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Earthquake Performance Analysis of Steel Structures with A3 Plan Irregularities

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Abstract: In earthquake engineering, a performance-based design method is used to determine the level of the expected performance of the structures under the earthquake effect. The level of performance is related to the damage situation that could be occurred in the structure after the earthquake. In the performance-based structural design, it is predicted that more than one damage levels emerge under one certain earthquake effect.
In this study, the seismic behavior of steel structures with plan irregularities in the Turkey Building Earthquake Code in the 2018 (TBEC-2018) is investigated by the nonlinear static analysis methods. The selected steel structures are located in İzmir, Turkey. The Turkey Earthquake Code in 2018 is considered for assessing seismic performance evaluation of the selected moment-resisting frame steel building. Four different A3 type irregularity was investigated. The steel building with no irregularity in its plan. was selected as the structure of the reference. The performance goals of the five different steel structures are evaluated by applying the pushover and procedures of the TBEC-2018. The steel structures were compared by obtaining pushover curves for both the X and Y directions. The results show that the effects of A3 type irregularity should be not considered in design and buildings without irregularities are safer.
Keywords: Steel structures, nonlinear static pushover analysis, performance analysis, plan irregularity, A3 type irregularity

1. INTRODUCTION

In Turkey, there are many buildings at the border and under the border of earthquake safety. Accurate modeling of the seismic action is important to observe the real behavior of the structure under earthquake forces. In the Turkey Building Earthquake Code in 2018 (TBEC-2018), performance-based evaluations were to the fore by using advanced knowledge of earthquake engineering. Earthquake resistant design of steel structures has been developing in the last years by means of analytical and experimental results. Although structural steel is in many ways an ideal material for earthquake resistance, care should be taken in the design and detailing of framing. Earthquakes which affect the structure during its service life may sometimes be very destructive in Turkey and also in the whole world. Therefore, the subject of earthquake engineering and earthquake-resistant design is getting to be more important in the world in recent years. The latest Turkish building earthquake code was brought into force in 2018 to analyze the structures according to earthquake-resistant design concept. The necessity of having regular structural systems is emphasized in the TBEC-2018 while in some conditions it is unavoidable to apply. The plan irregularities in the TBEC-2018 code are: These, A1- Torsional Irregularity, A2- Floor Discontinuities and A3- Projections in Plan. The case where Torsional Irregularity Factor, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum relative stories drift at any stories to the average relative stories drift at the same stories in the same direction, is greater than 1.2. Floor Discontinuities: In any floor, the case where the total area of the openings including those of stairs and elevator shafts exceeds 1 / 3 of the gross floor area. The case where local floor openings which make the safe transfer of seismic loads difficult to vertical structural elements. The cases of abrupt reductions in the in-plane stiffness and strength of floors. A3 – Projections in Plan: The cases where dimensions of projections in both two perpendicular directions in plan exceed the total plan dimensions of that stories of the building in the respective directions by more than 20%

The studies-based irregularity procedures have been realized for the reinforced structures (Giannakouras and Zeris, 2019; Krawinkler and Seneviratna ,1998). The most common assessment procedures are explained in four main guidelines/codes which are Applied Technology Council (ATC-40), Federal Emergency Management Agency (FEMA 356), FEMA440 and TBEC-2018. TEC-2007 came into use in 2007.

There are many studies related to the performance analyses. These studies evaluated seismic performance of existing low and mid-rise reinforced concrete buildings by comparing their displacement capacities and displacement demands under selected ground motions experienced in the world (Jialiang and Wang, 2017; Inel et al. 2016, Çavdar and Bayraktar, 2014; Duan and Hueste, 2012). In this study, the nonlinear static pushover analysis is used to estimate the expected seismic performance of a regular steel building and four different irregular steel buildings. The buildings are moment resisting frame steel building. The 3D pushover analysis is performed by using the finite element program SAP 2000 (Wilson and Habibullah,1997). Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP2000 provides default or the user defined hinge properties options to model nonlinear behavior of components. In this study, user-defined hinge properties are implemented. Seismic performance evaluation is carried out in accordance with the recently published TBEC-2018 that has similarities with FEMA-356 guidelines.

2.THEORY

2.1. Performance Levels of Buildings Under Earthquake Effects According to TBEC–2018

As shown in Fig. 1(a), five points labeled A, B, C, D, and E define force–deformation behavior of a plastic hinge. The values assigned to each of these points vary depending on type of element, material properties, longitudinal and transverse steel content, and axial load level on the element (TBEC,2018; ATC-40, 1996; FEMA-356, 2000). The definition of user-defined hinge properties requires moment-curvature analysis of each element. Mander model (Mander et all., 1988) for unconfined and confined concrete and typical steel stress-strain model with strain hardening for steel are implemented in moment-curvature analyses. The points B and C in Fig. 1 are related to yield and ultimate curvatures. The point B is obtained from SAP2000 using approximate component initial effective stiffness values as per TBEC-2018.

Similar to ATC and FEMA, three limit conditions have been defined for ductile elements on the cross section in TBEC-2018. These are Limited Damage Zone (LD), Controlled Damage Zone (CD) and Prevention Damage Zone (PD). Limited damage limit defines the beginning of the behavior beyond elasticity, safety limit defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength, and collapsing limit defines the limit of the behavior before collapsing. This classification does not apply to elements damaged in a brittle condition. Elements that the damages with critical sections do not reach LD are within the Limited Damage Region, those in-between LD and PD are within Controlled Damage Region, those in-between CD and PD are in Advanced Damage Region, and those going beyond PD are within Collapsing Region (Fig.1b).

3. METHODS

3.1. Description of Investigated Steel Buildings

The steel buildings are typical beam-column steel frame buildings. A typical floor plan is shown in Fig. 2 reference steel building which has no irregularity. The steel building has 7 spans in the X direction and 5 spans Y direction. The all-steel buildings were chosen 5 stories, first story is 4.0 m and other stories 3.0 m in height. Column dimensions in first and second stories are HE 400B profile and HE 360B profile for other stories. Beam dimensions in first and second stories are IPE 400 profile and IPE 360 profile for other stories. Secondary beams both X direction and Y direction were chosen IPE 270 profile. For the reference steel building where the slabs act as rigid diaphragms on the horizontal axis, two horizontal translocations per floor and independence levels for the rotations around the horizontal axis will be considered. Independence levels of the floors will be defined for the center of mass of each floor and additional eccentricity will not be applied.

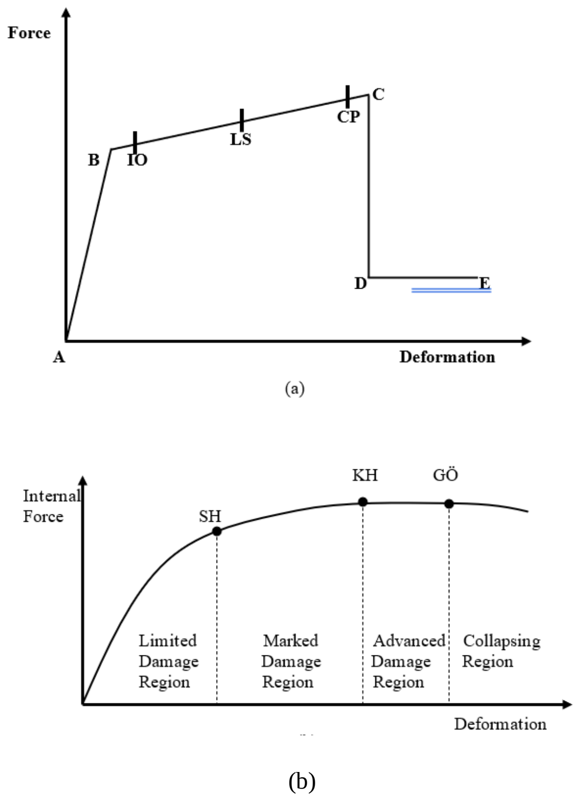


Fig.1. Building performance levels according to TBEC-2018.

However, the validity of this approach is checked especially in cases of irregularities in the floor plans. According to the TBEC-2018, in the seismic zones, it shall be verified by the calculation that the floor systems can transfer the seismic loads safely between vertical structural elements. The dead load is G =4.78 kN/m2 for all the floors. The live load is Q= 4.9 kN/m2 for each floor except the top floor where the live load was considered as 2.25kN/m2. The steel structures are thought to be housing and its coefficient of live load addition is taken as n = 0.3. The steel structures are in İzmir and in first-degree seismic zone. A design ground acceleration of 0.4g and soil class ZC that are similar to class C soil of FEMA-356 is considered in the analyses. Three-dimensional finite element model of the regular steel building and of the steel buildings with A3 irregularities was prepared in SAP2000 structural analysis program shown in Fig. 3-7. The pushover analysis is performed by using the finite element method Structural Analysis Program-2000 (SAP2000). Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP2000 provides default or the user defined hinge properties options to model nonlinear behavior of components. In this study, user-defined hinge properties are implemented. Seismic performance evaluation is carried out in accordance with the recently published TBEC-2018 that has similarities with FEMA-356 guidelines.

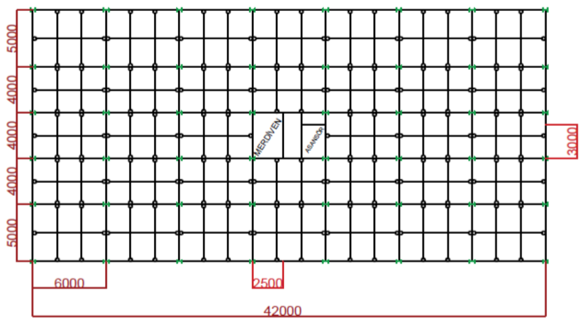
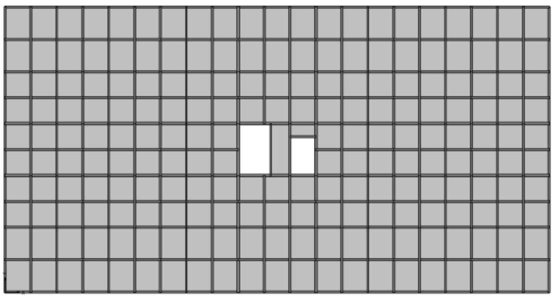


Fig. 2. Typical floor plan of the building.



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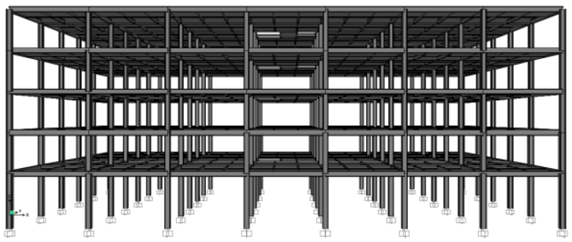

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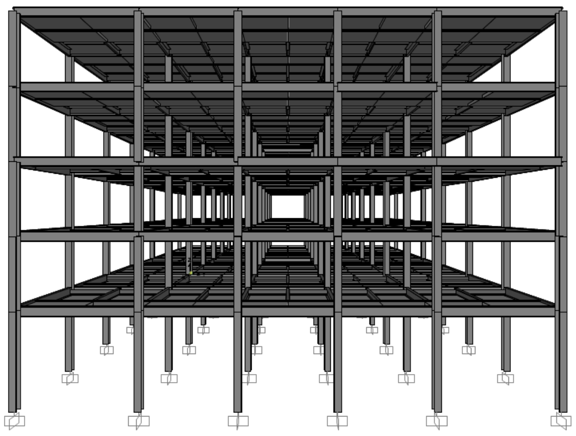

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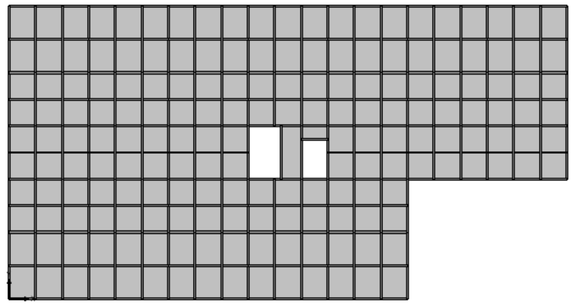
Fig.3. Plan view (a) XZ view(b) YZ view (c) of Regular steel building.



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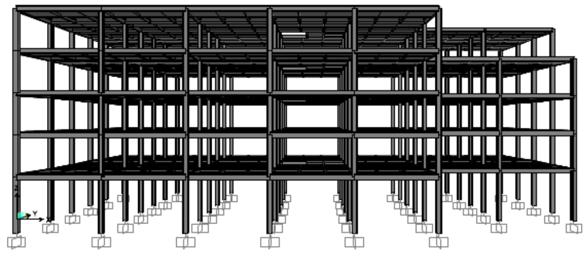

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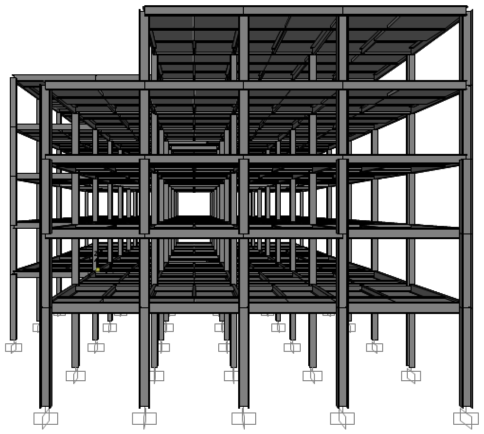

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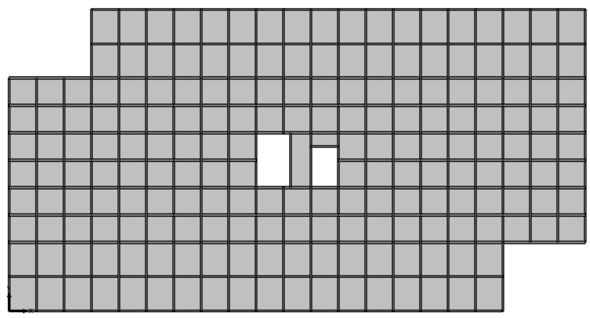
Fig.4. Plan view (a) XZ view(b) YZ view (c) of Model 1 steel building.



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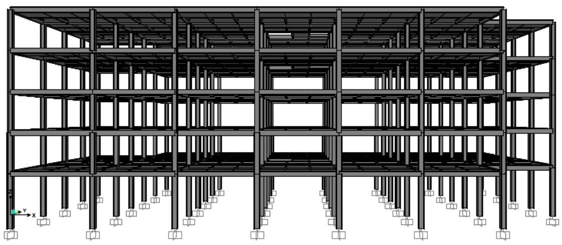

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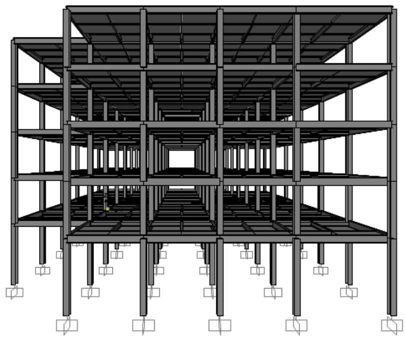

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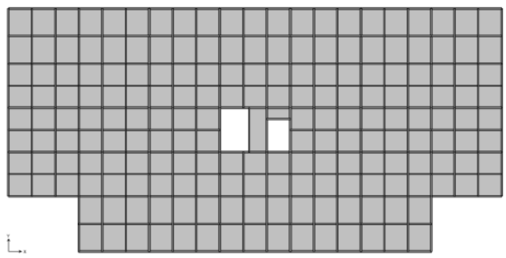
Fig.5. Plan view (a) XZ view(b) YZ view (c) of Model 2 steel building.



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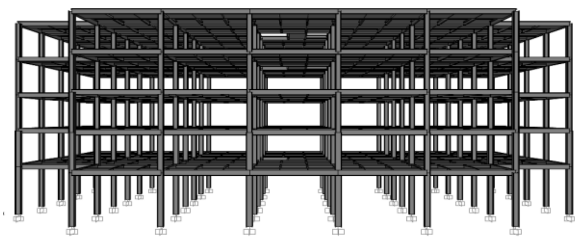

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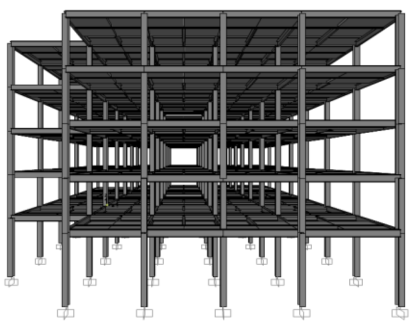

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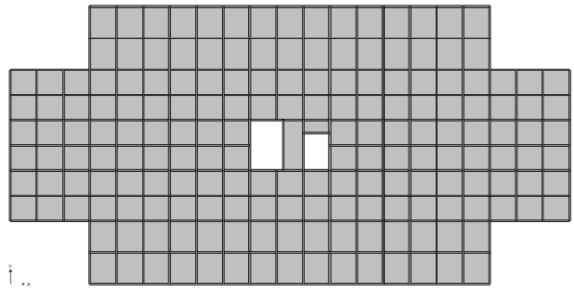
Fig.6. Plan view (a) XZ view(b) YZ view (c) of Model 3 steel building.



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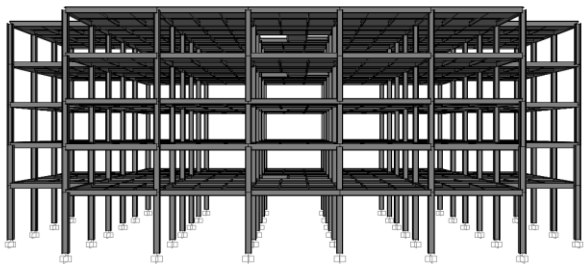

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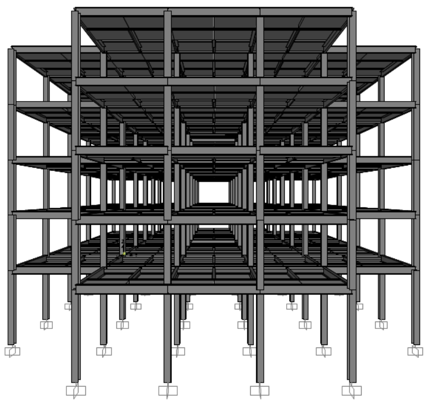


(c)

Fig.7. Plan view (a) XZ view(b) YZ view (c) of Model 4 steel building.

3.2. Performance Evaluation with Nonlinear Pushover Analysis

The aim of the nonlinear pushover analysis methods to be used for determining the structural performances of the buildings under seismic effect and for the strengthening analyses is enabling the measurement of the plastic deformation volitions regarding the ductile behavior and internal force volitions concerning the brittle behavior for a given earthquake. Afterwards, the magnitudes of the mentioned volitions are compared with the deformation and internal force capacities that are defined in TBEC-2018 and structural performance evaluation shall be conducted both at sectional and building level.

According to TBEC-2018, to be able to use the pushover analysis, the torsional irregularity coefficient (ηbi) that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition ηbi < 1.4 for each floor. The torsional irregularity of the buildings is provided.

Moreover, in accordance with the earthquake taken into consideration, the ratio of the active mass of the primary (dominant) vibration mode was calculated taking the linear elastic behavior as a basis point to the total mass of the building (except for the masses of the basement floors covered by the rigid frames) should be above 0.95 (TBEC, 2018). Because the building provides all these conditions, the nonlinear pushover analysis is utilized. Before incremental pushover analyses, a static analysis is done by taking into consideration vertical loads that are harmonic with the masses. This analysis is force-controlled and the results of this study are assumed as initial conditions of incremental pushover analyses. The vertical loads in nonlinear static pushover analyses are assumed as follows:

Vertical Load Combination (TBEC, 2018)

$$G+nQ=G+0.3Q \quad (1)$$

In Eq. (1), G is total dead load, n is the live load participation factor, Q is total live load stories of building, respectively.

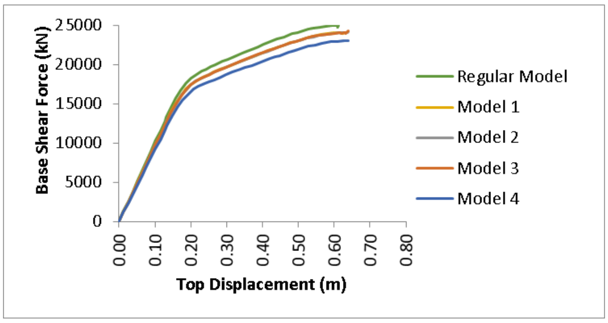
The pushover analysis of the selected structures is actualized under DD-2 (design earthquake) (10% in 50-year hazard level) as proposed in the TBEC-2018. Nonlinear static pushover analyses are determined by SAP2000. A design performance level is a statement of the desired structural behavior of a building. After determination of damage regions of sections, the performance. levels of the steel buildings are controlled. It is seen from Fig.8 that the based shear force and top displacements through the steel frame structures of models of in the X and Y direction after pushover analysis is under design earthquake (10% in 50-year hazard level).

Since A3-projections plan irregularity was examined in the study, all values related to the structure were taken as the same but this irregularity value was changed. In comparison to the regular model, the maximum base shears forces decreased by 18% in the X direction and by 26% in the Y direction. The highest decrease. in the X direction was determined in Model 4, while the highest. decrease in the Y direction was determined in Model 4.

According to TBEC-2018, the buildings that satisfy the conditions mentioned below can be agreed to be in Life Safety (LS) performance level provided that the brittle damaged components, if any, are strengthened:

(a) As the result of the calculations made for each earthquake direction applies on each floor, at most 30% of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) and at most the proportion of the columns defined in "paragraph b" can exceed the Advanced Damage Zone.

(b) The total contribution of the columns in the Advanced Damage Zone to the shear force that is borne by the columns in each floor should not exceed 20%. For the top floor, the ratio of the total shear forces of the columns in the Advanced Damage Zone to the total shear forces of all the columns at that floor can be at most 40%.



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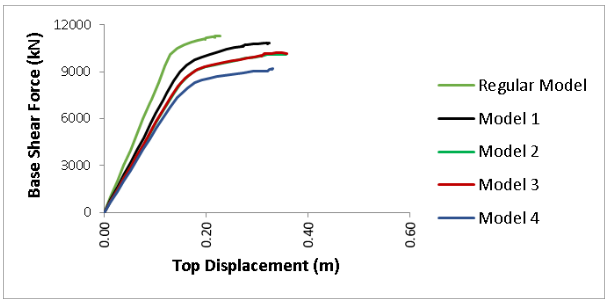


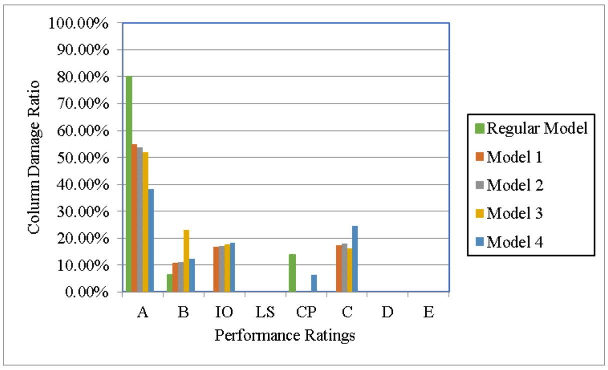
(b)

Fig. 8. Comparison of pushover curves for X and Y direction for different steel models.

The performance levels, LD, CD, and PD are considered as specified in this code and several other international guidelines such as FEMA-356 and ATC-40 (Fig. 1). Displacement volition estimates for earthquakes with a probability of exceedance of 10% in 50 years are compared for LD, CD, and PD displacement capacities. For any floor, if these ratios do not exceed the targeted performance level's ratio, it is concluded that the building is sufficient for CD under design earthquake.

It can be seen from the result under soil class ZC design earthquake of the pushover analysis through the X and Y direction (Fig.9a-b). It is concluded from nonlinear static pushover analysis under design earthquake that according to displacement target of the building, the buildings provided CD rating in the view of LD level targeted in TBEC-2018. According to TBEC-2018, the regular model is expected to satisfy LD performance levels, but irregular models are not expected to satisfy LD performance levels under design earthquake.

The highest decrease in the X direction was found in Model 4, while the highest decrease in the Y direction was found in Model 4. As Model 1, Model 2 and Model 3 had symmetry, the values for. the X and Y directions were highly close to each other. As the center of rigidity will get further away from the center of mass in irregular structures, the torque will create additional shear forces on vertical load-bearing structures. These will affect the earthquake resistance of the structure negatively.



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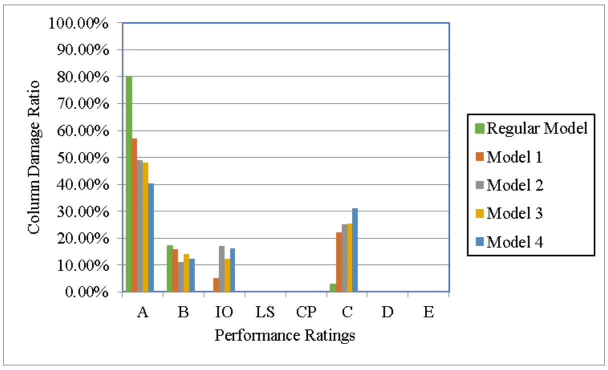

(b)

Fig.9. Columns performance levels of (a) X direction (b) Y direction of the steel building obtained by pushover analysis.

4. CONCLUSIONS

This paper investigates the seismic performance of five different buildings designed according to the provisions of TBEC-2018. The Pushover analysis was used to evaluate the seismic performance of the building. Performance evaluation is performed using the current Turkish Building Earthquake Code, TBEC-2018. The performance levels, LD, CD, and PD are considered as specified in this code and several other international guidelines such as FEMA-356 and ATC-40. Pushover analysis and criteria of TBEC-2018 were used to determine global displacements of the building corresponding to the performance levels considered above. Displacement volition estimates for an earthquake with probability of exceedance of 10% in 50 years are compared for LD, CD, and PD displacement capacities.

The pushover analysis is a simple way to explore the nonlinear behavior of the buildings. The results obtained in terms of pushover volition, capacity spectrum and plastic hinges gave an insight into the real behavior of structures. Pushover analysis is not only useful for evaluating the seismic performance of the structures, however, could also be helpful for selecting seismic details that are more suitable for withstanding the expected inelastic deformations. According to TBEC-2018, the regular model is expected to satisfy LD performance levels but irregular models are not expected to satisfy LD performance levels under design earthquake.

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Earthquake Analysis of a High-Rise Building Retrofitted with Support Braced Systems

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Abstract: The use of support braced systems represents one of the best solutions for retrofitting or upgrading the tall reinforced concrete buildings in areas with a high earthquake hazard. In this study, the behavior of a reinforced concrete tall structure under seismic loads is examined based on the Turkish Building Earthquake Code 2019 (TBEC-2019). Support braced systems were added to the 25-story structure on 0.4H and 0.8H levels (H is height of structure). For two different models, firstly, the Mode-Superposition Method for linear computational methods used within the scope of strength-based design is performed. In order to determinate more accurately the behavior of tall buildings, as in the earthquake regulations of other developed countries, the TBEC-2019 advises a nonlinear deformation-based design approach. In addition, the nonlinear time history analyses of these buildings were performed. As a result of these analyzes, it was determined whether the two models examined were within the targeted performance effects or not. In the model having support braced system, stiffness and shear forces in shear walls were increased. Thus, displacements, relative story drift, plastic rotations and bending moments of shear walls were decreased.
Keywords: Tall reinforced concrete buildings, Seismic performance evaluation, Mode-Superposition method, Support braced system, TBEC-2019.

1. INTRODUCTION

Support braced systems contain core wall and exterior columns which connected by rigid girders to core. These rigid elements depth size can be one or two-story height. When outrigger braced systems were exposed earthquake and wind loads, surrounding columns which restrained by outrigger beams resist core rotation. This resistance causes tension and compression forces on exterior columns (Taranath, 1974). After destructive earthquakes, many new and existing reinforced concrete tall buildings in first-degree seismic zone are needed seismic evaluation because of their unfavorable seismic behavior, due to strength and displacement problems in high-rise building. Especially, serious damages and many losses happened after 1989 Loma Prieta and 1994 Northridge earthquakes in the United States of America, 1995 Kobe earthquake in Japan; 1992 Erzincan, 1999 Marmara and Duzce, 2011 Van, 2020 Elazig and 2020 İzmir earthquakes in Turkey. Therefore, performance-based design procedures have been investigated for the structures recently. Performance-based design and evaluation methods developed to determine building security more realistically and contribute to strengthening structures that are not thought to have sufficient security. Few codes in the world have regulatory requirements towards performance based seismic design of high-rise buildings. Seismic Design Code for Tall Buildings in Istanbul was proposed in 2008; however, it has not been put into implementation yet. Turkey Building Earthquake Code (TBEC-2019) is published in 2019. There are several procedures for performance assessment in the literature. The most common assessment procedures are explained in four main guidelines/codes which are Federal Emergency Management Agency (FEMA-440), Applied Technology Council (ATC-40), FEMA 356, and Turkish Building Earthquake Code (TBEC-2019). As the tendency to build high buildings in Turkey increases, the TBEC-2019 has added special rules section for the design of high building systems under the influence of earthquakes.

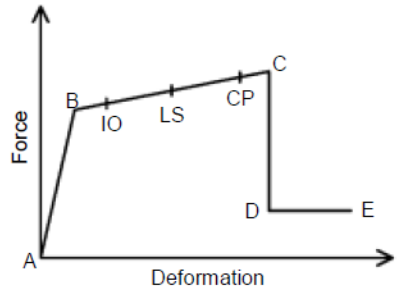
The most basic research topic in the studies on retrofitted with outrigger beams has been the location of the beam. The first study of externally supported systems was conducted by Taranath in 1974. In the study, the effect of on top displacement single outrigger under certain acceptance and simplifications was investigated. The use of buckling restrained braces (BRBs) represents one of the best solutions for retrofitting or upgrading the numerous existing reinforced concrete framed buildings in areas with a high seismic hazard. The effectiveness of BRBs for the seismic retrofit of reinforced concrete (RC) was investigated by Castaldo. Many papers have been published on the topic of outrigger beams usage of high-rise building (Hoenderkamp and Bakker, 2003, Wu and Li, 2003, , Hoenderkamp, 2008, Liu etc., 2012, Patil and Keshav, 2016, Tavakoli, etc.,2019, Karki etc.,2020, Castaldo etc., 2021).

In this study, the nonlinear static pushover and time history analyses are used to estimate the expected seismic performance of a tall building, in the Istanbul city of Turkey. Linear and non-linear behavior of reinforced concrete high-rise buildings which height is H and has two support braced systems at 0.4H-0.8H location are investigated. For two different models, firstly, spectrum analysis according to mode superposition methods of linear computational methods which is used within the scope of strength-based design is performed. To determine more accurately the behavior of tall buildings, as in the earthquake regulations of other developed countries, the TBEC-2019 advised a nonlinear deformation-based design approach. For this purpose; a 25-storey reinforced concrete building with a total height of 100.0 meters was investigated with support braced systems and without support braced system. In addition, the nonlinear time history analysis of these buildings was performed. The building is typical beam-column RC frame buildings with shear walls. The building was designed according to TBEC-2019 considering both gravity and seismic loads.

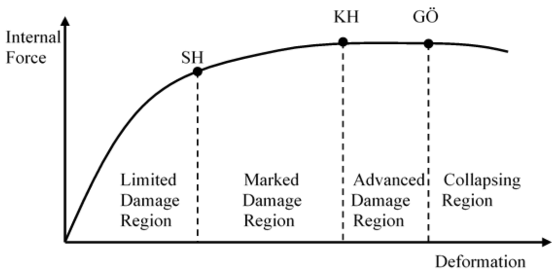
2. Theory

2.1. Performance Levels

TBEC-2019 defines three-stage process as it is explained earlier on PEER Performance Based Design approach. The tall buildings are defined Class 1 of Buildings that have heights presented in TBEC-2019. As shown in Fig. 1a, five points labeled as A, B, C, D, and E define force–deformation behavior of a plastic hinge. The values assigned to each of these points vary depending on type of element, material properties, longitudinal and transverse steel content, and axial load level on the element (ATC-40; FEMA-273) .



(a) ATC-40, FEMA-273



(b) TBEC-2019

Fig.1. Force-Deformation relationship of a typical plastic hinge.

Similar to ATC and FEMA, three limit conditions have been defined for ductile elements on the cross section in TBEC-2019. These are Limited Damage Zone (SH), Controlled Damage Zone (KH) and Prevention Damage Zone (GÖ). Limited damage Zone defines the beginning of the behavior beyond elasticity, safety limit defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength, and collapsing limit defines the limit of the behavior before collapsing. This classification does not apply to elements damaged in a brittle condition. Elements that the damages with critical sections do not reach SH are within the Limited Damage Region, those in-between SH and GÖ are within Controlled Damage Region, those in-between KH and GÖ are in Advanced Damage Region, and those going beyond GÖ are within Collapsing Region (Fig.1b).

3. Description of Investigated Reinforced Concrete Tall Structures

3.1. Analytical Model

In this study, two high-rise building models are designed. The designed model is preferred as a shear wall-framed bearing system. In the second model, steel braced system has been added to the existing bearing system, performance analyzes are made for the two models. A typical floor plan is shown in Fig. 3. The total height of the building from the foundation level is 100 m with 4 m story height. The buildings have an extremely regular structural floor plan. Typical floor plan of the building without outrigger beam and with outrigger beam as shown in Fig.3-4. Buildings consist of 2 basement story, 1 floor story and 23 normal stories. Basement story surrounded by rigid shear wall were used for the building model. The application floor plan for normal floors is given in Fig.2. The floor application plan for the outrigger beam model (Model 2) is given in Fig. 3. XZ application plan view for buildings is shown in Fig.4. The bearing element dimensions used for both building models are given in Table 1.

Structural element	**Section dimensions (mxm)**	
	Model 1	**Model 2**
Basement shear wall	30.0X0.30 - 0.30X30.0	30.0X0.30 - 0.30X30.0
Other shear wall	0.40X6.0 - 6.0X0.40 0.50X6.0 - 6.0X0.50 0.60X6.0 - 6.0X0.60	0.40X6.0 - 6.0X0.40 0.50X6.0 - 6.0X0.50 0.60X6.0 - 6.0X0.60
Columns	1.0X1.0 - 0.90X0.90 0.80X0.80	1.0X1.0 - 0.90X0.90 0.80X0.80
Beams	0.40X0.80	0.40X0.80
Slaps	hf= 0.15	hf= 0.15
Steel braced bottom/top title frames	—	"I" Profile 0.25X0.25X0.25 (h = 0.03)
Steel braced frame	—	Circle = 0.25 (t = 0.03)
Steel orthogonal frame	—	Square 0.25X0.25 (t = 0.03)

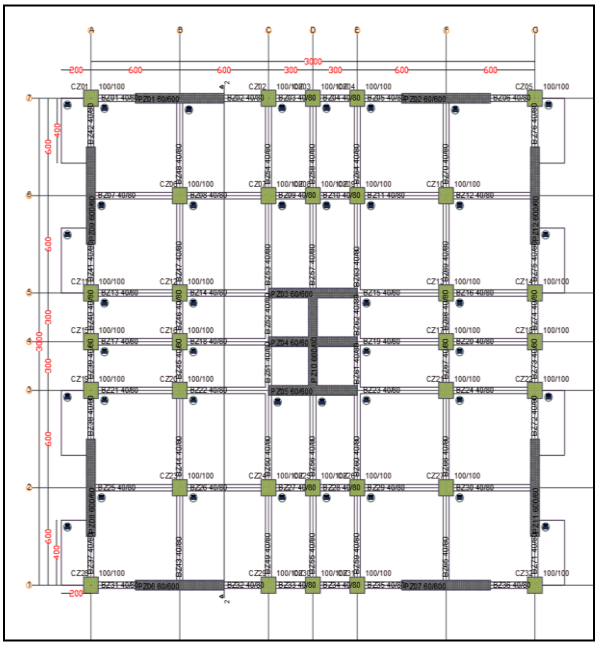


Fig. 3. Typical floor plan of the building without support braced system (Model 1) (Units are cm).

The buildings consist of concrete slabs sitting on beams supported by shear walls and columns for vertical load bearing system. The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns, beams shear walls. The lateral load carrying system of the building consists of shear walls with coupling beams distributed in the floor plan as required by architectural needs. The projected concrete class is C50/60 (according to EN 206-1 standard) and projected reinforcing steel class is B420

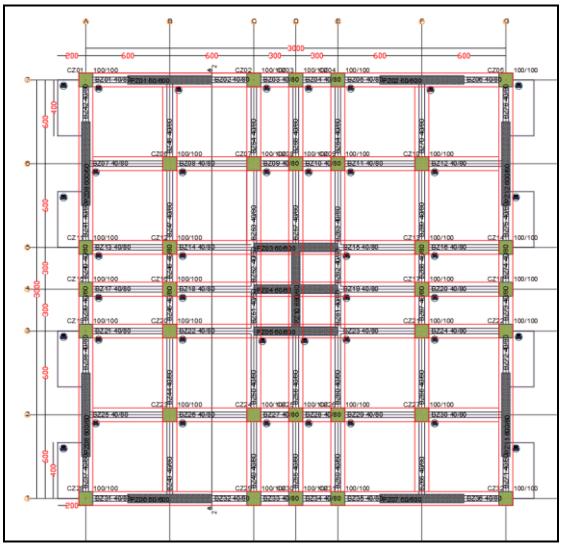
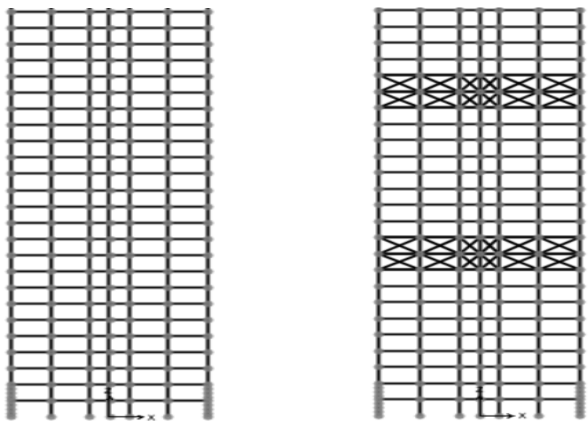


Fig. 3. Typical floor plan of the building with support braced system (Model 2).

(according to EN 10080 standard). A design ground acceleration as 0.4g and soil class ZC are considered.in the analyses. The dead load is G =3. 5 kN/m^2 for the basement floors, G=2 kN/m^2 for the normal floors except the top floor where the dead load was considered as G = 1.5 kN/m^2. The live load is 2 kN/m^2 each for housing rooms and hallway. The live loads are 1.5 kN/m^2 for the top floor (EN 498 standard). The structure is thought to be a housing and its live load contribution factor is taken as n = 0.3. The high-rise buildings were analyzed in detail by performing nonlinear dynamic analyses according to the TBEC-2019. The limitation of relative displacement and second-order effects are described in TBEC-2019 (section 4.9). For a shear wall or column according to TBEC-2019 (section 4.9.1.1), the difference in displacement between consecutive two floors are expressed as reduced relative displacement (Δi). Effective relative displacement in any direction will be calculated by Eq. (1).



(a) Model 1 (b) Model 2

Fig. 4. XZ application plan view for buildings.

$$\delta_i = \frac{R}{I}\Delta_i \tag{1}$$

In Eq. (2), I is Building portance Factor and R is Structural Behavior Factor. The effective relative displacements made in the investigation will not exceed the limit value given in Eq. (2). In the given equation, the coefficient of λ expresses the ratio of elastic design spectral acceleration at the level of DD-3 ground movement to elastic design spectral acceleration at the level of DD-2 ground movement with the earthquake direction. κ coefficient will be taken 1 for reinforced concrete buildings. DD-2 is the probability of exceedance of the design earthquake within a period of 50 years is 10 %. DD-3 is the probability of exceedance of the design earthquake within a period of 50 years is 50%. hi is story height.

$$\lambda \leq 0.008\kappa \tag{2}$$

The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns and beams. When determining seismic performance of the designed structure, Seismic Load Reduction Factor is taken as Ra=1. In addition, building importance factor is applied as I = 1. The rigidities of cracked sections are taken instead of the rigidities of uncracked sections. The information level coefficient is taken as 1 for extended information level. Predominant mode periods of the buildings in X and Y directions are 2.62 s, 2.58 s, and 1.93 s, 1.86 s respectively, based on cracked section properties. The period value in the X and Y directions for the model retrofitted with outrigger beam has decreased by 26.34%.

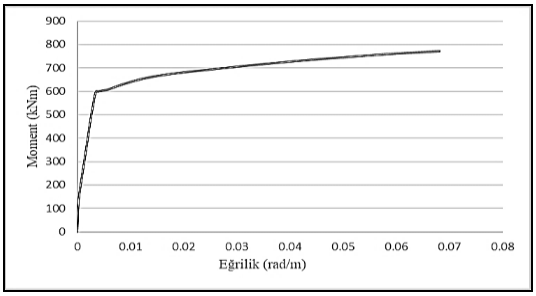
The Response2000 program is utilized during the preparation of material properties, obtainment of moment-curvature relations of each structural elements and definition of axial load-moment (PM) interaction diagrams for the columns. Effective cross-section rigidity calculation of remaining parts between plastic hinges in the columns and beams is made according to TBEC-2019. The effective cross-sectional rigidities of the columns, beams and connecting beams to be modeled according to the lumped plasticity behavior are determined according to Eq. (3). Moment- curve diagrams for beams and columns are given in Fig.5.

$$(EI)_e = \frac{M_y L_S}{\theta_y\, 3} \tag{3}$$

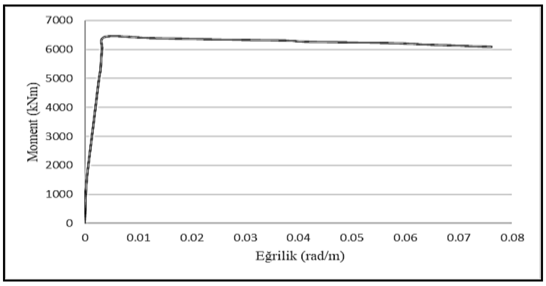
In Eq. (3), My and ϴy show the averages of the effective yielding moments and yielding rotations of the plastic hinges at the ends of the frame element. LS is the spanning shearing. The yielding rotation of the plastic hinge (ϴy) will be calculated by Eq.(4).

$$\theta_y = \frac{L_S\, \emptyset_y}{3} + 0.0015\eta\left(1 + 1.5\frac{h}{L_S}\right) + \frac{f_{ye} d_b\, \emptyset_y}{8\sqrt{f_{ce}}} \tag{4}$$

In the Eq. (4) Øy demonstrates the effective yielding curvature in the plastic hinge section, while h is the cross-section height. In the continuation of the formula, η =1 in beams and columns, η = 0.5 in shear walls will be taken. db shows the average diameter of the reinforcement steels, while the fye and fce show the average yield resistance of the reinforcement with the average pressure resistance of the concrete.



(a) Moment-curve for beams



(b) Moment-curve for columns

Fig. 5. Typical moment- curve diagrams

4. Nonlinear Seismic Performance. Evaluation of the Building

Regarding the definition of high-rise buildings whose design and construction should be avoided because of their unfavorable seismic behavior, types of irregularities in plan and in elevation. Irregularity calculations were done by applying the procedures defined in the TBEC-2019 for these buildings. The case where Torsional Irregularity Factor (ηbi), which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum relative story drift at any story to the average relative story drift at the same story in the same direction, is greater than 1.4.

The torsional irregularity coefficient (ηbi) that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition ηbi < 1.4 for each floor. The torsional irregularity and inter-story stiffness irregularity ratios of the buildings is provided. There are no local slab abrupt reductions in the plane stiffness and strength of floors and seismic loads are safely. transferred to vertical structural elements. Therefore, floor discontinuities irregularity (A2) does not exist. Since the re-entrant corners in both two principal directions in plan do not exist, there is A3 type irregularity in the structure.

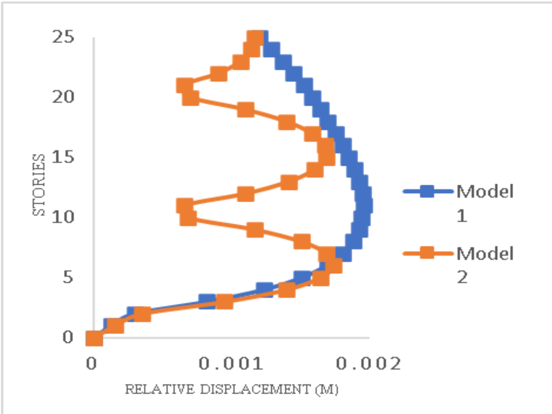
Vertical Load Combination (TBEC 2019)

$$G + nQ = G + 0.3Q \quad (5)$$

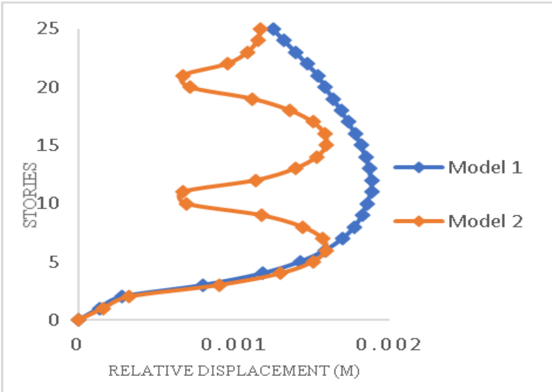
In Eq. (5), G is total dead load, n is the live load participation factor, Q is total live load stories of building, respectively.

In this calculation, cracked section bending rigidities of columns, beams shear walls are determined by analyzing bearing system under the vertical loads that is harmonic with masses according to TBEC-2019.

The lateral displacement values of the taal buildings are given in Fig. 6. As seen from the figure, Model 2 has also made smaller displacements than Model 1.



(a) X direction



(b) Y direction

Fig. 6. Relative Displacements in the X and Y direction

4. 1. Performance Evaluation with Nonlinear Dynamic Analysis

It is assumed that nonlinear dynamic analysis defines structure behavior ideally because of the seismic loads directly applied to structure (Li, 1996). The aim of nonlinear dynamic analysis is integration of equations of the motion of the system step by step by taking into consideration of nonlinear behavior of bearing system. For each time increment, it is calculated that displacements, plastic deformations, internal forces are occurred in the system and maximum values of them during earthquake. The Newmark's method is used for solving the dynamic equilibrium equations. Although not as simple as the central difference method, it is perhaps the most popular method because of its superior accuracy.

The selection and scaling of the acceleration records used within the scope of this study were made within the framework of the principles given in TBEC-2019. Accordingly, at least 11 earthquake records should be used in the analysis. Earthquake records were obtained from the "Peer Strong Motion Database" database (Peer, 2021). In addition, Duzce-Turkey earthquake record is added to the analysis. The features to be considered when choosing an earthquake are given below.

Earthquake magnitude = 6.0-7.5 Mw

Local ground conditions = ZC

Distance to active fault plane = 10-30 km

In accordance with these features, earthquake records are selected. As two different models will be compared within the scope of this study, earthquake records matching two model periods will be selected. In this context, the Model 1's natural period is 2.62 s and the period for Model 2 is 1.93 s. The scaling interval of the earthquake records to be used will be between 0.2 and 1.5 times of these period values. From the above information, the scaling range of 0.43 s and 4.38 s were determined. Response spectra and target spectrum of scaled acceleration records are given in Fig. 7. According to nonlinear time history analysis, story drifts values for both models are given in Figs. 8-9. As seen from the figures, the designed structures provide the necessary conditions.

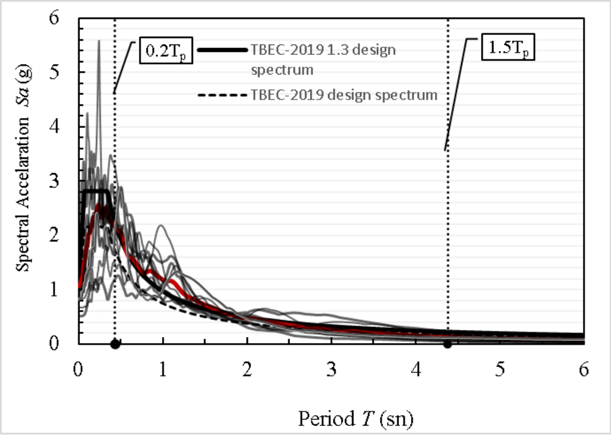


Fig. 7. Reaction spectrums for scaled acceleration recordings (PEER, 2021).

As can be seen from Fig. 8-9, it has been observed that there is a significant decrease in storey drifts in floors where external support braced systems are applied.

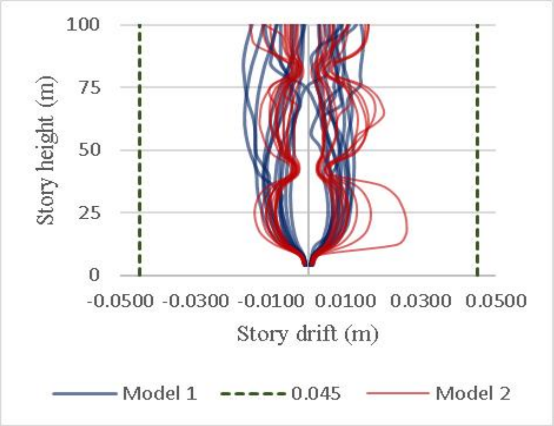


Fig.8. Story drifts for each earthquake recording in the Models

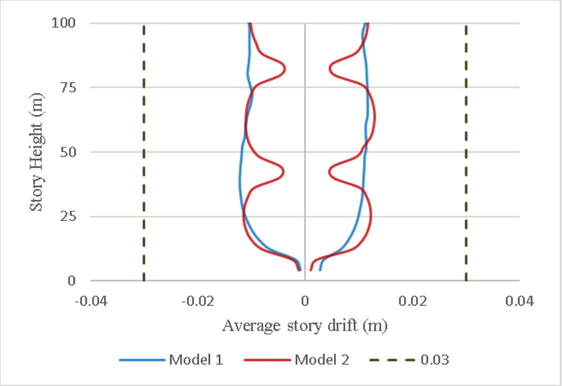


Fig. 9. Average story drifts for scaled acceleration recordings.

4.1.2. Control of Column Plastic Rotations

The plastic rotation limit for columns is calculated using Eq.(4). The calculation of rotation limit value for the 100x100 cm column used in the models is shown below.

Selected longitudinal top reinforcement: 30Ø22

Selected transverse reinforcement: 10Ø12/10

Plastic hinge length: 1.0/2 = 0.50 m

Shear span: 4.0/2 = 2.0 m

Yield and failure curvature for the typical beam section are determined by the moment-curvature diagram calculated by the Response 2000 program (Fig.5b).

$\theta_p^{(GÖ)}$=0.015 radyan

As a result of nonlinear analysis in the time history analysis for calculating the plastic rotations of the columns, the curvature values of the column ends were calculated for each earthquake record. The rotation values of the 100x100 column for the Model 1 are shown in Fig.10 and Model 2 in Fig. 11. It was observed that the rotation values decreased in the model with support braced system (Model 2). While the maximum average rotation value for the Model 1 is 0.011 rad, the maximum average rotation value for the Model 2 has decreased to 0.0082. Approximately 25% reduction has occurred. Life Safety performance level is provided for both models.

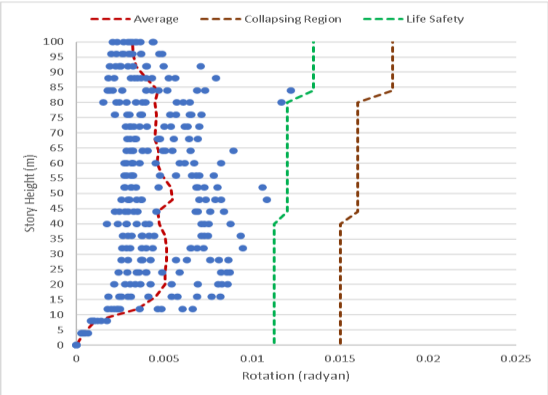
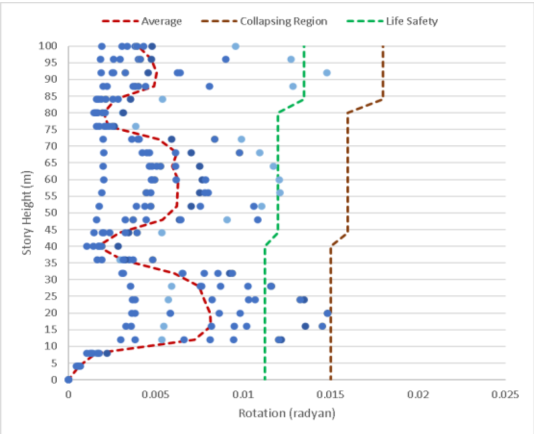


Fig.10.Rotation values of the 100x100 column for Model 1



. **Fig.11**.Rotation values of the 100x100 column for Model 2

5. Conclusions

Today, the construction of tall buildings is increasing, accordingly, earthquake analysis of tall buildings has become increasingly important. In this study, two buildings with the same bearing system and dimensions, however, additional support braced systems added to the bearing system of one of them were designed. The designed buildings in Istanbul/Turkey are considered. One of the most important reasons for the selection of the existing structure in Istanbul is that the dangerous fault lines are present within the boundaries of this province and this city is under danger of approaching and inevitable Great Istanbul Earthquake likely greater than Mw 7. Thus, investigation of earthquake performances of this or similar tall buildings are very important. In line with this information, linear and nonlinear analysis of designed buildings according to TBEC-2019 was carried out. Mode Superposition Method was used in linear analysis and Non-linear Time History method was used nonlinear analysis method and the results were obtained as follows for linear and nonlinear analysis.

The period value in the X and Y directions for the model retrofitted with support braced system has decreased by 24%. In Model 2, the amount of relative displacement compared to the Model 1 has decreased 15% respectively in the X directions. The performance of Model 2 retrofitted with support braced system is quite satisfactory in terms of exceedance of the design value of the maximum ductility capacity. This means that the support braced system exhibits a significant reserve capacity even under rare earthquake events.

In the shear wall elements where the distributed plastic hinge is accepted, the strain limit is calculated according to the ratio of reinforcement in the section and the transverse reinforcement status in the calculation made with TBEC-2019. The strain limit is determined according to the strain failure of the reinforcement.

According to the nonlinear calculation results, it was observed that the load transfer of retrofitted with support braced system is stopped by the hinge development in the diagonal ties. After load increment in the continuing push steps was covered with core shear wall which behaves as a cantilever frame.

It has been observed that by placing retrofitted with support braced systems in different positions, reduction values of 40% can be achieved in terms of shear wall bending moment and base displacement. As a result of the non-linear time history analysis, it is seen that in the evaluation of X and Y Direction line in ZC local floor class design earthquake, the level of performance of Pre -Collapse which is the target performance for the buildings is provided.

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