beams and P-M2-M3 nonlinear hinges for columns (Figure 9).

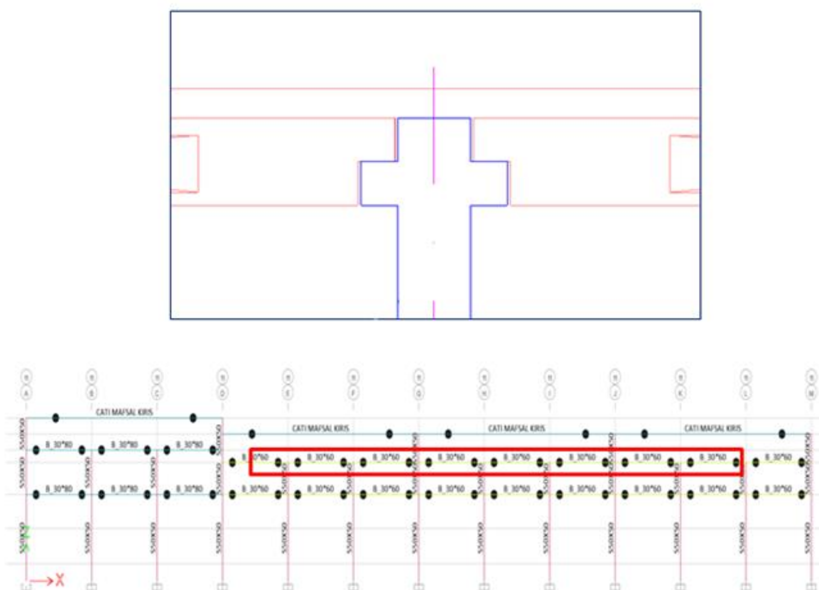


Figure 9: Nonlinear hinge definition for columns and beams

Modelling of roof rigidity of the roof panels are done according to definition in TBDY 2018 [5] attachment 8B (Figure 10). Rigidity is defined as cross members between roof girder members at panel level.
Rigidity of cross members is calculated as (EA)e≈ 400000 kN as per TBDY 2018.

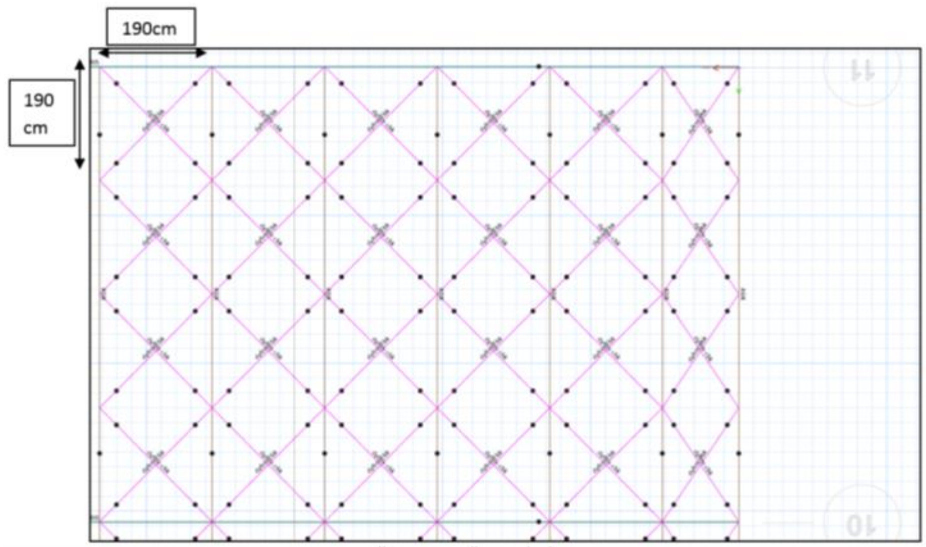


Figure 10: Roof Rigidity Modelling as per TBDY 2018 Attachment 8B

3.3. Assessment Analysis of Existing Building
3.3.1. Linear Dynamic Analysis

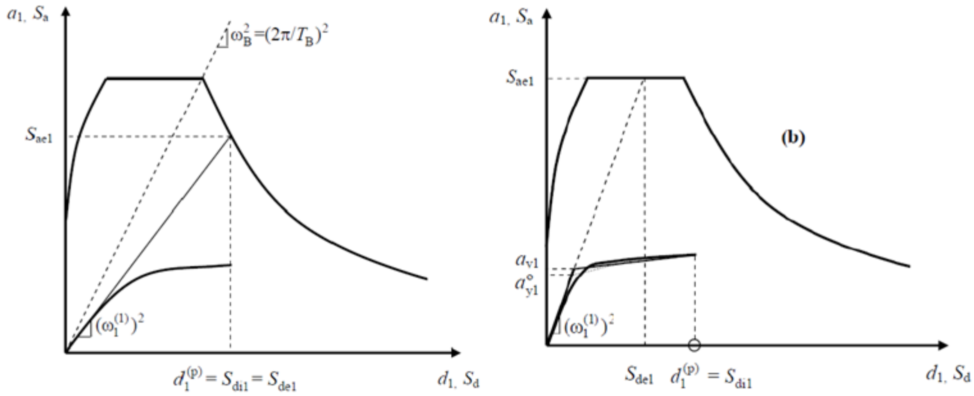
A Linear dynamic analysis (modal analysis) is performed with existing conditions. Modal Participating mass ratios are calculated (Table 4). First mode is determined as X direction with 3.686 sec. period 0,891 modal participation and Second mode is as Y direction with 3,33 sec. period and 0,7789 modal participation. Because effective modal mass participations are more than 0,70 and storey torsion is under limit (1,4), nonlinear fixed first mode pushover analysis will be used as per TBDY 2018.

Table 4: Modal Participating Mass Ratios

TABLE: Modal Participating Mass Ratios				
Case	Mode	Period	UX	UY
		sec		
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 described in TBDY 2018 is followed (Figure 11). For this purpose a pushover is performed until top floor displacement is reached until 4% of height of building just for purpose of getting a capacity curve.



a) b)

Figure 11: Target Displacement Calculation as per TBDY 2018 a) T1 > TB b) T1 < TB

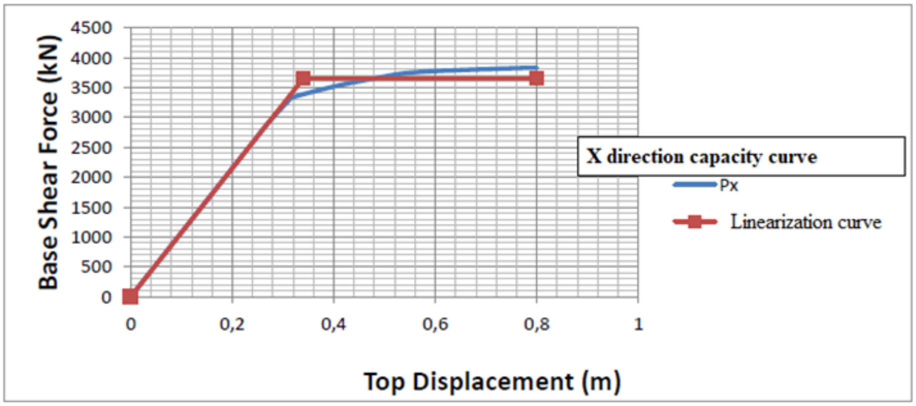


Figure 12: X Direction Capacity Curve

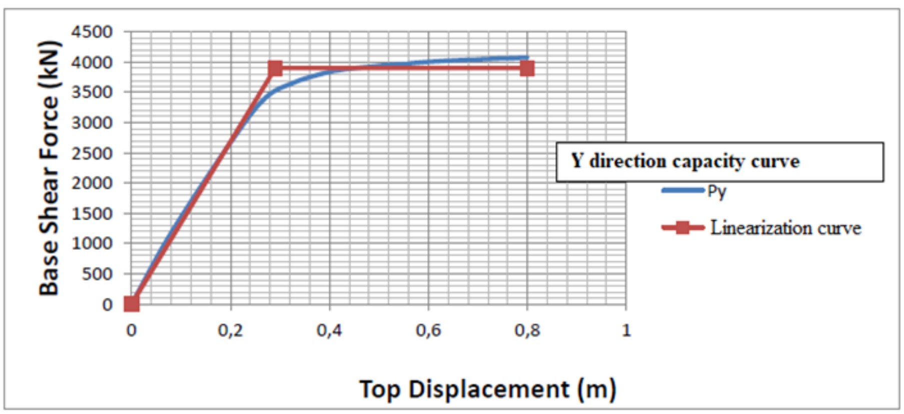


Figure 13: Y Direction Capacity Curve

Target displament is calculated as described in TBDY 2018 (Figure 11). Calculated target displacement and related parameters are given in Table 5.

Table 5: Target Displacement Calculation for X and Y Direction

								Target displacement (m)
Tx	Sae(T)/g	Sae(T)	Sde (m)	ay1	Ry1	CR1	Sdi1	uxN1
3,686	0,04612	0,45244	0,15571	0,80325	0,56326	1	0,15571	0,155708931
Ty	Sae(T)/g	Sae(T)	Sde (m)	ay1	Ry1	CR1	Sdi1	uyN1
3,333	0,05101	0,50036	0,1408	0,85827	0,58299	1	0,1408	0,140797034

A new nonlinear fixed first mode pushover analysis is run until calculated target displacement in order to get performance of the building.

3.3.3. Performance Analysis Results for Existing Building

Assesment results calculated at target displacement of Pushover analysis are presented in Table 6. and Table 7. Column rotations were under limited damage level for DD2 earthquake level as desired. But beam rotations at mezzanine level were calculated higher than limited damage level. Additionally those beams are determined as brittle.

Table 6: Column Rotation and Shear Assessment Results at Target Displacement for DD2 earthquake level.

Column Damage Levels					
Storey	Limited Damage	Kontrolled 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 earthquakel level.

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

Beam Shear Check			
Storey	Ductile	Brittle	Total
2	0	6	6

Soil bearing capacity were calculated as 400 kPa. All soil stresses under foundations are calculated under bearing capacity for static vertical and dynamic lateral earthquake loads. . That means no foundation problem is determined for the building.

Storey torsion, soft storey, weak storey irregularities are checked and determined as satisfying TBDY 2018 using linear modal analysis under DD-2 earthquake level. Storey drift check is performed and results are presented in Table 8. Storey drifts determined as not satisfying limited damage level.

Table 8: Storey Drift Check under DD-2 Earthquake Level

Storey Drift Check (Fema 356)				
		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	13200 mm			

There has been determined some stability problems in the building structure. The building has a pin connected frame system in one direction (Figure 2), but no frame system on other direction (Figure 1). Only girders are connecting the frames in this direction. This results instability for earthquake loads in this direction.

As a result of assesment analysis building columns satisfies rotation and shear criterias for limited damage level under DD-2 earthquake level. But beams does not satisfying desired limits. Additionally building has storey drift problem which is over limited damage level. Because building height 12.0 mt (Figure 2) more displacement may result more damage for non structural walls and non-structural members. This may cause long repairement time and loss of money and even may cause loss of lifes. Moreover building has some stability problems to be solved.
Building needs retrofitting and due to the client does not prefer stopping production for retrofitting purpoes for a long time, classical retrofitting techniques (adding R/C shear wall, column jacketing etc.) can not be applied. Retrofitting using friction type dampers are applied as described in following section.

4. Retrofitting of The Building

4.1. Analysis Methods

Because buildings having damping devices are not included in TBDY 2018, ASCE 7-16 and ASCE 41-16 are decided to be used as main code together with TBDY 2018 to be followed. But results and stability conditions should satisfy TBDY 2018 as well. Earthquake forces will be obtained as per TBDY 2018 for sure. Main objectives were;

- Steel frame system and connections used in damping system are designed to stay linear under DD-2 earthquake level (ASCE 7-16 Section 18.2.1.2).
- Capacity of steel frame system and connections used in damping system are designed to be at least 20 % over the member demands calculated under DD-1 earthquake level (ASCE 7-16 Section 18.2.1.2)
- Displacement capacity of dampers will be at least 30 % higher than maksimum displacement of the dampers under DD-1 earthquake level (ASCE 7-16 Section 18.2.4.6)

Two level of earthquake were used DD-2 and DD-1 with the following analysis methods.

- Modelling, Analysis and assessments are performed as per TBDY 2018 section 15 and section 5 under DD-2 earthpquake level. Fixed first mode nonlinear pushover analysis is performed because the structure behaviour satisfies related conditions in TBDY 2018 Section 5.6.2.2.
- Nonlinear time history analysis is the main analysis method under DD-1 earthquake level because of possibility of nonlinear behavior. Linear modal analysis are permitted if and only if the structure behaves linear (or almost linear). In this study retrofitted structure behaviour was determined almost linear but because of the nonlinear damping devices included in retrofitting, a time history study is performed with 2 earthquake records (Düzce and Erzincan) as proposed in the 5ICEES kongres papers 9386 and 10637 no [11].

4.2. Final Retrofitting Studies

4.2.1 Modelling

After many trial studies and discussion with the client the damper locations are fixed as shown in figure 14, figure 15 and figure 16.

44 pcs 60 kN damper is used in X direction and 52 pcs 120 kN damper is used in Y direction. Histeretic both linear and nonlinear behaviour of the dampers are represented in Etabs model using plastic wen spring modelling. Modelling was linear for equivalent viscous damping calculation but bilinear plastic for nonlinear pushover and nonlinear time history analysis. Loads, load combinations, nonlinear hinge properties are modelled just as described for existing performance assessment of the structure (See Sections 3.1 and 3.2). Dampers are placed in toggle type of steel braces (Figure 17).

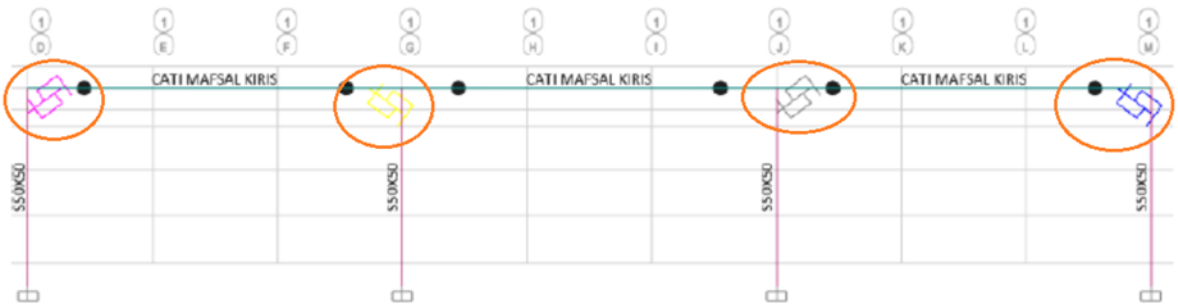


Figure 14: Damper Application in X Direction (Dampers marked with red circles are in diagonal position close to top of the columns and between column and beams).

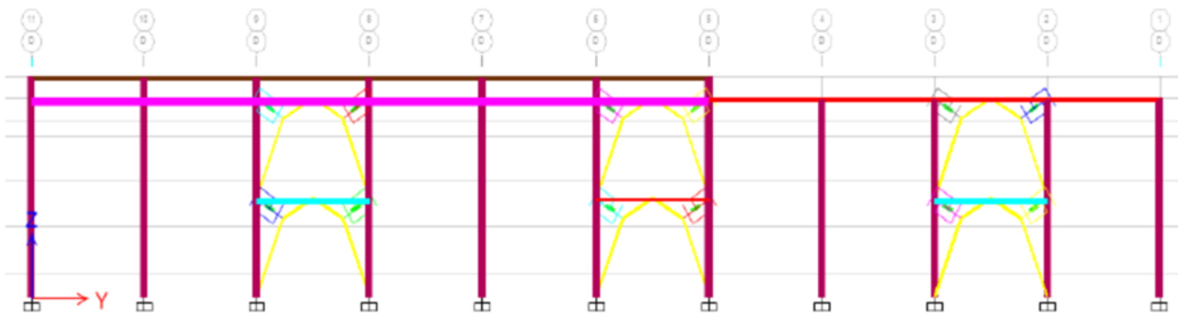


Figure 15: Damper Application in Y Direction (Dampers are in toggle braces)

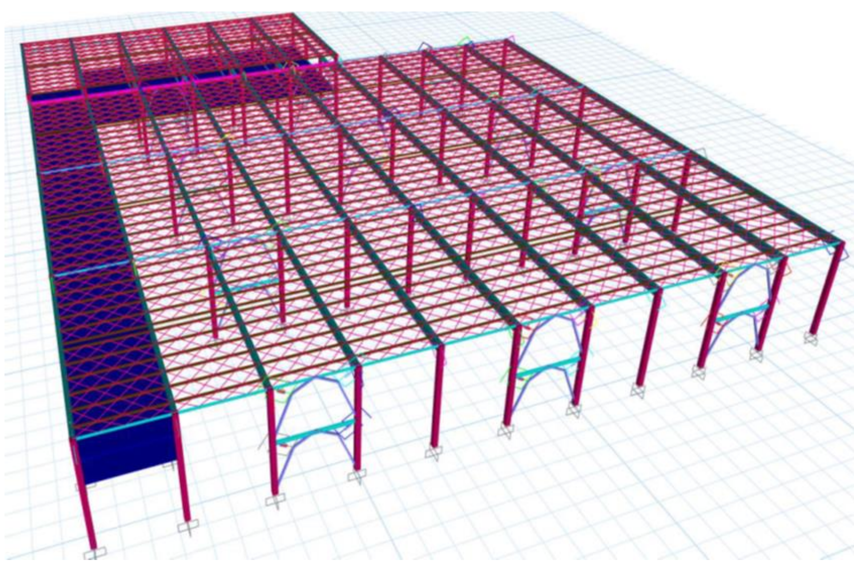


Figure 16: Etabs Model View of Damper Application.

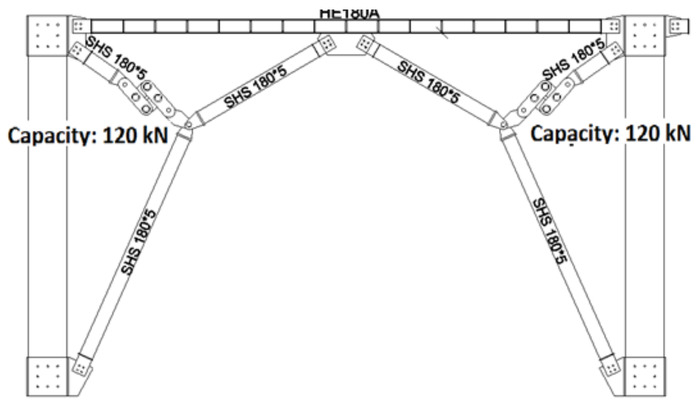


Figure 17: Damper location in Toggle Type of Bracing.

4.2.2 Damping Analysis

Equivalent Viscous damping calculations are performed using ASCE 7-16 section 18.7.3.2.2 and equations 18.7-47 and 48. Calculated parameters and are presented in Table 9. Additional damping added to the system with seicmic dampers are calculated as 6,55 % and 15,8 % for X and Y direction respectively. Because more dampers are used with higher capacities, more additional damping is obtained for Y direction as requested. Reduction factors are calculated as 0,79 and 0,67 for X and Y directions. That means 21% decrease in demands for X direction while 33% decrease in demands for 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 DD1 and DD2 earthquake level -linear model-

	Initial	Effective	
T_x Period (Sec.)	1,808	2,05	
T_y Period (Sec.)	0,952	1,398	
Equivalent Ductility -X	0,77784		
Equivalent Ductility -Y	0,46372		
Additional Damping -X	0,06554		
Additional Damping -Y	0,1582		
Under DD1 Earthquake Level			Under DD2 Earthquake Level
Reduction factor - X	0,79		0,77
Reduction factor - Y	0,67		0,60

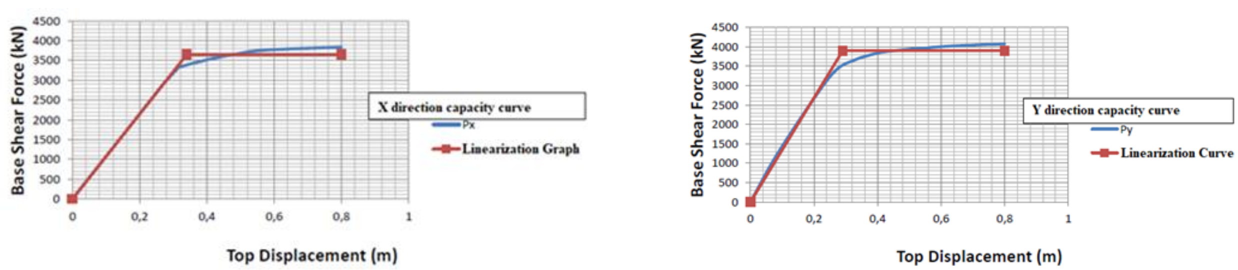
4.2.3 Assessment Analysis of Retrofitted Structure

Assesment analysis will be performed following section 5 of TBDY 2018. Because the structure has to meet Turkish codes in retrofitted condition. Because periods of the structure over 70% for first effective modes (Table 10), nonlinear pushover analysis are permitted as per TBDY 2018. Seismic mass of the structure is calculated as 39680,23 kN. Spectrum which will be used for pushover analysis are reduced by reduction factor presented in Table 9 for both X and Y directions in order to consider damping affect of damper.

Table 10: Mass Participation Ratios (non-linear model)

TABLE: Modal Participating Mass Ratios				
Case	Mode	Period	UX	UY
		sec		
Modal	1	2,407	0,8811	0,0000201
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	0,0364	0,0000312
Modal	10	0,409	0,001	0,001
Modal	11	0,401	0,00002587	0,000002288
Modal	12	0,388	0,0006	0,0002

A nonlinear fixed first mode pushover analysis are performed for X and Y direction respectively in order to determine target displacement of the retrofitted structure (Figure 18). Dampers are modelled as bilinear plastic springs in the modelled in order to represent nonlinear behaviour.



(a) (b)

Figure 18: a): X Direction Push Over Capacity Curve b): Y Direction Push Over Capacity Curve

Target displacements are determined as detailed in section 3.3.2 and presented in Table 10. 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 10: Target Displacement Calculation of Retrofitted Structure.

								hedef deplasman (m)
Tx	Sae(T)/g	Sae(T)	Sde (m)	ay1	Ry1	CR1	Sdi1	uxN1
2.416	0.055588	0.545316	0.080627	2.817628	0.193537	1	0.080627	0.080627318
Ty	Sae(T)/g	Sae(T)	Sde (m)	ay1	Ry1	CR1	Sdi1	uyN1
1.586	0.0686	0.672968	0.042879	1.347561	0.499397	1	0.042879	0.042878674

4.2.4 Assesment Analysis Results for Retrofitted Building

Retrofitted structure is evaluated again using nonlinear fixed first mode pushover analysis up to the target displacement determined in the previous section. Assesment analysis are performed both for DD1 and DD2 earthquake level and it is determined that all column and beam plastic rotations of the structure satisfied limited damage level and all columns and beams perform ductile behaviour for both of the earthquake level (Table 11 and Table 12).

Table 11: Column Damage Levels and Shear Check of Retrofitted Building Under DD1 earthquake level.

Column Damage Levels					
Storey	Limited Damage	Kontrolled 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 12: 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

Storey drift checks are carried out for the retrofitted building under DD2 earthquke 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

Pin connections are controlled for DD1 earthquake level for overturning and pin failure. They are both determined as safe for retrofitted building.

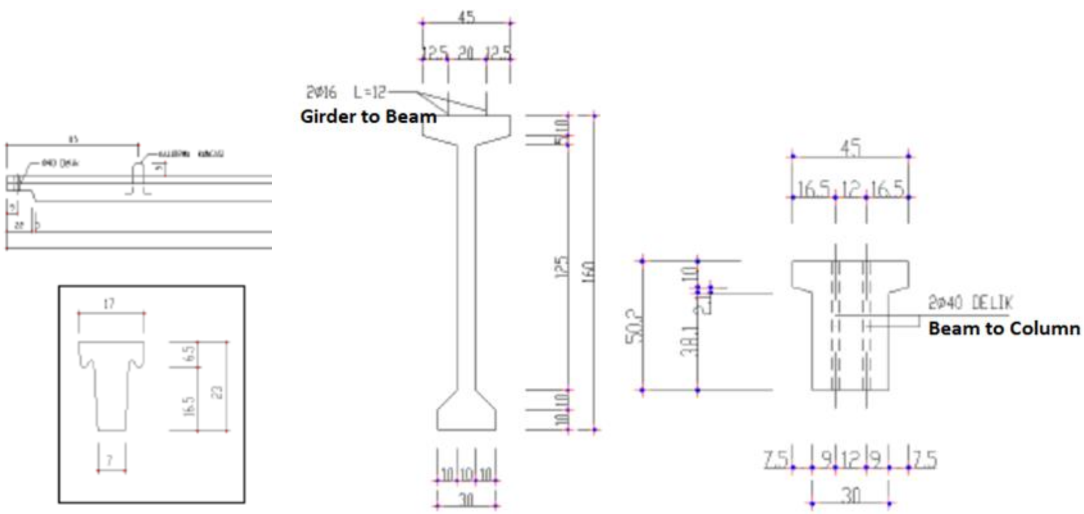


Figure 19: Pin Connection of Girder to Beam and Beam to Column

Additionally stresses under foundations are checked and found to be less than 400 kPA which is soil bearing capacity.

5. Conclusions

Reinforced Concrete Prefabricated Building is determined as Controlled Damage Level as per TBDY 2018 under DD2 earthquake level and some of the beams perform brittle behaviour as detailed in section 3. Because Client requires Limited Damage Level for the structure a retrofitting study is performed using Friction Dampers.

After retrofitting studies as detailed in section 4, building performance has been upgraded to Limited Performance Level and all of the column and beam members performed ductile behaviour satisfying Limited Performance Level of TBDY 2018.

Retrofitting installation works are performed without stopping functionality of the Building with some safety precations and small separations (Photo 2-3).

Photo 2-3: Installation Photos

References:

[1] Ersoy, U., Özcebe, G. ve Tankut, T., (2000), 1999 Marmara ve Düzce Depremlerinde

Gözlenen Önüretimli Yapı Hasarları, 10.Prefabrikasyon Sempozyum Bildirileri Kitabı, Deprem ve Prefabrikasyon, Sayfa 1, İstanbul.

[2] Özmen, G., Yüzügüllü, O, ve Zorbozan, M., (1997), Yeni Deprem Yönetmeliğinin Prefabrike Yapılar Bakımından İrdelenmesi, Türkiye Deprem Vakfı Yayınları, TDV/TR 97-006, İstanbul.

[3]] Palanci.M and Şenel Ş.M."Rapid seismic performance assessment method for one story hinged precast buildings"Structural Engineering and Mechanics 48(2) 257-274 (2013).

[4] Cahit Gürer Kocatepe Univercity Yapı Teknolojileri II.

[5] TBDY 2018- Turkish Building Earthquake Code

[6] DD-1 Earthquake level with 2% exceedence probability in 50 years.

[7] DD-2 Earthquake level with 10% exceedence probability in 50 years.

[8] DD-3 Earthquake level with 50% exceedence probability in 50 years.

[9] DD-4 Earthquake level with 68% exceedence probability in 50 years.

[10] ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings; American Society of Civil Engineers: Reston, VA, USA, 2017.

[11] 5. International Conference on Earthquake Engineering and Seismology (5ICEES) 9386 and 10637 no papers Erkuş B, Yıldırım S, Güler M.D., Özer C, Sütçü F, Alhan C. 2019