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Research Article

Nonlinear Seismic Performance Assessment of Energy-Based Steel Structures with Rigid Connections

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Keywords

Steel structures, High-rise building, Rigid connections, Energy-based approach, Energy dissipation, Seismic performance.

Abstract

In evaluating the seismic performance of existing multi-story buildings, strain-based static pushover and time-history nonlinear analyses are traditionally used, as stated in the regulations. In addition, by evaluating the performance levels of symmetrical and asymmetrical structures, it will also be possible to design the mechanism with damping energy capacity that will completely absorb the earthquake energy in case of an inadequate situation in absorbing the earthquake energy. In this study, using real earthquake data, the usability of the energy-based performance analysis approach in the structure was investigated on a 20-story steel structure with rigid connections for symmetric and asymmetric structures. This study was conducted to demonstrate the usability of the energy-based method in evaluating the performance level of buildings under the effect of several earthquakes. In addition, the energy-based method was preferred to determine how much earthquake energy generated in the buildings could be distributed in symmetrical and asymmetrical structures. The selected 11 earthquake data were scaled and the input energies to the structure and the energies distributed by the structure (dissipated hysteretic energy, modal energy, dumping energy, and kinetic energy) were analyzed. Additionally, moment-rotation and axial force-strain behaviors were evaluated for the performance levels of the inspected symmetric and asymmetric structures. At the end of the study, the energy-based concept was confirmed by comparing it with the static pushover analysis results in determining the seismic performances of high-rise structures with very low dynamic mass participation rates. Energy-based analyses have shown that the symmetrical structure absorbs earthquake energies more than the asymmetrical structure and remains at the immediate occupancy (IO) level, as expected. It was concluded that the asymmetric structure remained at a controllable damage (LS) level. Energy-based evaluation has been used as a performance analysis method in the literature. In addition to performance analysis, this study aimed to calculate how much energy is required for the damping mechanism to be added to the structure so that an asymmetric structure behaves like a symmetric structure. This constitutes the original value of the article. So, it has been determined that in order for the asymmetric model to be at the immediate use level, a damper with an energy capacity of 0.062 m2/sec2 is needed, which can absorb earthquake energy from the outside, depending on the building mass.

1. Introduction

The stored energy resulting from the movement of underground faults causes an earthquake in the form of earth vibrations, causing displacement and cracks in structural members. For the earthquake-resistant structural design, the rules of existing standards are applied. The performance evaluation and the seismic performance analysis of structures are important to determine how a structure will behave under an earthquake. The seismic performance-based design is emerging for a certain performance level. In other words, using seismic performance analysis shows if the project design reaches its seismic performance targets. Also, engineering performance can be evaluated through the indicators, such as displacement, acceleration, and force. In this work, in addition to the classically recommended static pushover analysis and nonlinear performance time history analysis, an evaluation will be done according to the energy-based method on the high-rise building, which has not been studied before, as this work.

Nonlinear static procedure (NSP) is based on pushover analysis used for building evaluation and design verification. As the NSP method is restricted to single-mode response, it is used for low-rise buildings with the behavior, dominated by fundamental vibration mode. By applying NSP to a tall building, in which higher mode contributions to the response are important, the seismic demands derived from NSP are greater in the upper story of the structure. A new pushover procedure as consecutive modal pushover (CMP) has been examined for accounting for higher-mode effects in tall buildings. The consecutive modal pushover is a procedure that utilizes multi-stage and single-stage pushover analysis. The CMP procedure could accurately predict the seismic demands of tall buildings, and could effectively overcome the limitations of the modal pushover analysis (Poursha et al. 2009). A generalized pushover analysis (GPA) is a newly developed procedure under earthquake ground motions. In this procedure, a prescribed seismic demand is attained for each generalized force vector by applying different generalized force vectors separately to the structure in an incremental form with increasing amplitude. A generalized force vector combines modal forces and shows the instantaneous force distribution. The generalized pushover analysis (GPA) compared to the response history analysis in estimating maximum member deformations and member forces gave successful results (Sucuoğlu and Günay 2011). The N2 method is a method that has been extended using high-rise buildings to take into account the effects of higher modes. In this method, the structure remains in the elastic range while vibrating in the higher modes. The seismic demand of displacement and story drift can be

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obtained from static pushover analysis and standard elastic modal analysis results. Using approximated methods, such as extended N2 provides us with the estimation of structural response. The accuracy decreases by increasing the height of the building and the intensity of ground motion. Generally, conservative results were obtained from the N2 method (Kreslin and Fajfar 2011). The upper bound (UB) method is used as a new method that at each analysis step considers only the first two modes of vibration. While adequate results were obtained from the adaptive upper bound procedure, it has been found that results are closer to nonlinear time history analysis (Rahmani et al. 2019). An adaptive version of the capacity spectrum method is determined to examine the capability of providing satisfactory predictions of seismic demands by comparing it with other nonlinear static procedures. The results obtained from the proposed procedure have proven a more accurate solution than the other nonlinear static procedures (Ferraioli et al. 2016). The seismic evaluation of the structures is an important issue in estimating the seismic demand of high-rise buildings. The nonlinear time history analysis method (NLRHA) is one of the most accurate methods. The computation requirement of nonlinear time history analysis is complex and time-consuming for high-rise buildings. In this case, the seismic demand estimate of mid- and high-rise buildings will not be simply possible by the conventional pushover procedure. The spectrum-based pushover analysis (SPA) is an effective method for estimating the seismic demand of high-rise buildings. In this proposed method, the complexity of dynamic coupling effects of modes because of the nonlinear seismic performance of the building is intended to be simplified. Thus, this method is adopted as a consecutive technique to consider the simplified coupling effect. The results obtained from the proposed spectrum-based (SPA) pushover analysis have very close values to the nonlinear time history analysis method for inter-story drift ratio and hinge plastic rotation with a good prediction in the seismic demand of structure (Liu and Kuang 2017).

The theory and design of seismic-resistant steel frames have progressed since the destructive earthquakes happened. The seismic design approach has been reviewed since the 1970's. It has been recognized that dissipation of earthquake input energy is possible with the plasticization of the structural members, which have to be compatible with the plastic deformation capacity of the structure. The relevant seismic design is characterized by strength and ductility requirements that both represent the seismic resistance of the structure. By using the theory of seismic design verifying the available structural ductility and the energy dissipation capacity has become possible (Mazzolani and Piluso 1996).

Seismic code is generally based on a force-controlled design using the base-shear concept as a capacity design of the structure. The response modification factor (behavior factor), which is used to design the structure at the ultimate limit state by taking into account the dissipation capacity energy, due to the formation of plastic deformation is the most important parameter in the corresponding approach. Ferraioli et al. work on the behavior of steel moment-resisting frames using a code-designed method. 12 steel moment resisting frames, half regular and half irregular in elevation of steel frames, have been designed and analyzed to evaluate over strength, redundancy, and ductility response modification factors. The analyses were done by performing static pushover analyses and nonlinear incremental dynamic analyses. The redundancy reduction factor, recommended by the Eurocode 8 and Italian Seismic Code compared to results for multi-bay and multi-story frames, is conservative which is not necessary to be so conservative. Cause in the high-rise frames the effect of axial force is to reduce the plastic moment capacity in the first story columns. In this case, the ultimate displacement capacity of the structure and the ductility reduction factor are limited by the rotation capacity of first-story columns which are limited too. Proposing a local ductility criterion for controlling the axial load level tried to ensure more ductile behavior for columns, because of the recommended conservative behavior factor value. Also, it has been concluded that the inelastic behavior of the structure under earthquake ground motion may be influenced by the characteristics of connections and the connection type of column base-to-foundation. As a result, a full-strength connection is needed to achieve valid results (Ferraioli et al. 2014). Ferraioli also works on the seismic response of steel moment-resisting frames by developing a multi-mode pushover procedure to evaluate the approximate analysis. The instantaneous force distribution derived from modal combination occurred on the structure during dynamic response to a seismic excitation while the inter-story drift reaches its maximum value. The results obtained from the multi-mode pushover procedure compared to the "exact" values of nonlinear response history analysis and the results from other nonlinear static procedures. As a result of the proposed procedure, it is obtained that this method has an accurate estimation of the seismic demand for steel moment-resisting frames (Ferraioli 2017).

Energy-based design is a new method developed recently to design buildings more reliably against earthquakes. The energy-based design method can be calculated based on the earthquake input energy and the displacements of the structure under earthquakes. There are many studies about the energy-based design method in the literature. Housner was the first to propose using energy methods in earthquake-resistant building design in 1956. Housner used multi-degree-of-freedom systems and single-degree-of-freedom systems in his research. It is stated that when the energy enters the system, the multi-degree-of-freedom systems under the influence of earthquakes remain constant over a wide period range, with certain velocity spectra of earthquakes (Housner et al. 1997). Housner also used another method which seismic energy input to single degree-of-freedom as an alternative design method. So, the requirement for the energy demand of the building to be less than or equal to the energy dissipation capacity is provided by using the concept of energy in the building design (Housner et al. 1997).

Although the application of the energy concept was limited to single-degree-of-freedom systems, the basic design of energy on the multi-degree-of-freedom systems was investigated in 1999 by Shen and Akbas. Comparing the energy input of multi-degree-of-freedom systems with single-degree-of-freedom systems, the energy dissipation capacity of the structure was obtained. Their main purpose was to associate the energy response of structural and ground motion properties. In addition, Shen and Akbas also analyzed the energy response of some frame-type structures, exposed to different ground motions to calculate the nonlinear time history method (Akbas et al. 2001; Akbaş and Shen 2003).

By working on the energy method suggested by Housner, other researchers developed a proper earthquake-resistant design method that can be applied to multi-story buildings, so that this method is not limited to single-story buildings. According to research on single-degree-of-freedom systems, they calculated the ratio of nonlinear maximum displacement to linear maximum displacement. They also tried to determine an appropriate upper limit by estimating the strain energy. Also, according to work on single degree of freedom systems and determining the energy input and hysteresis energy demand in low-rise ductile moment frames, it was obtained that the high modes are important in multi-story ductile moment frames and the building energy demand was concluded lower through equivalent single degree of freedom systems (Akiyama 1985; Bozorgnia and Tso 1986; Kuwamura and Galambos 1989; Nurtuğ and Sucuoğlu 1995; Chai and Fajfar 2000; Tso et al. 2013).

Among the literature studies, new methods have been studied to simplify the hysteretic and input energy spectra. By taking into account the desired target displacements of structures, a new method for building design is presented using the energy-based method. The effect of horizontal displacement of floors was investigated in a pre-dimensioned steel-framed building system according to the 2007 Turkish Earthquake Code (TEC) (TEC 2007). Also, the changes in dimensions of the steel-frame systems due to different relative story drifts and horizontal displacements at various floor levels were obtained. As known, generally the methods used in the seismic resistance of structures show a linear elastic behavior. It is also important that the non-linear behavior of structures under the influence of earthquakes is taken into account in seismic designs. The non-linear behavior of the building elements and the usage of energy balance were based on these studies. It has been observed that the energy-based design method provides better results for medium-height framed structures (Leelataviwat et al. 1998; Ramadan et al. 2006; Merter 2008; Swensen and Wong 2011; Merter et al. 2012a, b; Gioncu and Mazzolani 2013; Landolfo 2018; Saleem and Qureshi 2018).

To develop a method of characterizing energy in structures the major problem is using absolute or relative energy in the analysis another problem in evaluating energy and its transferring is the inelastic deformation that happened during the earthquake and caused the determination of energy induced. However, many kinds of research in this area, are inadequate and need to be renewed to determine the energy dissipation, that occurs by irrecoverable deformation. According to studies on this subject, a computation method is derived, to evaluate characterizing energy and transferring various energy components in structures during an earthquake. Using the mentioned method, the results showed the affection of energy distribution by different yielding levels. The different yielding levels affect the energy distribution. While observing no definite trend for plastic energy, it has been observed that the damping energy varies depending on the yielding level. The predominant damping energy and plastic energy determines the tendency of the input energy. Furthermore, according to the results obtained, by applying a control force in the direction opposite to the displacement and velocity responses of the structure, a decrease in the amount of input energy to the structure was observed because of the control energy which reduced the displacement and velocity responses. Withal, as the control force depends linearly on velocity, the control energy shows a similar behavior as damping energy by accumulating and disappearing during the earthquake. So, the control force could be found efficient and effective in protecting structures from damage caused by dissipating large amounts of energy during an earthquake happening (Wong and Yang 2001, 2002; Wang and Kevin 2009).

Investigating the earthquake behavior of structures according to energy-based analysis has continued in the last few years. Statistical evaluation is made for energy input, cyclic energy, and maximum inelastic displacement demand of an elastoplastic system with a certain degree of freedom under far-fault ground motions recorded in compact soils. The aim was to examine the effect of seismic and structural parameters on the maximum inelastic displacement demand of the structure, and also obtain the relationship between this demand of the structure, the energy absorption, and the dissipation capacity of the structure. On the other hand, seismic axial earthquake loads were investigated for columns in moment-resisting steel frames under severe ground motions. As increasing axial loads, maximum lateral load level, and related lateral displacements in low, medium, and high seismic force-resisting systems were investigated, the stability of the column with bending moments for high frames has been concluded as a necessary event (Shen et al. 2015; Kaboodkhani et al. 2024). In another study, a design procedure was investigated for buckling-restrained braced steel frames based on energy balance and the assumption of equal energy. By entering the seismic energy into the structure and using the concept of energy balance, it has been concluded that the energy balance concept, which provides a simplified procedure for energy-based seismic design, can be safely applied to the seismic design of low-rise buildings with buckling-restrained braces (Demir; Kim et al. 2004; Elghazouli 2010).

In the last few years, using multi-degree-of-freedom frames with different height and span configurations is another research type to determine the seismic energy demands of moment frames with high ductility levels according to the energy-based method. Linear and nonlinear time history analysis of low, medium, and multi-story steel frame systems are generally examined under scaled different earthquake acceleration records. According to the results obtained from this type of analysis in multi-story steel frame systems, a significant decrease has been concluded by entering the energy into the frame systems. As the total energy input and hysteretic energy dissipation values are concluded as not constant values, but they have a significant effect on estimating the hysteretic energy demand spectra of steel structures (Léger and Dussault 1992; Riddell and Garcia 2001; Sture 2001; Manfredi 2001; Chou and Uang 2003; Estes and Anderson 2004; Benavent-Climent 2007; Surahman 2007; Leelataviwat et al. 2008; Fabrizio et al. 2011; Mezgebo and Lui 2017; Papazafeiropoulos et al. 2017; Merter and Uçar 2018; Li 2019; Doğru and Akbaş 2020; Dedeoğlu et al. 2021).

This study is intended to evaluate the seismic performance of high-rise steel buildings using an energy-based approach. Considering literature studies, the energy-based approach is mostly used in low and medium-rise steel frames. This study evaluates the energy-based method of 20-story steel structures with symmetrical and asymmetrical plans, included in the high-rise building category. To obtain the seismic energy demands as total input energy, total dissipated energy, and hysteretic energy dissipation of structural members, both symmetric and asymmetric models with rigid connections were analyzed under eleven scaled earthquake acceleration records by considering the seismic parameters of building region and horizontal elastic design spectrum diagram depending on the natural vibration period. Also, the earthquake acceleration records were selected among the most widely used records in the literature for comparison with similar studies. By obtaining the dynamic parameters expressed, it will be determined how much energy generated by earthquakes is covered by symmetrical and asymmetrical building models. It is aimed to ensure that energy-based evaluation management is included in the relevant regulation and becomes one of the performance analysis methods. It is important to reconcile the damage indices in structures with the energy distributed so that the structures continue their functionality without being damaged by an earthquake that may occur. In other words, this study was conducted to find answers to the questions "What is the energy capacity of a building under the influence of an earthquake?" and "What is the energy required for the building to withstand this earthquake without being damaged?" If the amount of energy that the structure is insufficient to absorb is known, it will be known how much energy the energy absorption mechanisms that will be added to the structure later should be designed. In addition to evaluating the performance of a building with the energy-based method, it was also revealed how much energy capacity a damping mechanism can be designed not to meet the earthquake energy.

2. Numerical Modeling

Energy in the seismic analysis is expressed as input energy and distributed mechanical energy. For this reason, basic energy quantities; such as energy input and energy dissipation of structural systems under the ground motions need to be introduced. In the energy-based design method, the input energy value of multi-degree-of-freedom systems under earthquake effect was defined by Housner in 1956, shown in Equation 1 as follows (Doğru and Akbaş 2020).

$$E_I = \frac{1}{2} M_t S_V^2 \quad (1)$$

Where; E_I is the total energy entering the multi-degree-of-freedom system during an earthquake, M is the total mass of the system, and S_V is the elastic spectral velocity value (maximum relative speed of mass). As known this energy is dissipated (consumed) by the elastic and plastic behavior of the system, so Eq. 2 can be written for total energy as follows (Merter and Uçar 2018; Doğru and Akbaş 2020).

$$E_I = E_e + E_p \quad (2)$$

Where; E_e is the elastic energy of the system, stored in sections, and shows linear elastic behavior without exceeding the elastic limit. E_p is the plastic energy of the system stored at the plastic hinge points, which shows non-linear behavior. The lateral force-displacement diagram (Merter and Uçar 2018; Doğru and Akbaş 2020) which describes the elastic and plastic energy as an ideal elastoplastic behavior is presented in Figure 1.

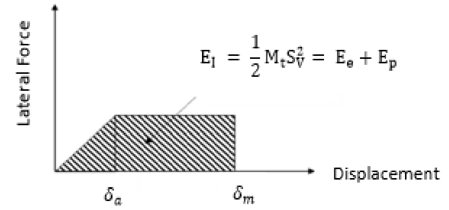


Figure 1. Lateral force-displacement diagram

In energy-based seismic design, earthquake ground motions are taken into account to calculate the energy affecting the structure under earthquake loads. The energy input is equal to the sum of the energy components kinetic, damping, strain, and hysteretic energy in the structural system. Therefore, the amount of energy entering the system and the way of distribution are important. The energy balance equation of a single-degree-of-freedom system is expressed as Eq. 3 shown below (Merter and Uçar 2018; Doğru and Akbaş 2020).

$$E_K(t) + E_D(t) + E_E(t) + E_H(t) = E_I(t) \qquad (3)$$

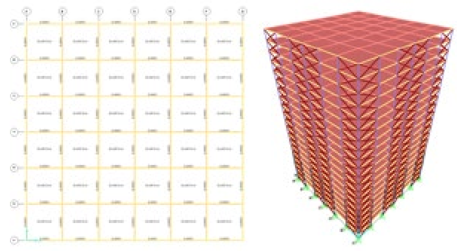
Where; E_K is the relative motion energy of the building system known as kinetic energy, E_D is the energy consumed by modal damping known as damping energy, E_E is the energy consumed by recoverable elastic deformation known as elastic strain energy, and E_H is the energy consumed by plastic deformation known as hysteretic or inelastic energy (Merter and Uçar 2018).

3. Evaluation of Seismic Energy Demands

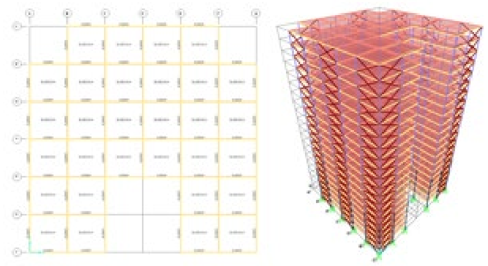
3.1. Description of structures

This article presents the results of two types of 20-story steel structures, consisting of symmetric and asymmetric plans designed with rigid beam-to-column connections. Both types of buildings consist of a 6-span with a 6 m distance, and the normal floor height of the building is designed as 3 m. The plan dimensions are 36 m wide and 36 m long.

The SAP2000 (SAP 2000) and PERFORM 3D (PERFORM CD) were used in this study, for modeling and analysis. By considering the SAP2000 program as a more suitable program for modeling high-rise buildings, it is chosen to design the models. The PERFORM 3D program is one of the most powerful software for nonlinear analysis and functional design of structures. In this case, the non-linear analyses were examined in the PERFORM 3D program to evaluate accurate and real results. In both models, European type Box-profile for columns, I-profile for beams, and UNP profile for braces were selected. The profile section of the columns was chosen as Box 380x380x40, Box 340x340x40, and Box 300x300x30 for the first five floors, 6-10 floors, and the last 10 floors, respectively. Also, the profile sections of the beams and braces were selected as ILS600 and DUBUNP 260, respectively (EUROCODE 3). The material properties were considered with values of minimum yield stress f_y equal to 355MPa for all the columns, 275MP for all the beams and braces, values of expected yield stress f_{ye} equal to 390.5MPa for all the columns, 302.5MPa for all the beams and braces. For nonlinear analysis, the nonlinearity property is defined for the steel material. kinematic hardening tangential elasticity modulus E_T = 171.96kN/cm2 strain at maximum tensile stress, ε was chosen as 0.10, as recommended in TS648 (Saafan 1965; Bozkurt et al. 2022). When creating a geometric model, the design variable and the dimensions are defined parametrically (Sathyamoorthy 2017). The composite material of the floor with a 12 cm thickness design as membrane type, and used in the flooring system is C25/30 in all stories. The Young's modulus (E) and the Poisson's ratio were also assumed to be 210 GPa and 0.3, respectively. This study was carried out considering that the column and beam connections in the buildings will transfer moments to each other. It is accepted that there are rigid connections in the structure subject to examination. Column-beam connections are designed to transfer moments to each other with the help of welds and bolts. The floors are also assumed to have rigid diaphragm movement in the horizontal plane. In the design of steel frames, the response modification factor R, the system over strength D, and the occupancy importance of structure I, were taken as 8, 3, and 1 respectively. Fixed loads including element weights on the floors are taken as 5.5 kN/m^2, and live loads as 3 kN/m^2. Fixed loads including element weights, live loads, and snow loads on the roof floor are taken as 5.5 kN/m^2, 1.5 kN/m^2, and 1.55 kN/m^2 respectively. Figure 2 shows the SAP2000 finite element model of the steel structures (SAP 2000).



(a)



(b)

Figure 2. Finite element model of the 20-story building; a) symmetric model, b) asymmetric model

The values of 39.905987° latitude and 41.191118° longitude on the map of Turkey show the province of Erzurum and the location of the structures. The area where the structure is located is considered to be in the ZB local area class (TBEC 2018). The design of structures is done according to the Turkish Building Earthquake Code 2018 and Eurocode 3 (TBEC 2018; EUROCODE 3). In the TBEC 2018, earthquake levels are given according to the probability of exceeding them in 50 years. For performance analysis, the earthquake ground motion level was chosen as DD-1, an earthquake with a 2% probability of being exceeded in 50 years (recurrence period is 2475 years). It is assumed that the buildings are in a region with seismic parameters as S_s and S_1 which are 1.711g and 0.457g, respectively (TBEC 2018). SDS and SD1 are obtained as 1.540g and 0.366g for the design spectral acceleration coefficient, respectively (TBEC 2018).

3.2. Modeling of the structures

Varying views, directions, and structures floors were designed in the PERFORM 3D (PERFORM 3D). The support type of the columns was defined as fixed support. Also, the beam-column connections are considered as rigid connections. A fully rigid diaphragm has been applied to the system to ensure that the structure can transfer the loads to other bearings without bending and twisting under any horizontal load. Earthquake calculations assumed that the centers of mass and stiffness coincide (rigid diaphragm). So the diaphragm is defined separately for each floor and was accepted as a rigid diaphragm.

When the structure is exposed to an earthquake, the energy must be dissipated successfully. Inelastic damage may occur in the structure during this consumption process, but collapse should never happen. However, in severe earthquakes, large displacements in the structure are inevitable. Therefore, it is necessary to provide sufficient rigidity to the structure. The displacement between floors can be limited by identifying special drifts for the structures. In the PERFORM 3D, the simple drift was defined between certain points of the relevant floors. Withal, the distortion drift can also arise as a result of varying column loads. Especially, the distortional drift effect is defined within the unbraced frames, but not necessary for the braced ones. Thus, in each model, the distortion drift limitations are made between each floor in braced frames.

As a result of the effect of earthquake load on the structure, axial and shear forces are observed in the cross-sections. The cross-section can be useful for controlling the behavior of a structure. Also, the total load distribution between floors becomes important when a structure has more than one horizontal load-carrying system. For this reason, we can determine a cross-section for each floor in steel structure frames to compare all force bearded on these sections. In PERFORM 3D, the cross-sections are defined for all columns, beams, and braces on the floors in the whole structure. To identify the behavior of structural members, component properties must be defined in steel structures. The elastic or inelastic behavior of the structural members was determined by defining materials, cross sections, strength sections, and component properties in the PERFORM 3D.

The deformation-controlled and force-controlled actions that are expected or not expected to undergo nonlinear behavior in response to earthquake shaking need to be determined by evaluating the behavior of structural members. Buckling controls for columns need evaluating first. Based on ASCE 41, columns with an axial demand-to-capacity ratio greater than 0.5 (PUF/PCL>0.5) are classified as force-controlled (FC), while columns with PUF/PCL ratio less than 0.5 (PUF/PCL<0.5) are classified as deformation-controlled (DC). To determine the actual behavior of the columns, the behavior is defined as deformation controlled first. According to the results obtained from nonlinear static pushover analysis, the columns with capacity utilization larger than 0.5 (PUF/PCL>0.5) is defined as force-controlled (FC) especially in braced frames. Other columns remained as deformation-controlled (DC) elements. In addition, structural members were defined as FEMA beam and FEMA column with determined end zone in the end of the elements as presented in Figure 3.

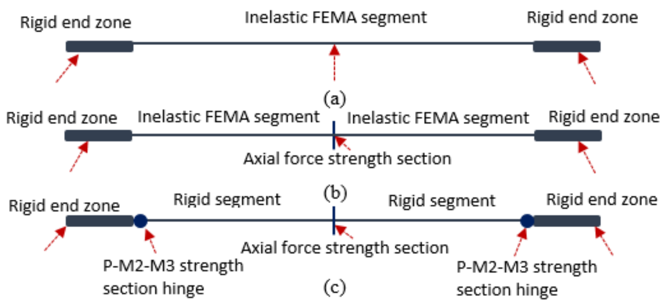


Figure 3. The end zone properties for structural members; (a) beams, (b) DC columns, (c) FC columns

4. Analysis of the Models

The stability of the structural system was checked under increased earthquake effects, and the condition that the columns were stronger than the beams was verified by considering the beam-column combination with a weakened beam cross-section. In this section, applied in symmetric and asymmetric models, linear analysis, non-linear static analysis (pushover), response spectrum and non-linear time history analyzes are explained.

4.1. Linear static analysis and design evaluation of the structures

First, the models were analyzed under the action of the gravity load. According to the modal results obtained from the static analysis made in the PERFORM 3D program, the first mode of the structure was obtained as 1.568 second in the symmetric model, while it was obtained as 1.916 second in the asymmetric model. Since the steel structure under investigation has 20 floors, the first mode period is expected to be long (greater than 2 seconds). Since the profiles forming the load-bearing columns in the structure have a high moment of inertia in both directions, there was no significant damage. In the asymmetric model an increase in the structure mode values has been observed as 22.19% compared to symmetric model.

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The symmetric and asymmetric 20-story steel structures, are designed based on Eurocode 3-2005 (EUROCODE 3) by using SAP2000 (SAP 2000) for the most critical load combinations, including earthquakes. According to the design results, for both symmetric and asymmetric model, it was determined that the center columns of the first 5 stories were exposed to the most intense stress and suffered more damage than the other ones. According to the design results it has been concluded that the structural elements can withstand the bending moment, shear force and axial loads against the most critical (inadequate) load combinations and that all elements are safe.

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4.2. Nonlinear static pushover analysis

According to the results obtained from the linear static analysis, nonlinear static pushover analysis was executed in PERFORM 3D (PERFORM 3D). Static pushover analysis was carried out by defining P-M2-M3 plastic hinges to the columns and M3 to the beams. Plastic joints are assigned to both ends of the elements at a distance of 0.5h. Here h indicates the section height of the building element. In static pushover analysis, the 1st mode is defined for the X direction, and the 2nd mode is defined for the Y direction. As for the vertical load, the G+nQ combination is given non-linearly. The FEMA 237 seismic force distribution protocol is used for pushover analysis. As a result of the analysis, the reference drift in both symmetric and asymmetric buildings was observed. Thus, the maximum displacement at the top point of the structure, in the symmetric model was obtained as 0.64 meters relative to the reference drift ratio point of 0.01064, and in the asymmetric model was obtained as 0.68 meters relative to the reference drift ratio point of 0.01159. The pushover analysis results showed that the symmetric model made 5.18% more displacement and 8.92% more drift than the asymmetric model when the consecutive pushing step was terminated (Figure 4).

In performance-based seismic analysis, the definition of limit states is necessary to identify different levels of damage, which can be described by material strains, rotations, or displacements occurring on the structural members. These limit states are determined as immediate occupancy (IO), life safety (LS), and collapse prevention (CP) ([TBEC 2018; ASCE 41-43; FEMA 273). At the IO level, the structure has minor defects in non-structural members. At the LS level, the structure is designed to have residual stiffness and strength in all stories, with permanent drift. in the CP level, the structures have minimal stiffness and strength in all stories, but columns and walls remain working. In this situation, non-structural components are damaged and the building is near to collapse (Merter et al. 2012a). The acceleration-displacement capacity curve of structures by defining the performance levels is presented in Figure 4.

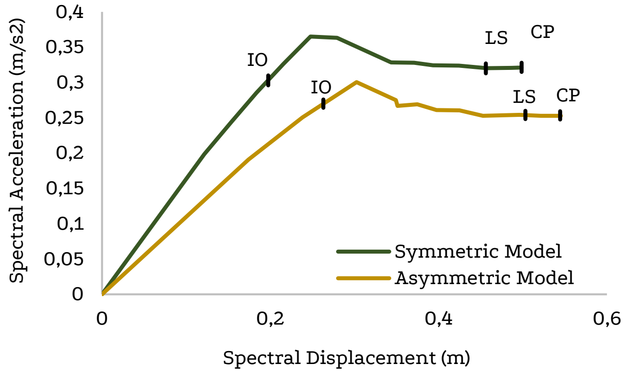


Figure 4. Acceleration – displacement capacity curve of the symmetric and asymmetric steel structures

Based on the horizontal elastic design spectrum diagram depending on the natural vibration period, the base shear–reference drift curves obtained from the nonlinear static pushover analysis are checked to calculate the target drift which is determined as a structure performance point to show the actual drift ratio limitation of steel structures under shear loads. As a result of nonlinear static pushover analysis, the target drift value for the symmetric and asymmetric building is defined as 0.0038 and 0.0046 as in Figure 5, respectively. This shows that the symmetrical building model is the most effective in resisting earthquake effects by causing permanent deformation.

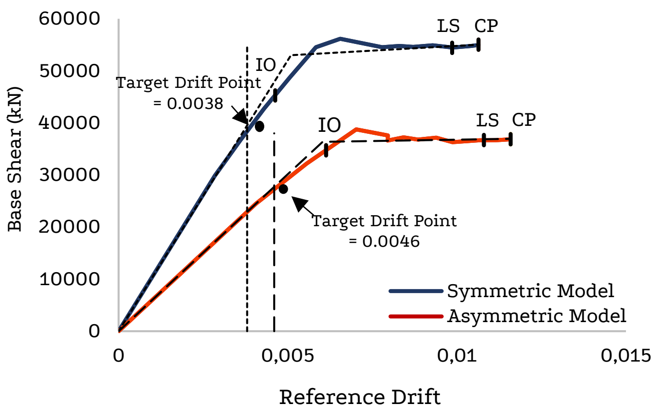


Figure 5. Base shear-reference drift curve with defining bilinear approximation and determining target drift for symmetric and asymmetric model

The limited damage, controlled damage, and migration prevention performance limit values were determined according to TBEC 2018. Plastic rotation values defined in TBEC 2018 were taken as a reference for damage detection. The damage limit value for limited damage, controlled damage, and migration prevention damage limit value is determined according to TBEC 2018 5C.1. When the static pushover analysis was performed, the building was first pushed up to 0.1 value as the allowable drift ratio. According to the mode period of the building and the results obtained after pushing the structure up to 0.1 drift ratio value, plastic hinge formation in the columns is detected, and the structure remains between controlled damage (life safety) and migration prevention damage (collapse prevention) boundary conditions. However, after finding the target drift point of the building from static pushover analysis results, we pushed the building according to the obtained target drift point for the second time and subjected it to nonlinear static analysis. According to the results of the applied second static pushover analysis, it was determined that the hinges formed in the columns remained elastic, and the structure remained between limited damage (immediate occupancy) and controlled damage (life safety) boundary conditions. According to the design scale, the hinge formed and the structure deformation are presented in Figure 6 for all deformation states of limit state groups. Thus, it was concluded that the columns had suffered controllable damage and the structure performed well.

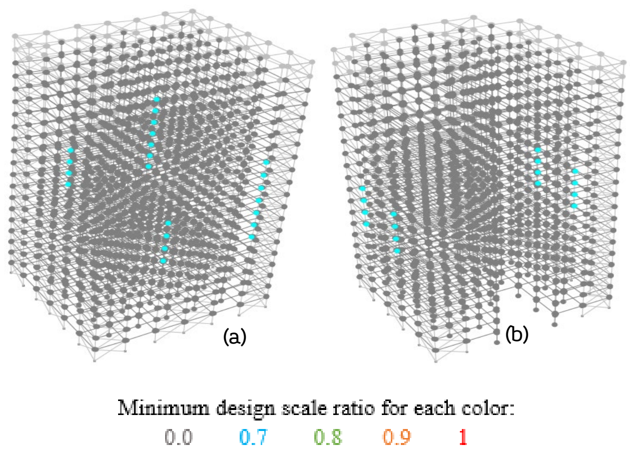


Figure 6. Hinge formation in columns of structures;(a) symmetric model (b) asymmetric model

In addition, the behavior of brace elements as tension and compression under the nonlinear static pushover analysis have been obtained. The axial force-axial strain hysteresis diagram of brace elements is presented in Figure 7, which is the buckling of the brace in tension and compression. It was determined that the tensile and compressive forces of the asymmetrical structural model were obtained at 2690.22 and 2346.32 kN, respectively. It is also obtained from Figure 7 that the final deformation of the structure under the influence of tension and pressure is 0.00314 and 0.00244, respectively. As expected, the steel structure bearing system behaved more rigidly under the effect of tension than under compression. As a result of nonlinear static pushover analysis, the beam's maximum moment and rotation values were obtained. The moment-plastic rotation hysteresis diagram and deflected shape of a beam element showing nonlinear behavior is presented in Figure 8. By evaluating the results, the maximum plastic rotation of a beam element was obtained as 0.01258 radians in response to a moment of 835.3314 kN.m in the symmetric model. Additionally, it was observed that the structure model reached the yield zone when it rotated around 0.00453.

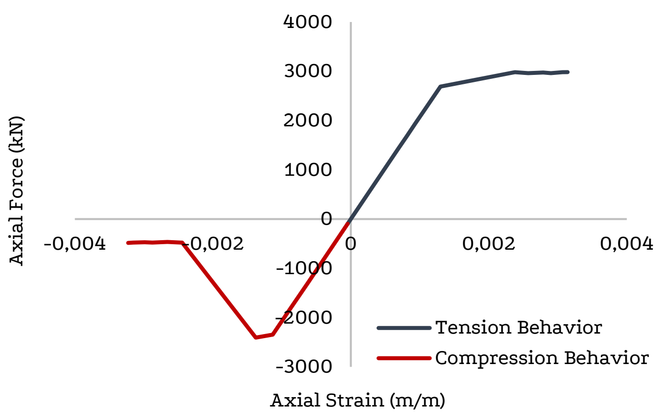


Figure 7. Axial force – axial strain diagram of brace element for symmetric model

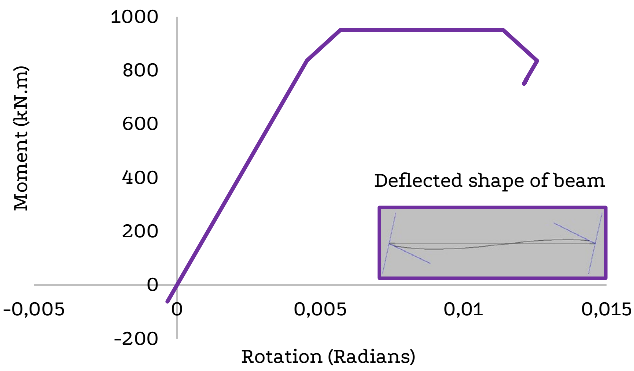


Figure 8. Moment-plastic rotation curve of beam element for symmetric model

As a result of nonlinear static push-over analyses for the symmetric and asymmetric steel structures, the story drift ratio values were obtained. By evaluating the analysis results, it has been confirmed that the maximum story drift occurs for all the low stories between 3-5 story building models. According to the TBEC 2018 specifications, it is known that the value of the story drift ratio of steel structure should be equal to or less than 0.008 (TBEC 2018; AFAD 2023). The maximum drift values given in Figure 9, obtained according to the non-linear analysis results, were calculated according to TBEC 2018 criteria, symmetric and asymmetric structures' largest relative story drift values were obtained at 0.00320 and 0.00349, respectively. It was determined that the relative story drift values in both building models were below the relative story drift limit values defined in TBEC 2018. However, by comparing both models, an increase in the average story drift ratio obtained about 9.5% for asymmetric buildings according to the symmetric building model. According to the results obtained from the nonlinear static pushover analysis, the increase in story drift ratio values in the asymmetric building is a torsion effect that occurs due to the irregular situation of the building in the plan. Figure 9 shows the story drift ratio value for two systems between 0.5% and 2.5% for all building systems.

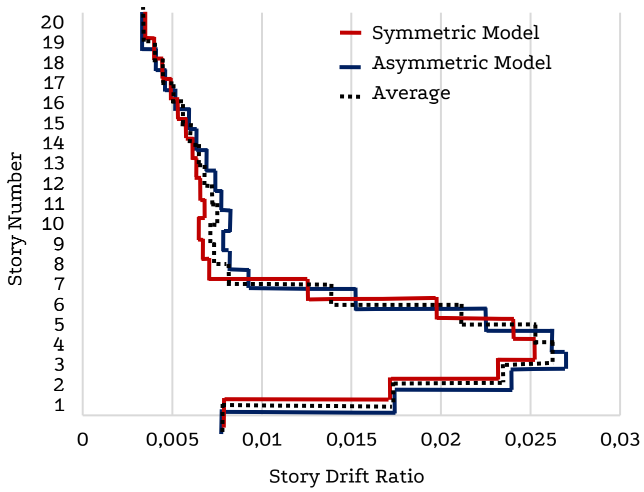


Figure 9. Maximum story drift ratio of nonlinear static pushover analysis for symmetric and asymmetric model

4.3. Response spectrum analysis evaluation of the structures

By obtaining the natural vibration period of the structure, Response Spectrum Analysis can be performed by determining the acceleration to the structure. After performing the Response Spectrum analysis, the displacement at the top of the structure was obtained for symmetric and asymmetric models. When the response spectrum analysis results of both types of buildings were compared, it was observed that the top displacements of the building in X, Y, and Z directions for the asymmetric model increased by 17%, 8%, and 3% compared to the symmetric one, respectively. As a result of the Response Spectrum Analysis, the maximum story drift ratios were found to be 0.0015 and 0.0019 on the 11th floor for the symmetric and asymmetric models, respectively. According to the results obtained from the analysis, it was determined that the percentage of story drift change on the 11th floor increased by about 26.5% for the asymmetric model compared to the symmetric model.

4.4. Nonlinear dynamic time history analysis with the energy-based method

The building design must withstand low-intensity earthquakes and exhibit non-linear behavior during service loads. The energy in the light earthquakes is dissipated (consumed) as elastic and damping energy in the structure. In moderate and more severe ground motions, a large amount of the entering energy is dissipated (consumed) as hysteretic and plastic energy (non-linear behavior). The energy-based design compares the total amount of dissipated energy by the structural members by showing elastic and inelastic behavior with the energy requested during the earthquake. Thus, when structures are exposed to the effects of severe earthquakes, they are damaged only at a level that can be repaired before collapse occurs. In general, the earthquake's intensity is based on the maximum ground acceleration of the ground movements. Earthquake energy generated due to maximum ground acceleration can be distributed in the structure and cause damage (Kato and Akiyama). In this study, earthquake records were obtained from the PEER Ground Motion Database (PEER Center) to evaluate structures under the earthquake ground motions. To find more accurate results from earthquake structure analysis, an earthquake magnitude of at least 5 and at most 8 Richter was selected. Eleven earthquake acceleration-time graphs of the steel structures created in the study, and selected to be used in time history analysis that taken into account for non-linear behavior, are shown in Figure 10. Some characteristics of the earthquake acceleration records used in the nonlinear dynamic time history analysis are given in Table 1. Where; NGA, is the PEER earthquake record number, Mw, is the moment magnitude of the earthquake, VS30 is the average shear wave speed calculated from the first 30 meters of the ground profile, PGA, PGV, and PGD, is the maximum ground accelerations, velocity and displacement of the earthquake acceleration record, respectively. In addition, the time duration is the duration of the earthquake that happened and the scale factor is the scaling factor of earthquake records obtained using a simple scaling method. As shown in the table, when the scale factors for the symmetric and asymmetric models are compared, a 12% reduction is achieved when the first mode period of the asymmetric model building is greater than the symmetrical one.

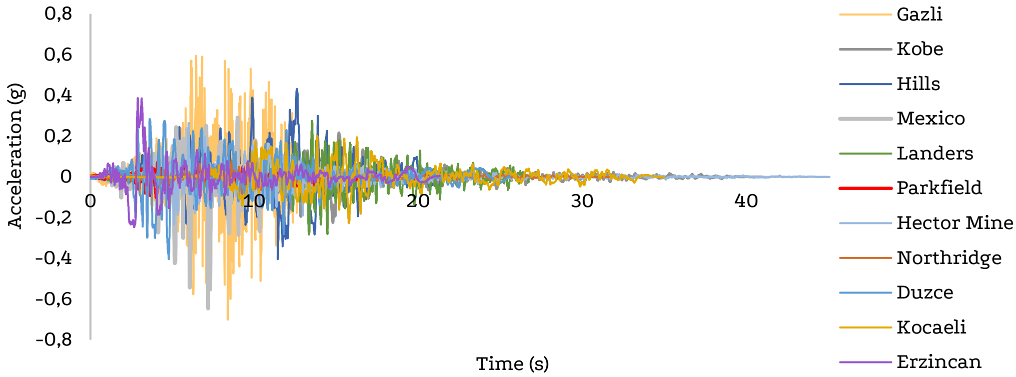


Figure 10. Earthquake acceleration records

Table 1 Basic characteristics of earthquake ground motions from the PEER database

Event & Year of Earthquake	NGA#	Time Duration (s)	M_w	V_{S30} (m/s)	PGA (g)	PGV (m/s)	PGD (m)	Scale Factor	
								Symmetric	Asymmetric
Gazli & 1976	126	13.51	6.8	259	0.8639	0.6765	0.2733	0.5785	0.5090
Mexico & 1980	265	24.53	6.33	471	0.6453	0.3358	0.0661	0.7745	0.6815
Hills & 1987	723	22.35	6.54	348	0.4318	1.3428	0.4617	1.1575	1.0186
Landers & 1992	848	28	7.28	352	0.4171	0.4341	0.1820	1.1985	1.0546
Erzincan & 1992	821	21.31	6.69	352	0.4961	1.0714	0.3199	1.0075	0.8866
Northridge &1994	1019	32	6.69	425	0.0864	0.0944	0.0319	5.78	5.0864
Kobe & 1995	1116	40.96	6.9	256	0.2333	0.3132	0.0971	2.1425	1.8854
Duzce & 1999	1605	25.88	7.14	281	0.5149	0.8423	0.4969	0.971	0.8544
Kocaeli & 1999	1176	35	7.51	297	0.3218	0.7188	0.6232	1.5535	1.3670
Hector Mine & 1999	1787	45.31	7.13	726	0.3281	0.4477	0.1976	1.5235	1.3406
Parkfield & 2004	4125	21.32	6.0	232	0.1030	0.0493	0.0229	4.853	4.2706

According to the Turkey Building Earthquake Regulations (TBEC, 2018) "the resultant horizontal spectrum will be obtained by taking the square root of the sum of the squares of the spectra of the two horizontal components of each earthquake record set selected for three-dimensional calculation. The amplitudes of earthquake ground motion components will be scaled according to the rule that the ratio of the average of the resultant spectrum of all selected records between the periods 0.2Tp and 1.5Tp to the amplitudes of the design spectrum defined according to 2.3.4 or 2.4.1 in the same period should not be less than 1.3. This period range may vary for insulated buildings (See 14.14.4.2). Both horizontal components will be scaled with the same scale coefficients" (TBEC 2018).

According to the TBEC (2018), the ZB local soil class is accepted for the symmetric and asymmetric steel frame structure models. The selected earthquake acceleration records were scaled and used with a simple scaling method to be compatible with the design acceleration spectrum created based on the DD-1 earthquake level (with a 2% probability of exceedance in 50 years) and the ZB local soil class for the Erzurum location. Comparison of scaled average spectrum curves of earthquake records with the design spectrum, taking into account the ground conditions in the specifications, for symmetric and asymmetric models are presented in Figure 11. As shown in the graph, the scaling of the response spectrum curves of the acceleration records is based on the range of 0.2T and 1.5T depending on the building first mode period. As shown in Figure 11, the earthquake spectrum acceleration values between $0.2T_p$ and $1.5T_p$ meet the horizontal elastic acceleration spectrum values in the same range. Since the dominant period value (T_p) of symmetrical and asymmetrical structural situations is quite large, earthquake spectrum scaling was done in long periods.

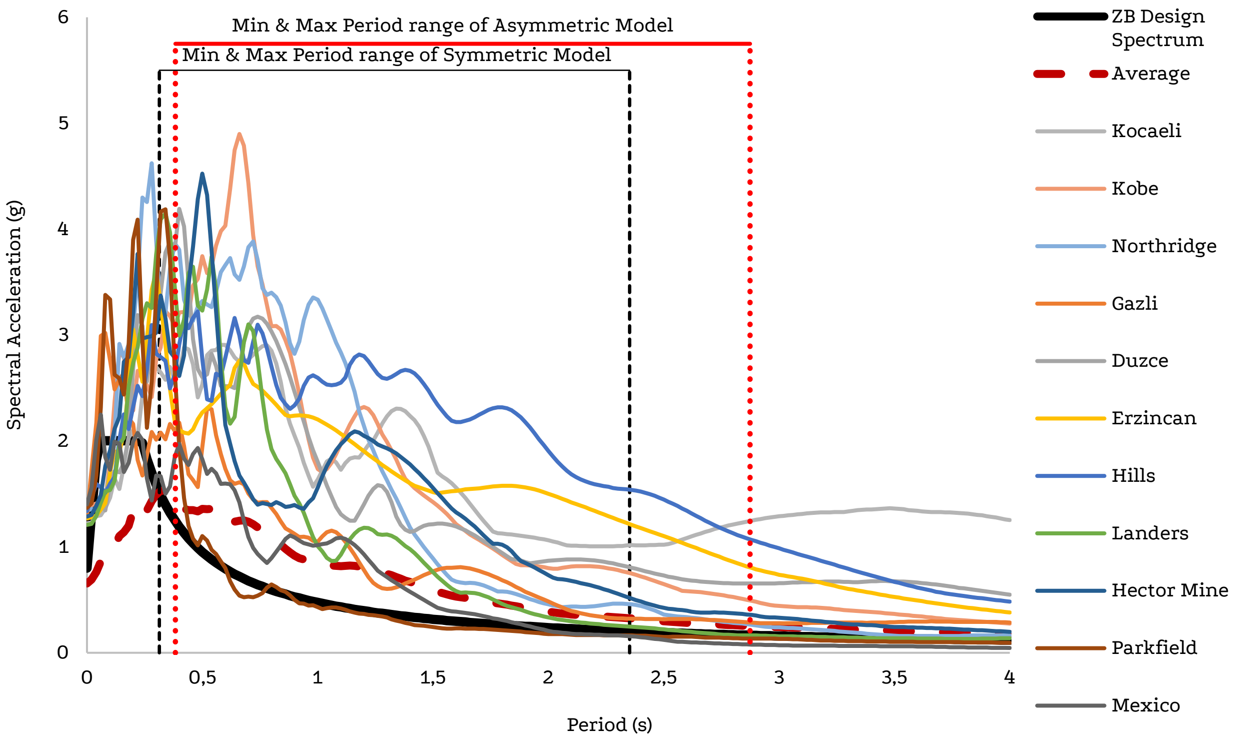


Figure 11. Acceleration spectra of earthquake ground motions and design acceleration spectrum for symmetric and asymmetric model

5. Nonlinear Dynamic Time History Analysis Result

According to the results of non-linear dynamic time history analysis using the energy-based method, the limit performance level of both symmetric and asymmetric models was found between limited damage (immediate occupancy) and controlled damage (life safety) values which shows the good performance of structures under eleven scaled earthquake acceleration records.

The floor horizontal displacements, time-dependent change of the base shear force, and stress-strain (σ - ε) relationships can be obtained from nonlinear dynamic time history analysis for selected earthquake acceleration records of 20-story steel structures. In addition, the time-dependent change of the total energy, kinetic energy, damping energy, elastic strain energy, and hysteretic energy (dissipated inelastic energy) entering the building systems due to the effect of earthquake ground motion can be evaluated.

Time-dependent displacements (δ-t) of symmetric and asymmetric models were obtained from nonlinear dynamic time history analysis performed for 11 earthquake records scaled according to the top of the building. As shown in Figure 12, according to the results obtained from the dynamic analysis, the maximum time-dependent displacement was found for the Gazli earthquake as 0.2939 meter in 12.17 second and for Landers earthquake as 0.3858 meters in 27.9 second for symmetric and asymmetric models, respectively. It has been determined that the structure with an asymmetric model in the time domain has 37.92% more dynamic displacement than the symmetric structure. It is thought that different earthquakes, occurring largest displacement at various time steps are due to the earthquake's orientation according to the structure, exposure to the source, and acceleration characteristics.

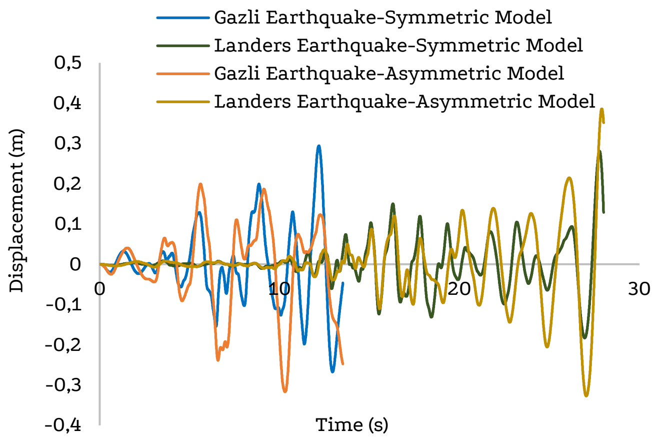


Figure 12. Displacement-time (δ-t) graph under Gazli and Landers earthquake ground motions for the top point of the symmetric and asymmetric models

Base shear is the maximum lateral force that will occur due to seismic ground motion at the base of a structure. Time-dependent base shear force diagrams of symmetric and asymmetric models were obtained from nonlinear dynamic time history analysis under scaled earthquake acceleration records. The maximum base shear force-time diagram has been obtained for the Erzincan and Kocaeli earthquakes, shown in Figure 13.

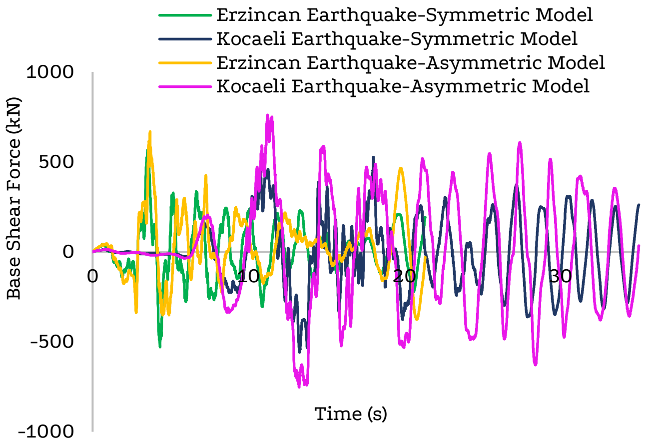


Figure 13. Base shear force-time (V_t-t) graph under Erzincan and Kocaeli earthquake ground motions for the symmetric and asymmetric models

According to the results, the maximum base shear force value for the Erzincan earthquake was 565.56 kN in 3.52 second, and for the Kocaeli earthquake as 760.95 kN in 11.2 second for symmetric and asymmetric models, respectively. It has been determined that the largest base shear force in the structure with an asymmetric model in the time domain is 25.67% greater than the structure with a symmetrical model. This situation is attributed to higher earthquake ground accelerations due to the structural period of the asymmetric building being larger than the period of the seismic building.

As a result of non-linear dynamic time history analysis using the energy-based method, the behavior of structural members for 11 earthquake acceleration records was examined. The moment-rotation and stress-strain values of the structural members affected by the earthquake were obtained and hysteresis curves of beams and braces were evaluated for the Gazli and Landers earthquake. According to the results, in both symmetric and asymmetric models, when the beams in the middle floors were exposed to more earthquake effects, the most deformation was observed for the braces on the second and third floors. The moment-rotation hysteresis curve of the beam on the 9th floor and the stress-strain hysteresis curve of the braces on the 2nd floor is shown as an example of the symmetric building in Figure 14 and Figure 15, respectively. When looking at the moment-rotation values, which are decisive in performance evaluation, in Figure 14, it was determined that the Gazzi and Landers earthquakes created the largest values in structures with symmetrical and asymmetric models, respectively. Since the moment-rotation values are calculated depending on the displacement values, it is concluded that it is meaningful that the maximum effect of the Gazzi and Landers earthquakes on the displacement values also creates on the moment-rotation values. Symmetrical and asymmetrical models reached yield moment and rotation values at the same time. However, the ultimate rotation values of structures with symmetrical and asymmetric models were 0.0092 and 0.00695. This situation showed that, as in the static push-over analysis, the symmetric model can make more permanent deformation than the asymmetric model. It was obtained from the moment-rotation curves shown in Figure 14 that the symmetric model achieved 24.62% more permanent deformation than the asymmetric one.

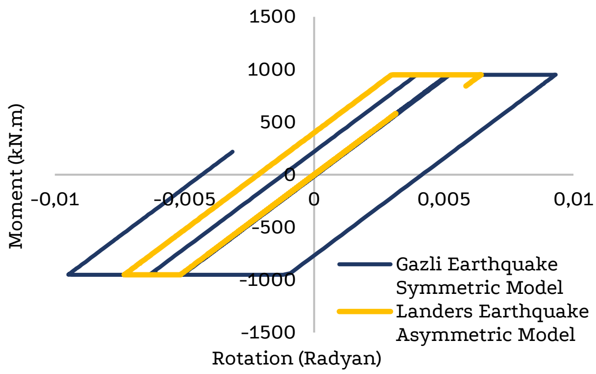


Figure 14. Moment–rotation hysteresis curve of the beam for Gazli and Landers Earthquake

It has been determined that the earthquakes that create the highest axial force-strain values in symmetrical and asymmetrical building models, as well as the displacement and the moment-rotation values, are for the Gazli and Landers earthquakes. An axial force of 2487.52 kN and a corresponding ultimate strain of 0.00120 in the symmetrical structure was achieved for the Gazli Earthquake. An axial force of 1941.63 kN and a corresponding final strain of 0.000937 in an asymmetric structure was achieved for the Landers Earthquake. The energy-based analysis implied that the symmetrical model had 21.945% more axial load and 21.91% more deformation capacity than the asymmetric one.

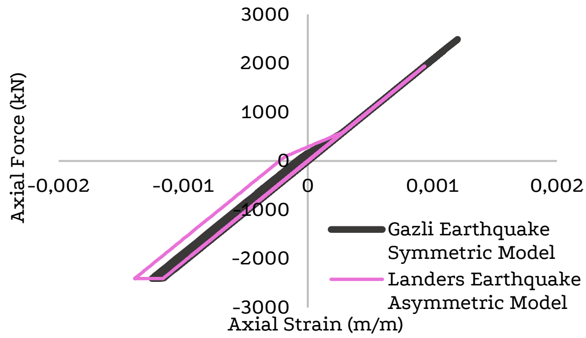


Figure 15. Stress-strain hysteresis curve of the brace for Gazli and Landers earthquakes

In addition, energy components - time graphs of the steel frames analyzed in the study were obtained. The time-dependent change of the total energy (EI) entering the structure due to the earthquake effect and the time-dependent changes of kinetic, damping, strain, and hysteretic energy types were obtained, which are the energy consumption of this entering energy by the system. Energy-time history graphs of the symmetric and asymmetric models are presented in Figure 16, Figure 17, and Figure 18 for Gazli, Northridge, and Erzincan earthquakes, respectively. In Figure 16, it was determined that the Gazli earthquake generated the greatest input energy in the symmetric model. However, the symmetric model absorbed more modal damping, strain, kinetic, and plastic energy than the asymmetric one. In Figure 17, more input energy was achieved for the asymmetric structure under the Northridge earthquake. Modal damping, strain, and kinetic energy, in the asymmetric structure absorbed more than the symmetric model. However, plastic energy absorption for the symmetric model achieved more than the asymmetric one. In Figure 18, results similar to the Gazli earthquake were obtained. In this situation, the symmetric model performed better than the asymmetric one in absorbing modal damping, strain, kinetic y, and plastic energy.

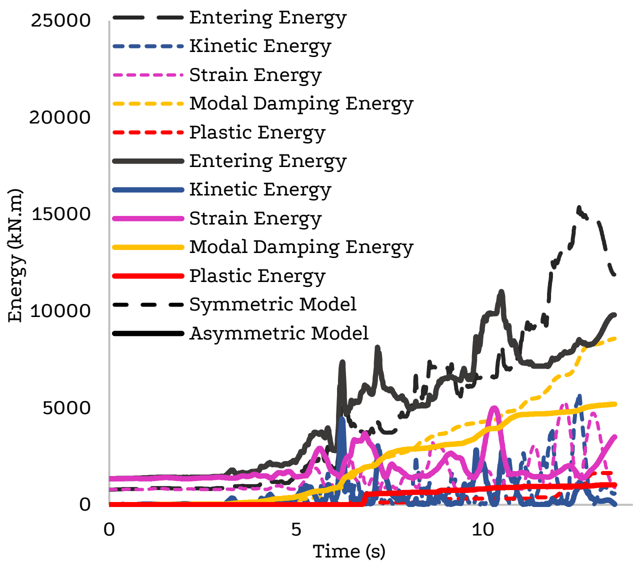


Figure 16. Energy components - time history (E - t) graph of the symmetric and asymmetric models under Gazli earthquake ground motion

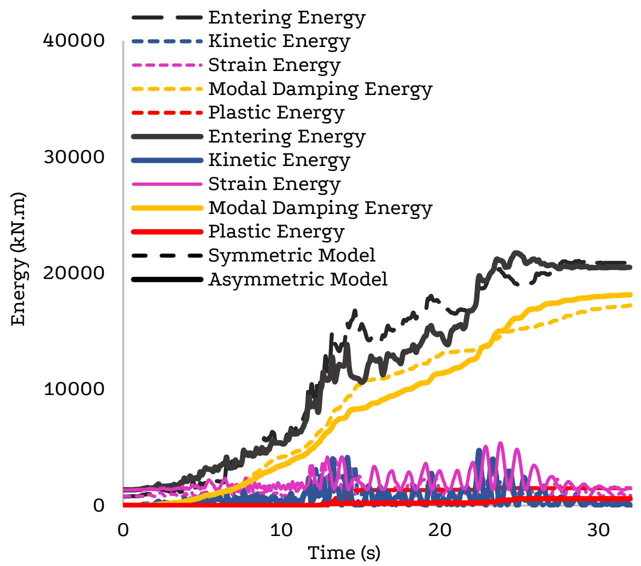


Figure 17. Energy components - time history (E - t) graph of the symmetric and asymmetric models under Northridge earthquake ground motion

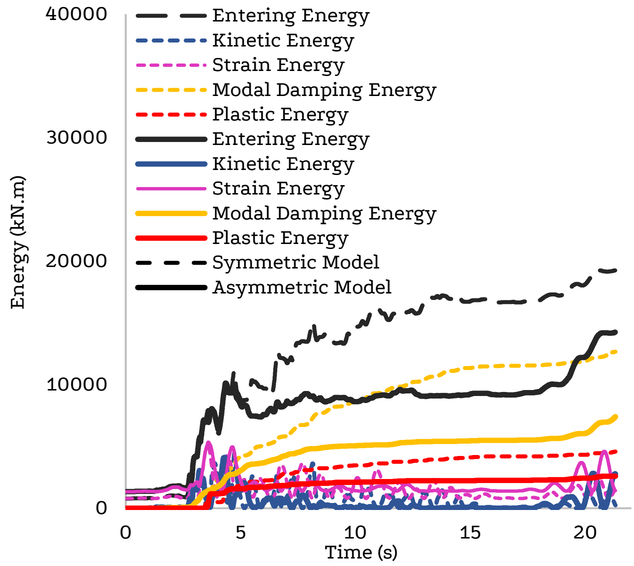


Figure 18. Energy components - time history (E - t) graph of the symmetric and asymmetric models under Erzincan earthquake ground motion

For both types of structures, the highest input energies were achieved for the Gazli, Northridge, and Erzincan earthquakes compared to the other earthquakes used in the study. The average input energy of these three earthquakes at the symmetric and asymmetric structures was approximately 18420 kN.m and 16030 kN.m, respectively. The average modal damping energy of symmetrical and asymmetrical structures against the earthquakes was 14431 kN.m and 10219 kN.m, respectively. For the symmetrical and asymmetrical structures, the strain energy was determined as 4333 kN.m and 4589 kN.m, respectively. The kinetic energy was obtained as 4411 kN.m for the symmetrical structure and 4223 kN.m for the asymmetrical structure. Finally, the plastic energies were calculated as 2514 kN.m and 1399 kN.m for the symmetric and asymmetric ones, respectively. In light of these obtained results, it was determined that the average earthquake input energy in the symmetric model was 12.975% more than the asymmetric one. It was concluded that the symmetric model absorbed 29.18%, 4.25%, and 44.32% more energy than the asymmetric in absorbing modal damping, strain, and plastic energy, respectively. However, it was also determined that the asymmetric model absorbed 5.59% more earthquake energy than the symmetric model in the elastic region due to irregular settlement.

When the seismic energy components obtained from time history analysis of selected earthquake ground motions for scaled earthquake acceleration records are examined; the entering energy to the structural system for each earthquake and the distribution rates of this energy assumptions in the structural systems are varied. This shows that each earthquake is unique and the structure reflects this situation. The energy analysis results were taken into account of several assumptions made for the structure and ground. It is necessary to investigate the dynamic properties of the structure, the characteristics of earthquakes, and the geological formation for detailed future work.

As a result of nonlinear dynamic time history analysis, the total energy entering the structures and the dissipated hysteretic energy values obtained during the ground motion at the structures are compared for both symmetric and asymmetric models. The seismic energy demand parameters; the total energy input and hysteretic energy amounts normalized by mass are presented in Figure 19 and 20 for each building type, respectively. For the symmetric building model; the maximum total energy input, E_I/m, is obtained as 2.09 m^2/sec^2 and 1.43 m^2/sec^2 for the Kocaeli earthquake for symmetric and asymmetric models, respectively. Also, the dissipated hysteretic energy for all models, E_H/m, is obtained as 0.33 m^2/sec^2 for the Erzincan and Kocaeli earthquakes. For both types of models, under 11 scaled earthquake acceleration records, the average total energy input value, E_I/m, is 1.43 m²/sn² and 0.99 m²/sec² for the symmetric and asymmetric models, respectively. In contrast, the average hysteretic energy, E_H/m, is obtained as 0.144 m²/sec² and 0.082 m²/sec². As a result, it has been determined that the different characteristics of earthquake ground motions affect energy demands. Also, the symmetric and asymmetric features of the building resulted in the structure having different energy absorption capabilities.

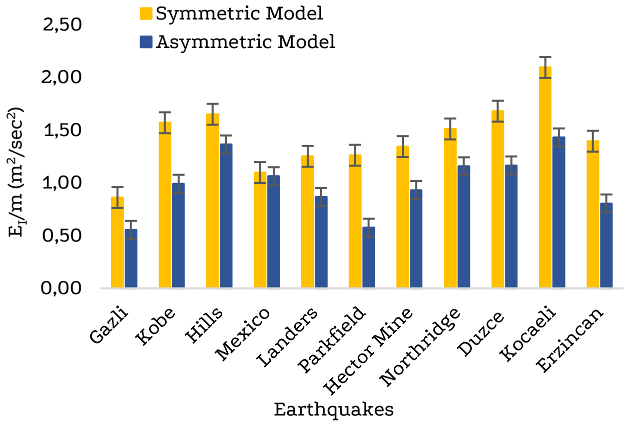


Figure 19. E_I/m parameter for the symmetric and asymmetric buildings

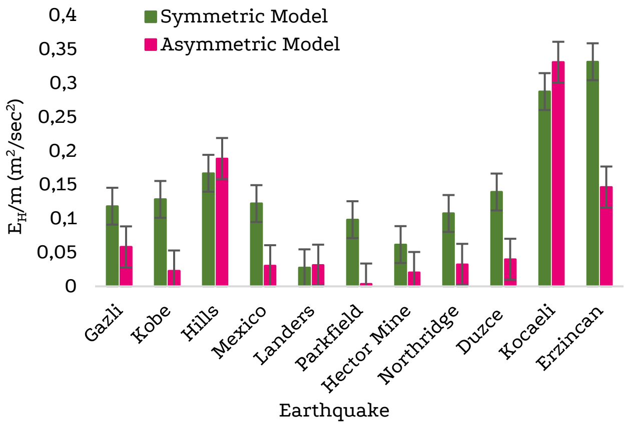


Figure 20. E_H/m parameter for the symmetric and asymmetric buildings

According to the dynamic analysis, the dissipated hysteretic energy during the ground motion period was obtained from the nonlinear time history analysis of the selected steel structure model for the effect of eleven earthquakes. The maximum dissipated hysteretic energies (E_p) of both the symmetric and asymmetric models were evaluated and presented in Figure 21. Thus, the maximum dissipated hysteretic energy was obtained as 4578 kN.m for Erzincan earthquake for the symmetric model, while it was obtained as 5859.6 kN.m for the Kocaeli earthquake for the asymmetric model. In terms of hysteretic energy, it was determined that the structure with the asymmetric model absorbed 21.76% more energy than the structure with the symmetric one. The irregularity of the asymmetric structure resulted in more flexibility when compared with the symmetrical one. This situation is explained by the fact that the hysteretic model is directly related to the flexible and plastic displacement of the structure.

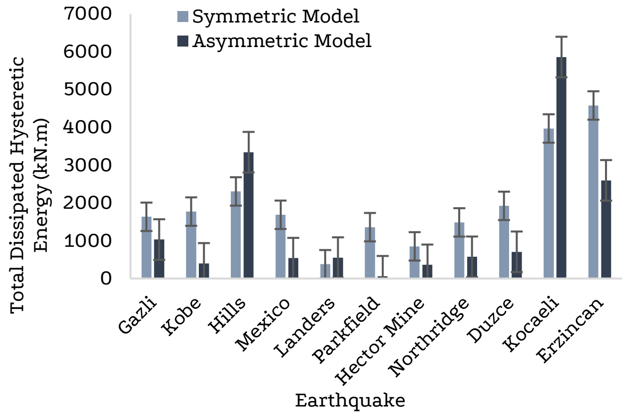
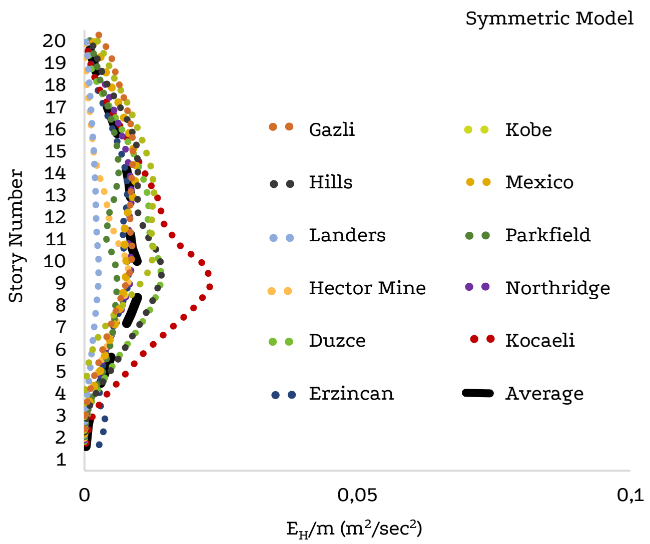
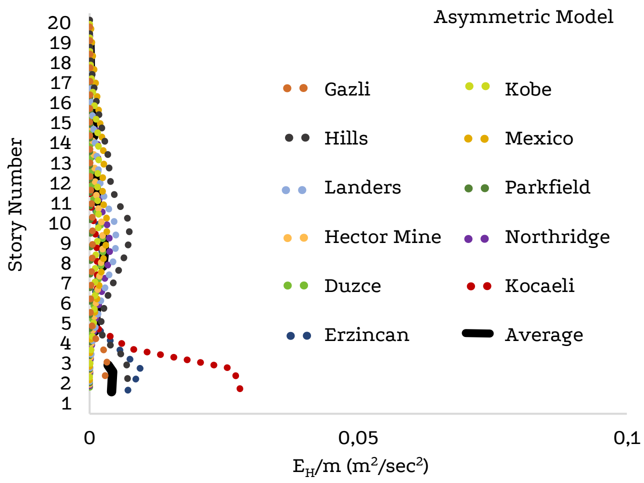


Figure 21. Total dissipated hysteretic energy values for symmetric and asymmetric models

The natural vibration periods of both models are close to each other, obtained from modal analysis, which are important parameters for hysteretic energy dissipation behavior. During nonlinear behavior, the hysteretic energy distributions occurred depending on the plastic deformation distributions among the structure. Although the 20-story building with different plan models has its own unique E_H/m distribution pattern, it has been observed that the distribution of hysteretic energy to each floor is not uniform, as shown in Figure 22. The hysteretic energy distribution of the models exhibited the normalized dissipation hysteretic energy according to mass, E_H/m, of symmetric and asymmetric models designed according to the conditions of relevant standards for considered earthquakes. The lowest value for E_H/m distribution was found on the upper floors. Furthermore, the highest E_H/m distribution was found for 8 and 9 floors under the Kocaeli earthquake.



(a)



(b)

Figure 22. Hysteretic energy distribution along the frames under the influence of earthquakes; a) Symmetric model, b) Asymmetric model

Based on the beam members with plastic deformation results from energy-based analysis, the average of dissipated hysteretic energy value, E_H/m, for the multi-story steel structure was determined across the floors. The resulting graphics are presented in Figure 23 for symmetric and asymmetric models. As seen in the graphs, the permanent deformation of the beams, obtained on the lower floors and reached the highest plasticization state in the middle of the floors, the plastic deformation in the beams decreased in the uppermost floors, and almost no plasticization occurred at the top floor and it was reached to zero. According to the energy-based analysis, it is understood in Figure 20 that the symmetric model has a more hysteretic damping energy mechanism than the asymmetric one. Figure 23 shows that the ability of a structure to change permanent shape is directly related to hysteretic energy. As shown, the greatest dissipated hysteretic energy, E_H/m, is found for middle floors as 0.011 m²/sec² in the symmetric building, which is found for lower floors as 0.006 m²/sec² in the asymmetric one.

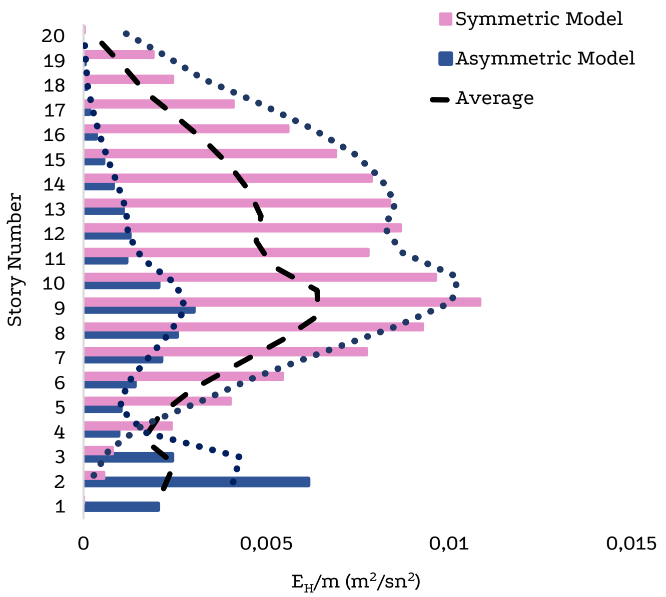
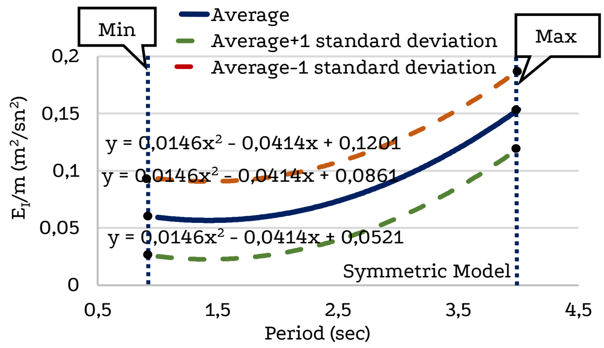


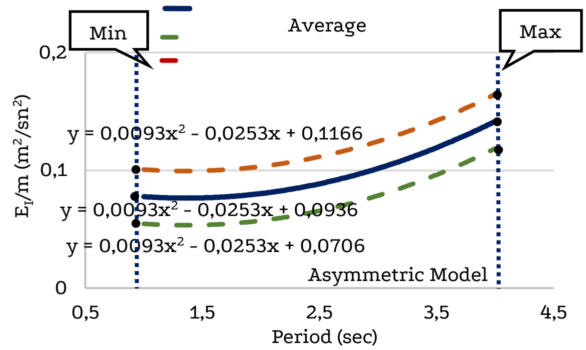
Figure 23. Average of hysteretic energy dissipation along the frames of symmetric and asymmetric buildings

The total energy spectrum, the total hysteretic energy spectrum, and the E_H/E_I spectrum are recommended to be used to estimate the total energy entering the structure and the ratio of total hysteretic energy to the total energy entering the structure at a 2% earthquake level with a probability of exceedance in 50 years which are presented in Figure 24 and 25, respectively. The changes of the energy spectra are given in 1 - 4 seconds. In the graphs, the limits of the total energy input are specified by adding standard deviation values with a trendline equation in which y is E_I/m or E_H/m or E_H/E_I values and x is period. The values of symmetric and asymmetric models are shown within the graphs.

When the energy spectra are examined, the results of using irregular plan shapes of steel structures that transmit moments by the earthquake ground motions show less total energy (E_I) input demand for the building. Likewise, it has been concluded that it has less energy absorption capability. Also, it has been observed that while the parameters E_I/m, which expresses the total energy input per unit mass of the building and E_H/m, which is the total hysteretic energy per unit mass of the building, in the range of 0.5-3.5 periods increase, in the range of 3-4 seconds started to decrease in the asymmetric model compared to the symmetric one. Thus, seismic energy demands were significantly affected due to the change in the natural vibration periods of the system.

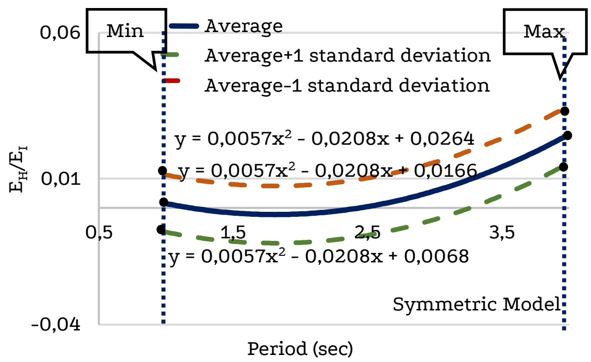


(a)

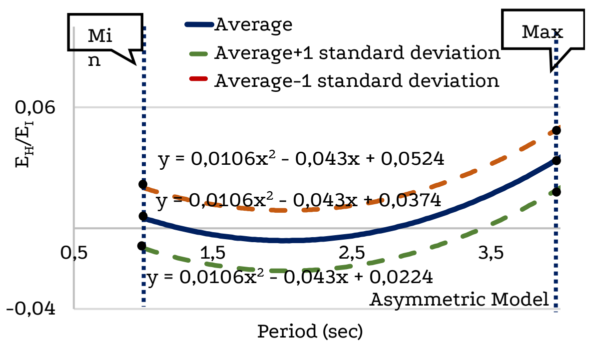


(b)

Figure 24. Total energy (EI/m) spectrum for 2% probability of exceedance in 50 years; a) symmetric model, b) asymmetric model



(a)



(b)

Figure 25. Hysteretic / Total energy spectrum for 2% probability of exceedance in 50 years; a) symmetric model, b) asymmetric model

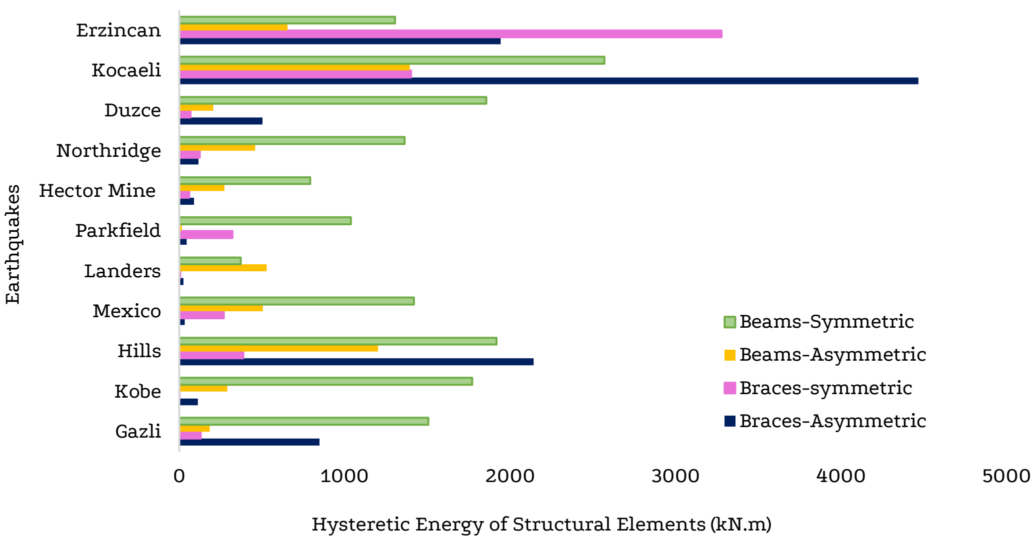
Using nonlinear dynamic time history analysis and an energy-based method, the amount of dissipated hysteretic energy of the structural members was obtained from the results. In this case, the total hysteretic energy dissipation of the beams and braces for the symmetric and asymmetric models was analyzed for eleven scaled earthquake records as in Figure 26. Furthermore, the total dissipation hysteretic energies of the columns are obtained zero or around zero not presented within the diagrams. By examining the results of energy-based analyses of eleven earthquakes, the columns in symmetric and asymmetric models showed elastic behavior during the earthquakes. However, beams and braces showed inelastic behavior.

As a result of the nonlinear dynamic time history analysis, it is determined that the earthquake characteristics and the symmetric and asymmetric nature of the structure, affect the energy dissipation capacity of the structural members. Generally, by considering the depth of the earthquake as an earthquake characteristic, by evaluating the energy absorption capacity of the beams, for example for the Kocaeli earthquake as a deeper earthquake, it was found that the symmetric building had the most hysteretic energy dissipation capacity at 2550 kN.m. The maximum dissipated hysteretic energy for the Kocaeli earthquake in the asymmetric building model was observed to be around 1400 kN.m. However, when a shallower earthquake is inspected, for example, the Hills earthquake, it can be seen that the symmetric building has an energy dissipation capacity of around 1900 kN.m, while the asymmetric building has an energy absorption capacity of around 1200 kN.m.

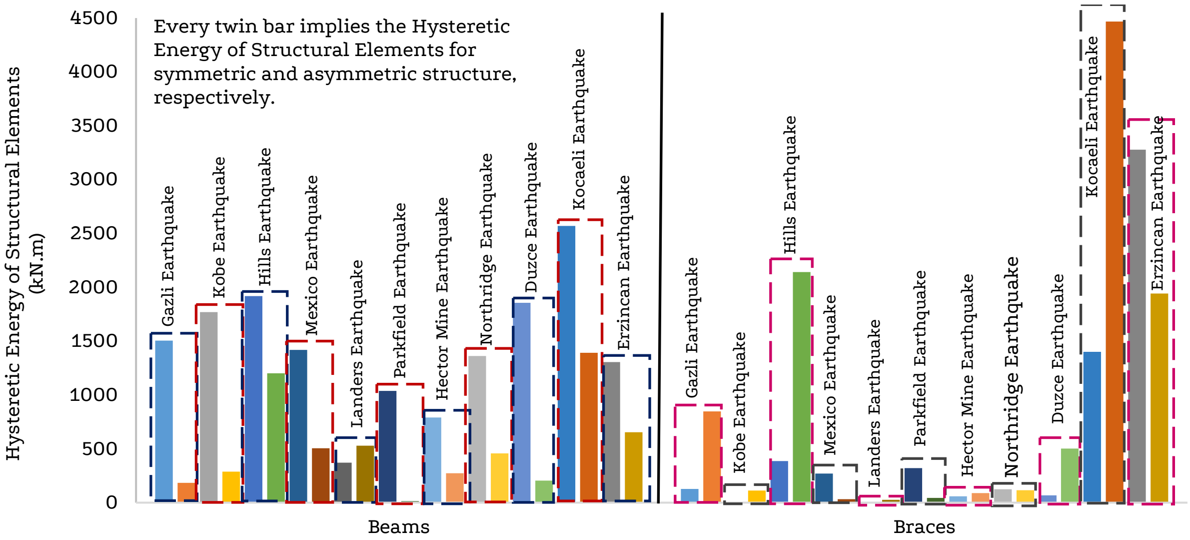
Thus, by evaluating the results obtained from the analysis in the context of the energy dissipation capacity of the beams in the same 20-story steel building, it was obtained that as the earthquake depth between a shallow and deep earthquake increases, the rigidity provided by the asymmetry of the structure has a more negative effect on the behavior of the structure. In addition, as the symmetry in the structure increases, it has been concluded that symmetric buildings under the influence of deep earthquakes have more energy dissipation capacity. However, in a shallow earthquake with the same effect in the same situation, it has been found that the energy dissipation capacity for both symmetric and asymmetric buildings is lower than the deeper ones.

However, according to the analysis in terms of the energy dissipation capacity of the braces, it was obtained that the braces in the asymmetric building work more than the beams and consequently absorb more hysteretic energy compared to the symmetric building model.

By examining the results of nonlinear dynamic time history analysis under the influence of earthquake acceleration records, the story drift ratio is obtained for both symmetric and asymmetric models. Story drift is the lateral displacement of a floor relative to the floor below, and the story drift ratio is the story drift divided by the story height. In this case, the story drift ratio of steel structures was obtained for the eleven earthquake acceleration records applied from nonlinear time history analysis using the energy-based method shown in Figure 27 for symmetric and asymmetric models.



(a)



(b)

Figure 26. Total hysteretic energy of structural members for symmetric and asymmetric model

According to the results obtained from the graphs, the highest value of story drift ratios was observed in the middle floors for both symmetric and asymmetric models, which is decreased by increasing the floor level. The maximum story drift ratio is obtained on the 11th floor as 0.0068 and 0.0053 for the symmetric and asymmetric models, respectively. In this case, it can be concluded that there is an increase of 28.5% in story drift ratio for the asymmetric model compared to the symmetric one.

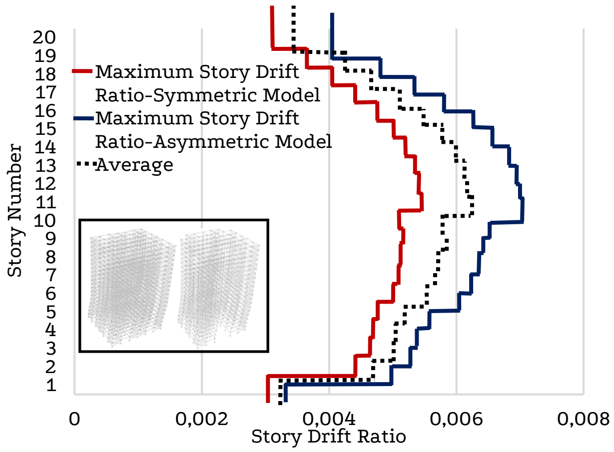


Figure 27. Average of story drift ratio for symmetric and asymmetric model

6. Conclusion

In this study, a series of nonlinear static and dynamic analyses were conducted to investigate the usability of energy-based seismic performance evaluation in multi-story buildings. To carry out the relevant analysis in the study, 20-story steel structure models with symmetrical and asymmetrical plan layouts were designed. In the study, the plastic joints were first defined to observe the bearing members' nonlinear behavior. Then, the nonlinear static and dynamic analyses were carried out using the Perform-3D. The analysis results are summarized as follows;

- The modal analysis results showed that the asymmetric model has a 22.19% more dominant period value than the symmetric model.
- The pushover analysis results showed the asymmetric model structure had 5.18% more top point of displacement and 8.92% more relative drift than the symmetric model structure. It also showed that the symmetric model has 22.65% more acceleration capacity than the asymmetric model. This also indicates that more earthquake input energy will be generated.
- It was determined that the base shear force in the symmetrical model structure was 30.98% higher than in the asymmetric model structure, due to the higher pseudo acceleration. In addition, based on the base shear force and top point displacement curve, it was revealed that the symmetric model has 23.64% more energy capacity than the asymmetric model.
- The pushover analysis results showed that the axial compressive force and tensile force at the moment of yielding were 2690.22 kN and 2346.32 kN, respectively, for the symmetrical model. The strain values at the moment of yielding were determined to be 0.00129 in the tension region and 0.0013 in the compression region. The ultimate strain values for the compression and tension were determined to be 0.00244 and 0.00314, respectively.
- The pushover analysis results showed that the moment values at yield and rupture in the symmetrical model structure were 836.33 kN.m and 950.264 kN.m, respectively. It was also obtained that the rotation value at the moment of yielding was 0.00453 and the ultimate rotation value was 0.0113. This showed that the symmetric model has a very high permanent deformation ability.
- The pushover analysis results showed that the symmetrical model structure had 8.30% less relative story drift than the asymmetric model structure. The response spectrum analysis results showed that the symmetrical model structure had 21.05% less relative story drift than the asymmetric model structure
- By identifying different levels of damage that occur at the structural members and determining the limit states for the structures, the damage level that occurred in both symmetric and asymmetric buildings remained between limited damage (IO) and controlled damage (LS).
- The energy-based analysis results showed that the highest displacements occurred in the symmetric model structure in the Gazli earthquake, and in the asymmetric model structure in the

Landers earthquake. It was determined that the symmetric model structure had 37.92% less displacement than the asymmetric model structure.

- The energy-based analysis results showed that the base shear forces caused by earthquakes were 25.67% less in the symmetric model than in the asymmetric model.

- The energy analysis results showed that at the same yield moment value (950.11 kN.m), the symmetric model reached 24.64% less yield rotation and 24.62% more final rotation value than the asymmetric model. This showed that the symmetric model can change permanent shape more than the asymmetric model.

- The energy-based analysis results showed that the symmetric model structure has 21.95% more axial load-carrying capacity than the asymmetric model structure. In addition, the analysis results show the symmetrical model had 21.91% more axial deformation ability.

- The energy-based analysis results showed more earthquake input energy under the Gazli, Northridge, and Erzincan earthquakes for the symmetrical and asymmetrical structures. It was concluded that, on average, the earthquake energy entering the symmetrical structure was 12.98% more than the earthquake energy entering the asymmetrical one.

- According to the results of the energy-based analysis, it was obtained that the symmetric model structure can absorb 29.18%, 4.25%, and 44.32% more earthquake energy in terms of modal damping energy, kinetic energy, and plastic energy, respectively, compared to the asymmetric model structure.

- However, the asymmetric model was obtained 5.59% more successfully than the symmetric model for absorbing earthquake energy in terms of strain energy. This is based on the fact that the asymmetric model makes more displacement than the symmetric one.

- Depending on the strain energy and displacement, it was determined that the asymmetric model structure provides 21.36% more hysteretic energy than the symmetric model structure.

- At the end of this study, it was found that the results of pushover, response spectrum, and the energy-based analyses performed on symmetrical and asymmetrical building models were generally compatible. In terms of relative axial load carrying and moment capacity, the energy-based method gave higher values than the pushover analysis. However, higher results for pushover analysis were obtained for the related deformation and rotation values than the energy-based method. These values did not change the fact that the symmetric model structure was at the immediate use level and the asymmetric model was at the usable damage level. In terms of relative floor, pushover analysis gave higher results by 53.21% and 65.71%, respectively, compared to the response spectrum analysis results for the symmetrical model structure. According to the asymmetric model, these rates were obtained as 45.55% and 73.15%, respectively.

- At the end of the study, energy-based results were obtained that a damping mechanism that can absorb 32.80% of the input energy can be placed in the damaged areas of the structure so that the asymmetrical model structure operates like a symmetrical model structure.

- The results showed that the energy-based analysis method could be an alternative to pushover analysis in evaluating the performance of a structure.

In this research, energy analysis was carried out for multi-story symmetrical and asymmetrical steel structures. For energy analysis, assumptions were made according to the relevant standards for the structure and ground. To perform a performance analysis of a real building according to the energy-based method, it is necessary to examine the geometry of the building, material quality, bearing system, ground conditions, and geological formation. After the stated parameters of the structure are determined, the earthquake records specific to the structure should be scaled and energy-based analysis management should be applied. According to the obtained results, it is suggested the required energy for the earthquake-damping system to be added to the structure can be determined using this method. At the end of the study, it was confirmed that energy-based evaluation can be an alternative to static pushover analysis for performance analysis. In addition, it has been generalized that how much energy may be required for the damping mechanism that can be used in the repair/strengthening of the structure can be calculated by energy-based evaluation. It has been suggested that the energy-based method can be used to design structural control systems that can be used in the future for the repair/strengthening of a structure that is found to be damaged.

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Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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