COMPARISON OF SEISMIC PERFORMANCE OF KNEE BRACED STEEL FRAME AND ECCENTRIC BRACED STEEL FRAME

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THESIS APPROVAL

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ÖZET

Yüksek Lisans Tezi

DİZ DESTEKLİ İLE EKSANTRİK ÇAPRAZLI ÇELİK ÇERÇEVELERİN SİSMİK PERFORMANSLARININ KARŞILAŞTIRILMASI

ABDINAJIB ABDULLAHI MOHAMUD

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Güçlü depremlerin etkisi altındaki binalar büyük yer değiştirmelere maruz kaldıklarından dolayı yapı elemanlarında önemli hasar oluşabilir ve buda yapının felaketle yıkılmasına neden olur. Bu nedenle yapısal elemanlarda aşırı hasarı önlemek için yapının yeterli dayanıklığa ve sünekliğe sahip olmaları gerekir. Bir yapının depreme karşı dayanıklığının ve sünekliğinin artırılabilmesi, yapıya perde duvarlar, çelik çaprazlar veya damperler gibi yapı elemanlarının yerleştirilmesi gerçekleştirilebilir. Diz destekli çerçeve (KBF'ler) ve Eksantrik Çaprazlı Çerçeve (EBF), sismik bölgedeki bir yapının depreme dayanıklılık kapasitesini artırmak için yaygın olarak kullanılan yapı elemanlarıdır. Bu nedenle, tez çalışmasında, SAP2000 bünyesinde doğrusal olmayan statik analizleri kullanarak çeşitli eksantrik çaprazlı çelik çerçeve ve diz destekli çelik çerçevelerin sismik davranışının değerlendirilmesi amaçlanmıştır. Bu amaç doğrultusunda, hem eksantrik çaprazlı çerçeveler hem de diz destekli çerçeveler ile modellenen 5 katlı çelik çerçeveli bir binanın sismik performans noktasının ve yaya kesme kuvvetlerinin “itme” analizleri olarak bilinen doğrusal olmayan statik analizlere belirlenmesine odaklanılmıştır. Yapılan bir çok analizler sonucunda, diz destekli çerçevenin kesme kapasitesinin, eksantrik çaprazlı çerçeveden daha yüksek olduğu bulunmuştur. Enerji dağıtma sistemi açısından KBF'nin enerji dağıtma kapasitesi EBF'den daha iyi olduğu görülmektedir. Sonuç olarak KBF çapraz çerçeve sistemi, iyi bir sünekliği ve yanal sertliği sahip olmasından dolayı sismik yüklere karşı en etkili yatay yük taşıyıcı sistemlerden biri olarak ve deprem kaynaklı binalarda oluşacak hasarın rehabilitasyona uygulanmalarında kolaylıkla kullanılabilir.

Anahtar Kelimeler: Diz destekli çerçeve (KBF'ler) ve Eksantrik Çaprazlı Çerçeve (EBF), İtme Analizi, Çelik Çerçeveli Yapılar, SAP2000.
2022, vi+ 65 sayfa.

ABSTRACT

MSc Thesis

COMPARISON OF SEISMIC PERFORMANCE OF KNEE BRACED AND ECCENTRIC BRACED STEEL FRAMES

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Buildings under the action of the strong earthquakes undergo large displacements and may suffer significant structural damage, and even be destroyed catastrophically. Therefore, the structure should have enough rigidity and ductility to prevent excessive damage in the structural members. This can be accomplished by placing to the structure elements to be able to enhance earthquake resistance of the structure, such as a shear walls or steel braces, or dumpers. For this structure the knee braced frame (KBFs) and the eccentric braced Frame (EBF) are commonly used structural elements to increase the earthquake resistance capacity of a structure in the seismic region. Therefore, the main aim of this thesis is to assess the seismic behavior of various eccentric braced steel frame and knee braced steel frame using nonlinear static analyses performed by SAP2000. This is achieved by focusing on a nonlinear astatic analysis, known as "pushover" analyses, to determine the seismic performance point of a 5-story steel frame building modeled with both eccentric braced frames and knee braced frames. As a results of several analyses, it was found that the shear capacity of knee braced frame is higher than that of eccentric braced frame. In terms of energy dissipating system, the energy dissipating capacity of KBF is better than EBF. Finally, it is concluded that since KBF combines excellent ductility and lateral stiffness it can be as one of the most effective lateral load bearing system against seismic loads and is easy for application in the rehabilitation of earthquake induced damage to buildings.

Key words: Eccentric, Braced Frame, Knee Braced Frame, Seismic Performance, Pushover Analyses, Steel Frame Structure, SAP2000.
2022, vii+ 65 pages.

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Last but not the least; I would like to place on record my sense of gratitude to everyone who directly or indirectly has lent their hand in this venture.

Abdinajib ABDULLAHI MOHAMUD

03/06/2022

CONTENTS

SYMBOLS and ABBREVIATIONS

Symbols	Definition
BRBF	Buckling Restrained Braced Frame
CBF	Concentric Braced Frame
CMDB	Cast Molecular Ductile Braced System
CP	Collapse Prevention
DL	Dead Load
EBF	Eccentric Braced Frame
IO	Immediate Occupancy
KBF	Knee Braced Frame
LFRS	Lateral Force Resisting System
LL	Live Load
LS	Life Safely
MBF	Mega Braced Frame
MRF	Moment-Resisting Frame
RLSD	Richard Lees Steel Deck
SCBF	Special Concentric Braced Frame
SC-CBF	Self-Centering Concentric Braced Frame
SRSS	Square Root of Sum of Squares
F_i	Earthquake Force at $\dot{I}^{th}$ Story
F_x	Seismic Lateral Force
F_y	Yield Strength
Δ	Design Story Drift
c	Vertical Deflection of Left Column
Δ_{max}	Maximum Amount of Drift
Δ_p	Plastic Story Drift
Δ_R	Vertical Deflection of Right Column
Δ_S	Drift at First Significant Yield
δTSE	Elastic Roof Displacement
δxe	Elastic Story Deflection

Δ_Y	Drift at Structural Collapse Level
θp	Column Rotation Angle
μs	Ductility Factor
μT	Period Based Ductility
φ	Resistance Factor
φi	Displacement Amplification Modifier at ith Story
Ωd	Design Over Strength Factor
Ωm	Material Over Strength Factor
C	Structural Over Strength Factor
Ac	Cross Section Area (m2)
BKS	Building Usage Class
BYS	Building Height Class
DTS	Earthquake Design Classes
EXN	Earthquake Loading In Negative X Direction
EXP	Earthquake Loading In Positive X Direction
EYN	Earthquake Loading In Negative Y Direction
EYP	Earthquake Loading In Positive Y Direction
Ed(H)	Horizontal Earthquake Component
Ed(z)	Vertical earthquake Component Strength
G	Dead Loads
GO	Collapse Prevention
H	Height of Floor
h	Section Height
HN	Building Height
I	Building Importance Factors
KH	Controlled Damage
KK	Continuous Usage
Lp	Length of Plastic Hinge
Ls	Shear Span Mass
SH	Limited Damage

FIGURES

TABLES

1. INTRODUCTION

Earthquake resistant structures should be designed to withstand large deformation caused by earthquakes in order to absorb and attenuate imposed energy. On the other hand, they should be stiff enough to transfer forces to the base without collapsing. Structures designed to resist moderate and frequently occurring earthquakes must have sufficient stiffness and strength to control deflection and to prevent any possible damage. However, stiffness and ductility are generally two opposing properties, it is desirable to devise a structural system that combines these properties in the most effective manner without excessive increase in the cost. Therefore, engineers design a structure not to remain in the elastic region under severe earthquakes because of the economic constraints. The inherent damping of yielding structural elements can advantageously be utilized to lower the strength requirement, leading to a more economical design. This yielding usually provides the ductility or toughness of the structure against the sudden brittle type structural failure.

The moment resisting frame possesses good ductility through flexural yielding beam elements, but it has limited stiffness. By introducing steel braced in structure to overcome the deficiencies in moment resisting frame. The braced element in a structural system is critical to the structural behavior during an earthquake. Steel braced is an efficient and cost-effective method of resisting lateral forces in a framed structure.

Steel braced increases shear capacity of the structure and can be used as retrofit as well. Braced provide stability and resist lateral loads. The steel braced are of different types, they are; concentric braced frame (CBF), eccentric braced frame (EBF) and knee braced frame (KBF).

CBFs are made up of the frame's plane-positioned diagonal braces, with both ends connected to the ends of other framing members, whereas EBFs do not have one or both ends connected to the ends of other framing members. Eccentric braced frame systems are braced frame structures distinguished by an eccentricity inserted into the beam separating a section known as the link. Figure 1.1 demonstrates a case of an EBF, with the link situated between two braced in the middle of the beam.

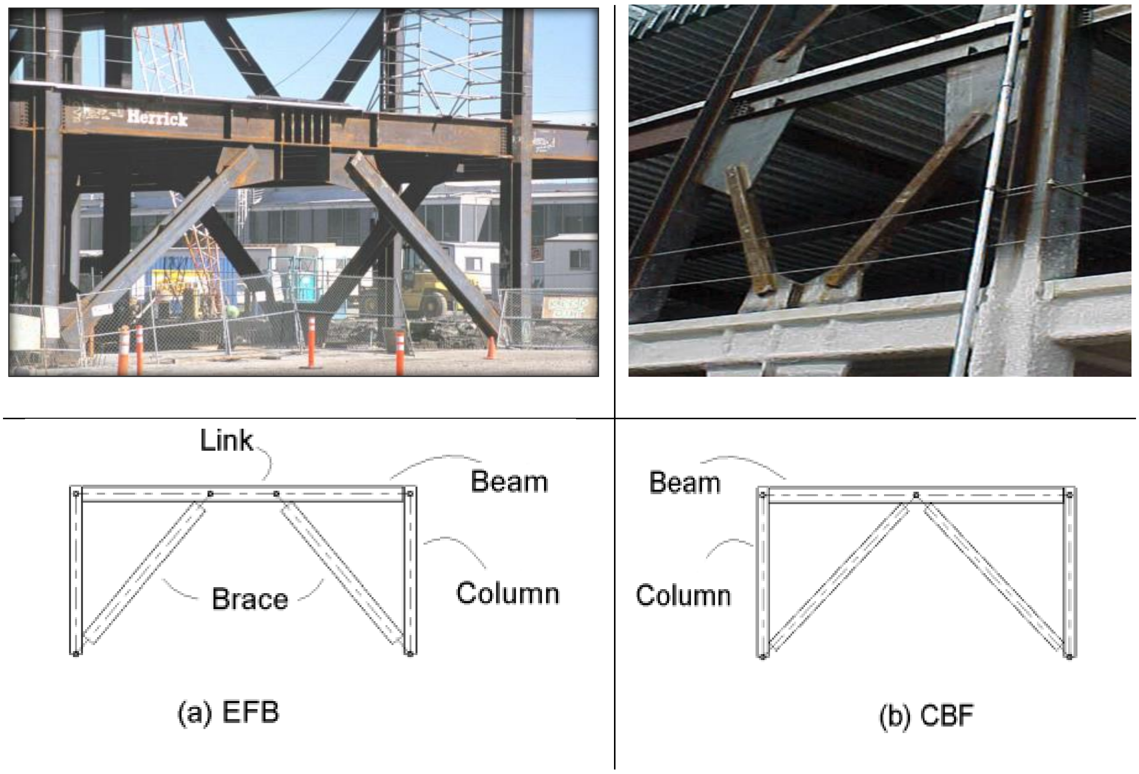

Figure 1.1 Examples for both eccentric braced frame (EBF) and concentric braced frame (CBF).

Members form a truss-like structure in CBF systems, generating a stiff frame, however, because of buckling of the diagonal braced its ductility is limited. Whereas EBF combines the qualities the combination of a moment frame and a concentric braced frame, reducing the shortcomings of each system and improving system performance in earthquakes. By a suitable choice of eccentricity, a sufficient amount of stiffness from the braced is retained while ductility is achieved through the flexural and/or shear yielding of a segment of the beam, which is called the link, created by the eccentric braced member.

Balendra et al. (1994) revised the system proposed by Aristizabal-Ochoa, (1986) who introduced a framing system, which combines the stiffness of a diagonal braced with the ductile behavior of a knee element. The revised system is called the knee braced frame (KBF). In this system, the non-buckling diagonal braced provides most of the lateral stiffness. The flexural or shear yielding of the knee element provides the ductility under a severe earthquake. In this way, the damage is concentrated in a secondary member, which can be easily repaired at minimum cost. The knee braced is a new braced system the joint between the beam and column to a short component. The name of this brief member is the "knee element" which is designed to yield in flexure as shown in Figure 1.2, where by buckling of the brace is prevented the link is crucial to the seismic performance of EBFs and KBFs, as will be addressed later. One end of the link may be linked to a column in some eccentric braced frame arrangements, as seen in Figure 1.3.

Figure 1.2 A recent example of construction KBF

Figure 1.3 Example connect a link to a column

The knee braced steel frame has excellent ductility and lateral stiffness. Because the knee element is properly fused, yielding occurs only to the knee element and no major elements are harmed. It performs better during seismic activity than other types of braced.

Several studies have been performed to evaluate various type of braced frames' performance during earthquakes. The primary goal of this thesis research is to analyze the behavior of both eccentric braced and knee braced frames when subjected to earthquake loads. 5-storey EBF and KBF prototype buildings are modeled in SAP 2000 and several nonlinear static pushover analyses have been performed to evaluate the seismic performance of these structures. These analyses cover the determination of the system performance, the shear capacity curves, the performance points, the plastic hinges and the stabilities of the both type of the braced systems. Results obtained from these analyses are compared to discuss the seismic performance of the both type of the braced systems in order to determine which type of the braced system has a better seismic performance.

2. LITERATURE REVIEW

2.1 General

In this section, the results of previous research on the thesis topic will be presented. There has been a lot of research done about knee braced frames and eccentric braced frames. The available body of literature spans several decades and is rapidly expanding. As a result, it cannot be adequately summarized in a single chapter. Instead, this section provides an overview of major references, as well as useful citations to previous studies that include comprehensive analyses of relevant literature. This chapter's literature review is divided into two sections: KBFs reviews, EBFs reviews.

2.2 Eccentric Braced Frame

Eccentric braced frames (EBFs) are a relatively new lateral force resisting system designed to provide predictable resistance to seismic events. EBFs that have been correctly conceived and elaborate will acting ductile when A connection element's yielding in shear or flexure occurs. This is the most alluring aspect of EBFs for earthquake resistant architecture.

David and Sarif, (2020) evaluated eccentric braced frames performance during earthquakes in various layouts. They modeled the eccentrically braced frames in SAP 2000 and analyzed these structures both linearly and nonlinearly. The analysis was linear provides information about ratios of mass participation and modal forms. The pushover analysis is part of the nonlinear analysis and provides details on the mechanics of breakdown and performance measures. This research was to contrast the effectiveness of eccentric braced frames to that of frames that resist exceptional moments, which aids in understanding the efficiency of both systems' structures. This analysis finds that all of the chosen eccentric braced frame layouts experience SMRF frames have a modest roof displacement that is significantly less than the intended drift experience enormous displacement.

Meynerd Rafael & Lukas, (2020) also investigated the position of plastic hinges and the deformation of plastic in structures with six, nine, twelve, and fifteen stories were assessed using an assessment of a four-eccentric braced frame under push-over nonlinearity performances. The majority of these buildings exhibit excessive plasticization of out-of-beam members, despite the AISC design provision allowing for moderate plasticization of these members. As a result, the beams coming out of the link may be at risk of flange and a web fracture. Similarly, this was contentious evidence in the Christchurch earthquakes. To address this issue, either fixed connections or extremely short shear connections, or braced members with less end moment force than out of link beam moment power is advised. The response coefficients of mutation are computed for with this modification.

David & Koboevic, (2008) conducted the progressive collapse the first phase of a larger study looking into the earthquake behavior of chevron-type eccentric braced frames (EBFs) designed to Canadian design standards. Greater height and shear-critical linkages are highlighted standard eastern and western North American locations. The 14- and 20-story frames' design techniques are both described and used. The significance of various the discussion of design requirements and the best a potential design sequence It was discovered that ductility requirements did not influence design. To determine whether the design processes produced the intended frame sensitivity, and non-linear time-history analysis was used to examine the earthquake reaction of the frames.

Shayanfar et al., (2014) investigated the behavior of eccentric braced frames with vertical links when subjected to earthquake loads. It is also worth noting that the frames were created using a capacity design method. Their results showed that the plastic hinges, inter story drifts, and plastic rotation of links are distributed more uniformly in the height of frames designed by the suggested method compared to the International Building Code 2009 method.

Ma et al (2022) analyzed two types of variable-cross-section shear links for horizontal eccentric braced steel frames (H-EBFs) using the web weakening principle. The first design was an I-section link (L2) with a lower web and wider flange than an ordinary shear link (L1), and the second design was an I-section link composed of two small I-section beams (L3). Cyclic loading tests were performed on three scaled specimens, L2 (H-EBF-L2), L3 (H-EBF-L3), and a traditional link, to investigate the seismic behavior of the two types of H-EBFs (H-EBF-L1). The results revealed that the shear links absorbed the majority of the inelastic actions of the three specimens, and that the failure modes were similar. in comparison to specimen.

Saghafi et al. (2016) investigated the seismic performance of retrofitted reinforced concrete buildings using EBFs with single and knee links. Different editions of the Iranian Seismic Code (Standard No. 2800) were reviewed for evaluation of modeled buildings. To that end, three medium ductility Moment-Resisting Frames (MRF) of four, eight, and twelve stories were modeled and designed for seismic loads using Standard 2800, 2nd edition. Models were seismically reloaded based on the Standard 2800, 3rd edition in order to evaluate buildings under modified seismic loads. Their results revealed that the stress ratios in most columns exceeded one. Buildings were therefore retrofitted with EBFs with knee and single vertical links, and their seismic performance was assessed using nonlinear static analysis.

2.3. Knee Braced Frame

Aristizabel-Ocho (1986) has proposed a framing system, which combines the stiffness of a diagonal brace with the ductile behavior of a knee element in order to overcome the deficiencies in moment resisting and concentric braced frames. However, this system was not suitable for earthquake-resistant design because the brace was designed to slender. Consequently, the brace buckles and leads to pinching of the hysteresis, which is not efficient for energy dissipation. Balendra et al., (1994) re-examined and modified this system by introducing the non-buckling diagonal brace that provides most of the lateral stiffness. The flexural or shear yielding of the knee element provides the ductility under a severe earthquake.

Dezhkam (2021) investigated how the way plastic hinges are built, as well as their distribution and failure mechanisms that can have a big impact on seismic structure design. Frame sensitivity to secondary effects, overall and local ductility, energy absorption, and structure resistance prior to damage, as well as general instability and destruction are all affected by the failure mechanism type. The general type failure mechanism is a variant of the second type failure mechanism in which plastic hinges are located at the two ends of beams and the first-floor columns at the foundation connection. According to studies, this type of system absorbs the maximum energy when used against earthquake. knee braced frames are useful as an energy dissipation system because they are made up of ductility and lateral stiffness, and they perform well against lateral stresses, particularly earthquakes.

Anitha & Divya (2015) performed a progressive collapse analysis in seismically active areas must meet two major criteria Such that it must be stiff enough to control the drift and prevent structural damage, as well as ductile enough to prevent collapse caused by dramatic deformation.

Thamarakshan & Arunima (2017) analyzed steel frames in various configurations of the knee braced frames using the time history analysis by ETABS. They compared results of time history analyses with those of the pushover analysis results.

Raphael et al (2016) investigated the seismic effect of knee braced steel frames. Pushover analysis was used to compare a double knee braced steel frame to a single knee braced steel frame. A non-linear analysis was used to study the effect of knee angle on a knee member angle study. Displacement and stiffness were investigated as parameters.

Mohamed & Zaki, (2014) developed seismic design system for the knee braced frame to addresses the shortcomings of the eccentric braced Frame that is more difficult to repair than the knee braced frame. The system also has good ductility, comparable to that of an eccentric braced frame with high base shear resistance. They looked into the reaction of a knee braced frame under seismic loading and investigated the many parameters that could improve the system's seismic response. A finite element analysis software was used to model one-bay single-story frames and perform nonlinear evaluations. The maximum system energy dissipation factor value was used as the criterion for selecting the optimum value for each parameter.

Goel and Chao (2008) introduced a conceptual framework to calculate the risk level that involves equating the work required to steadily raise the building toward the desired drift toward the work necessary to reach the same state by a comparable single degree of freedom in elastic-plastic system (Leelataviwat et al. 1999; Lee and Goel 2001).

Leelataviwat et al (2010) were outlined a structural system's design strategy known as a knee braced moment frame (KBMF). The frames for KBMF was built with a special yield mechanism, which causes the knee braces to yield and buckle, while beams are considered to be plastically hinged at their ends outside of the knee parts.

3. MATERIALS and METHODS

In this section, theoretical background, the details of the material and methods used for the structures evaluated within the scope of the thesis are explained.

3.1 Theoretical Background

The static pushover analysis is one of the popular tool for seismic performance evaluation of existing and new structures. The pushover analysis provides adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. This is analogous to static load pattern chosen for nonlinear static analysis, and the structures are forced to cross a predetermined roof sway. The goal a structure's individual drift is computed for each design seismic spectrum and the performance of structures developed. In its first step the structure's yield and hinge formation process are determined. The quantity and arrangement of hinges at same buildings' roof drift are shown Figure 3.1. In general, how many hinges are there in structures developed approach is superior to the number of hinges in buildings constructed using the standard method. For instance, there are 41 and 31 hinges, respectively, in 20-story buildings built using standard procedures.

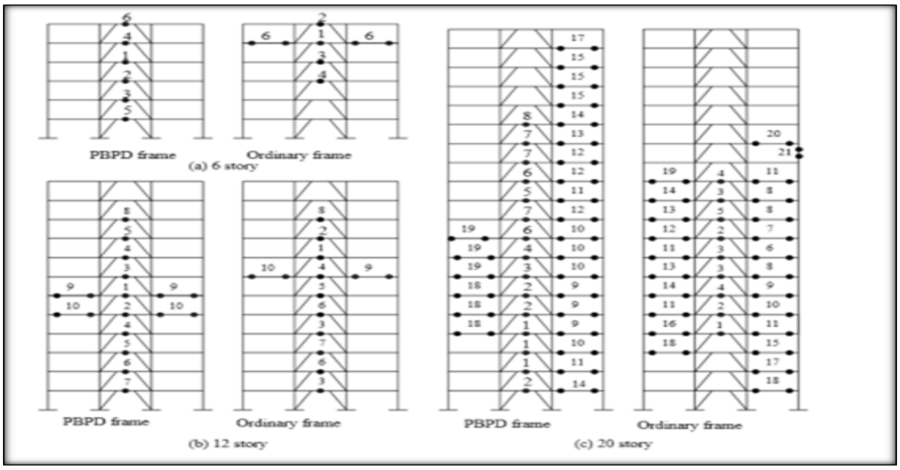

Figure 3.1. Comparison of hinge formation order.

3.1.1. Damage limits and zones in structural members

Pushover analysis needs the development of the force deformation curve for the critical section of beams and column by using the guideline in FEMA 356 code. Such a curve is presented in Figure 3.2. Linear response from unloaded condition (A) to effective yield (B) describes load deformation relation. Then, the stiffness reduces from point B to C. Immediately occupancy performance level (IO) (the structure is controlled against an earthquake of 50 percent occurrence probability in 50 years), The building is protected against an earthquake with a 10% chance of occurring in 50 years thanks to its life safety performance level (LS) and collapse prevention level (CP) (the structure is controlled against an earthquake of 2 percent occurrence probability in 50 years).

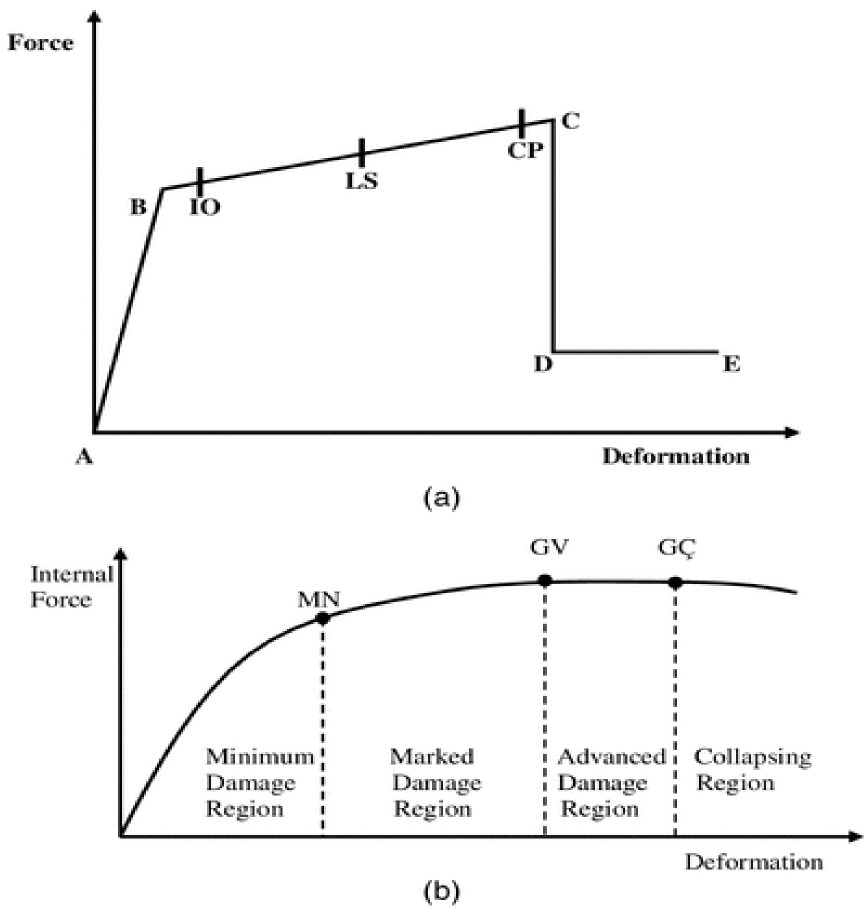

Figure 3.2. (a) Force deformation curve (b) Damage areas specified for sections

Figure 3.2(b) shows different performance level of building described along with a force displacement curve which shows the behavior of global structure against lateral load. The first region up to the point (MN) indicates that building is at operational level where very little damage, temporary drift, structure retains original strength and stiffness, all systems are normal means. At the region from point (MN) to (GV) the building is in the immediate occupancy level where little damage, temporary drift, structure retains original strength and stiffness, elevator can be restarted. The region from the point (GV) to (GÇ) represent the safety level of the building where fair damage, some permanent drift, some residual strength and stiffness left, damage to partition, building may be beyond economical repair. After this point the collapse of the building occurs with severe damage, large displacement, little residual stiffness and strength but loading bearing column and wall function.

In Turkish Building Earthquake Regulation 2018, Table 5 C.5, the limit values for the link beam rotation angle are given for the performance levels, and these values are explained in the table below:

Table 3.1. Short tie beam plastic rotation angle limit values

Element Type	Plastic Rotation Limits		
	IO	LS	CP
Steel Tie Beam			
$e \leq 1.6 \frac{M_{pe}}{V_{ye}}$	0.005	0.12	0.15

Performance design levels is defined immediately occupancy performance level (IO) life safety performance level (LS) and collapse prevention level (CP). The performance levels of the tie beams are obtained by controlling the short link beam rotation angles obtained as a result of the non-linear static pushover analysis. The limit values is specified in the regulation (Table 3.1).

3.1.2 Limitation of inter story drift

Inter story drift must be limited in order to protect the structural frame's lateral displacements from damaging the delicate non-structural components. The ductile frame members are not cracked or harmed, and the masonry infills, in particular, are not damaged, which significantly reduces the performance of the entire building.

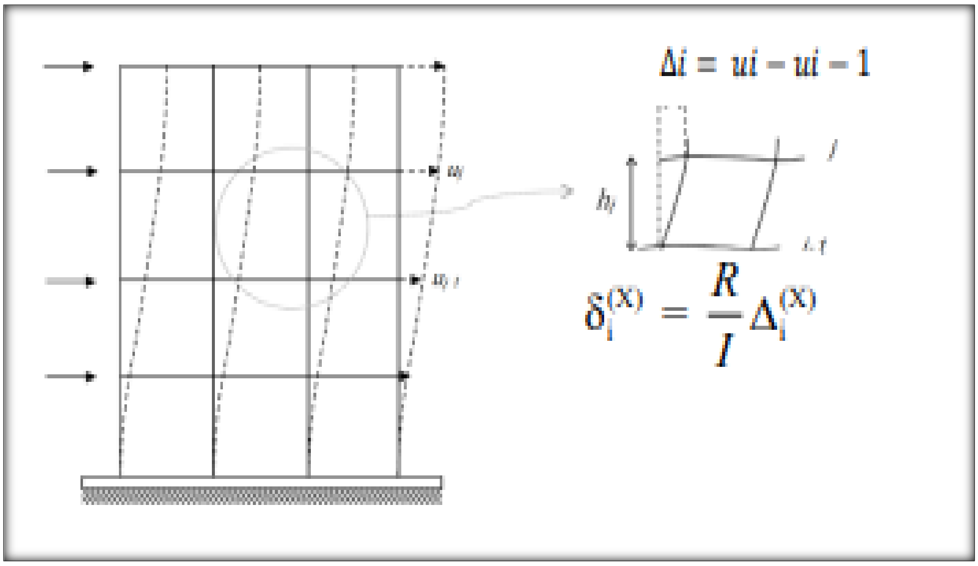

Figure 3.3. Inter story effective drift in a frame inter story drift δi at the i^{th} story

The Turkish Building Earthquake Code 2018 encourages the development of such interface connections by establishing stricter drift restrictions on lower bounds and adaptable connections between infill frames for direct contact links (Figure 3.3).

The drift for infills rigidly connected to the frame is calculated by the following relation given below

$$\boldsymbol{\lambda} \frac{\boldsymbol{\delta^{(x)}}_{\boldsymbol{imaks}}}{\boldsymbol{h_i}} \leq 0.008\mathrm{k} \tag{3.1}$$

Where λ is the empirical coefficient used to limit the relative story drifts. $\boldsymbol{\delta^{(x)}}_{\boldsymbol{imaks}}$ (x) is the maximum value of the effective relative floor offsets in the second floor of the building for earthquake generation. $\Delta_i^{(x)}$ is the displacement as result of shear acting in the earthquake direction and it represents relative the displacement difference between two consecutive floors. $\Delta_i^{(x)}$ is calculated using $\mu_i^{9x)}$ and $\mu_{i-1}^{9x)}$ (x) that are reduced displacements at i and (i-1) stories for any column or wall in the seismic direction.

If there is flexible material between wall and columns or shear walls, then the limit is used as 0.008 κ.

The drift limits for infills rigidly connected to the frame is defined as

$$\boldsymbol{\lambda} \frac{\boldsymbol{\delta^{(x)}}_{\boldsymbol{imaks}}}{\boldsymbol{h_i}} \leq 0.016\text{k} \tag{3.2}$$

If there is not any flexible material between wall and columns or shear wall, then the drift limit should be less than 0.016 κ. λ represents the spectral acceleration ratio of DD-3 to DD-2, which typically falls between 0.4 and 0.5. κ is 1.0 for concrete structures and 0.5 for steel structures. The relation clearly indicates that in flexible, long-term frames, the inter story design is controlled by drift limit as opposed to design forces.

A function of the structural ductility factor and the ductility reduction factor is the energy modification factor is given by Equation 3.1. As shown in Figure 3.3, the approach suggested by new mark Hall (1982) is also used in this study to connect the structural ductility factor and the ductility decrease factor. Figure 3.4 depicts pieces of the energy modification factor computed using Equation 3.2. For elastic systems, according to the selected design spectrum, the design pseudo acceleration can be specified as

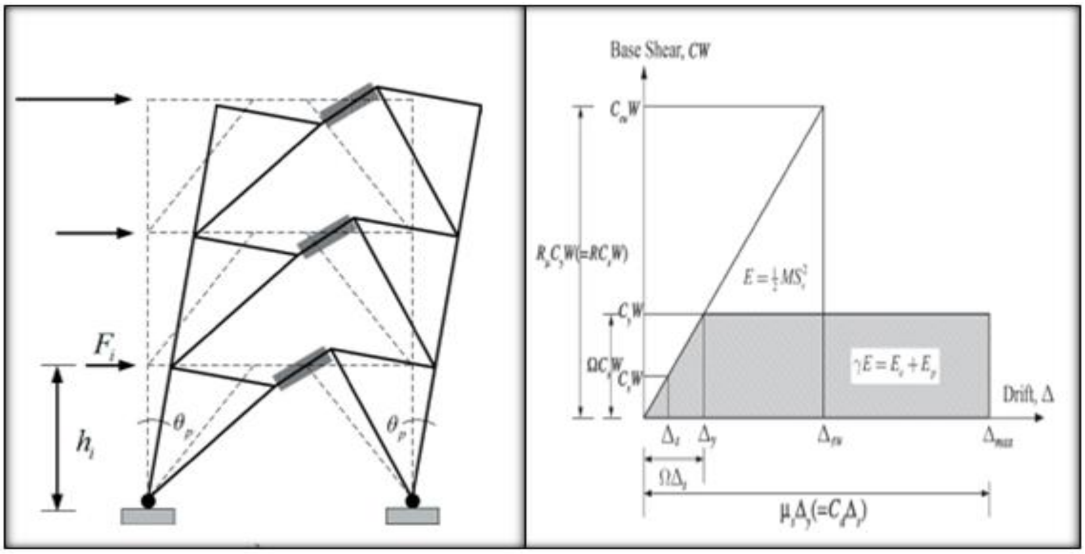

Figure 3.4. Performance-based design concept

The usual elastic design pseudo-acceleration spectra provided construction regulations, can be used to calculate the design elastic energy demand, E. For elastic systems, according to the selected design spectrum, the design pseudo acceleration can be specified as where $\emptyset_p$ is the structure's The distinction between the global inelastic drift and preselected target drift.

3.1.3. Desired release mechanism

In the case of an oscillating mechanism exposed to intended lateral pressures and driven to the desired translation limit. Figure 3.5 depicts a typical shear frame.

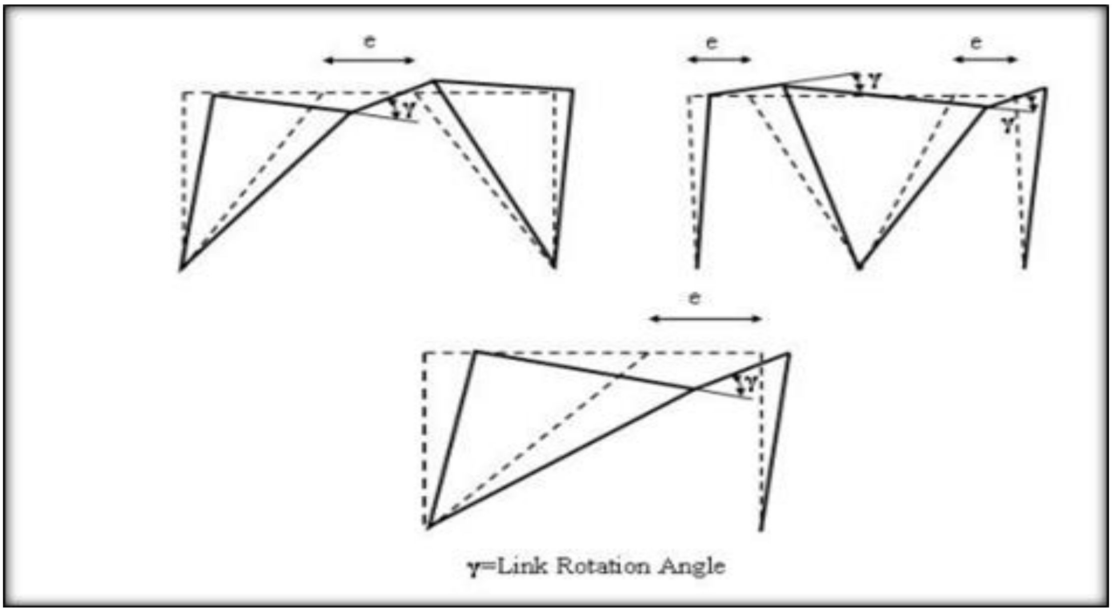

Figure 3.5. Link rotation angle

The goal of this mechanism model is to keep performance-based design deformation to a minimum at the beam point and at the lowest branch's base. Selecting a desired oscillation mechanism, nonlinear relationships between force and deformation, and target offset as performance limit states for certain damage levels is critical from the start of the design phase to achieve more predictable structural performance under strong earthquake ground motions. Determine proper design lateral forces, choose a desirable yield mechanism, and offset for specified hazard levels should all be part of the design process from the beginning.

Yiğitsoy (2010) also investigated the length of the connection (e) is the most important aspect in the yield mechanism of links. Link types are usually classified using the length of the normalized link that is defined by the ration, e/ (M_p/V_p) where V_p is the plastic shear capacity and M_p is the plastic moment capacity of the link. Where the length of the normalized link, plastic shear capacity and plastic moment capacity are calculated using the equations given below.

$$M_p = Z_X F_Y \tag{3.3}$$

$$V_P = 0.6 F_Y (d - 2t_f) t_w \tag{3.4}$$

$$e \leq 1.6x \frac{M_P}{V_P} \tag{3.5}$$

Where with Z_X is plastic modulus of the section, f_Y is minimum yield stress, d is the height of the section, t_f is the flange thickness of the section and t_w is the web thickness of the section.

Shear forces influence yielding is defined by the inequality, e≤1.6 M_p/V_p that is are known as shear yielding links. If the normalized link defined by the inequality e≥2.6M_p/V_p is satisfied it means the link is at the end and is subjected to a large bending moment, and it yields before it reaches its plastic shear capacity. This is called as flexure yielding linkages. The link yields both due to shear force and bending moment for intermediate lengths if the following inequality condition meet 1.6M_p/V_p<e<2.6M_p/V_p,. The links in this interval are defined as combined shear and flexural yielding links.

The length of the ligament is classified as 3 different types as short, medium and long, as indicated below. The effect of tie beam length on the post-yield mechanism states of the bow beams is defined as follows in the American regulation AISC 341 10 and the new Turkish Building Earthquake Regulation-2018 provides the following limits.

- Tie beam length e$\leq 1.6x \frac{M_P}{V_P}$ if short (cutting effect active)
- Tie beam length $1.6 \frac{m_p}{v_p} \leq e \leq 2.6\, m_p/v_p$ if medium (combined effect (bending and shearing)
- Tie beam length $e \geq 2.6 \frac{m_p}{v_p}$ if long (bending active)

The shear bearing capacity used in the design of the tie beam must be greater than v_d, which is the largest of the shear force values under the considered loads.

3.2 Prototype Steel Farmed Structures with Braced Systems

3.2.1 Geometric information

Two five floors steel framed structure braced with both knee braced frame and eccentric braced frame are detailed here. Figure 3.6 shows a typical floor plan with (KBF) and (EBF). Three-dimensional overall system view generated by SAP2000 for an eccentric braced frame and a knee braced frame are given in Figure 3.7 and Figure 3.8 respectively. The schematic floor heights of the 5-storey steel framed structure (1-1) Section) for both EBF and KBF are given Figure 3.9 and Figure 3.10 respectively.

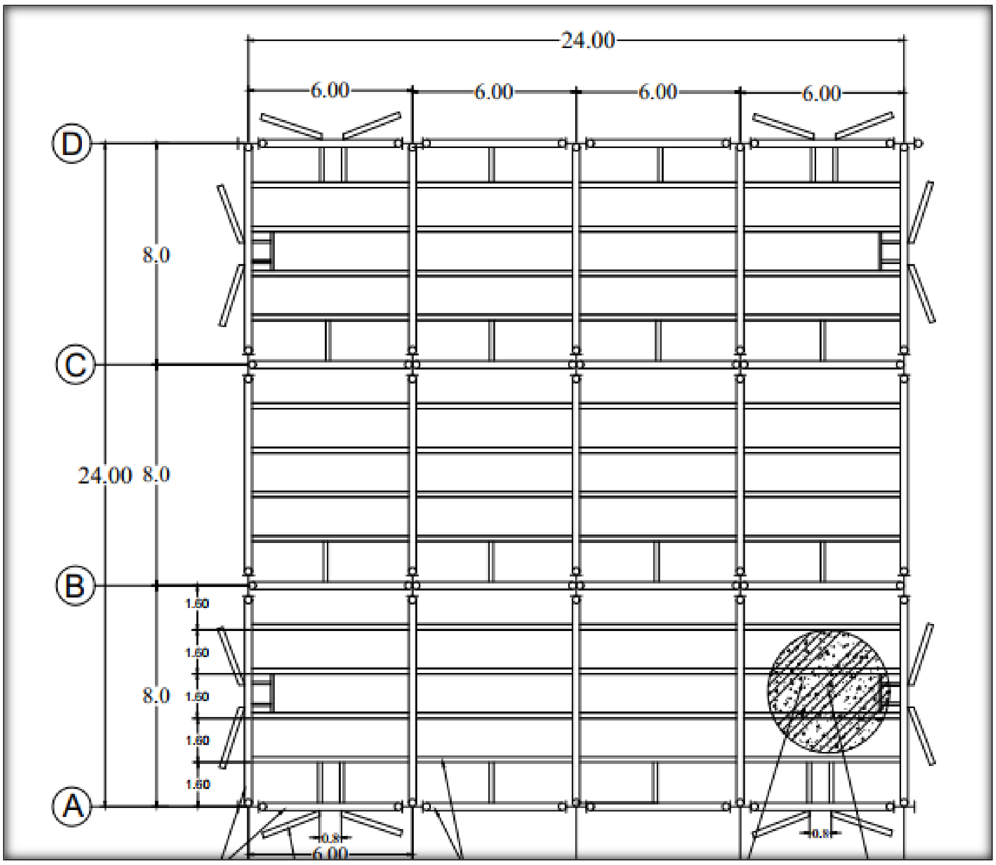

Figure 3.6. Normal floor system plan of (EBF and KBF)

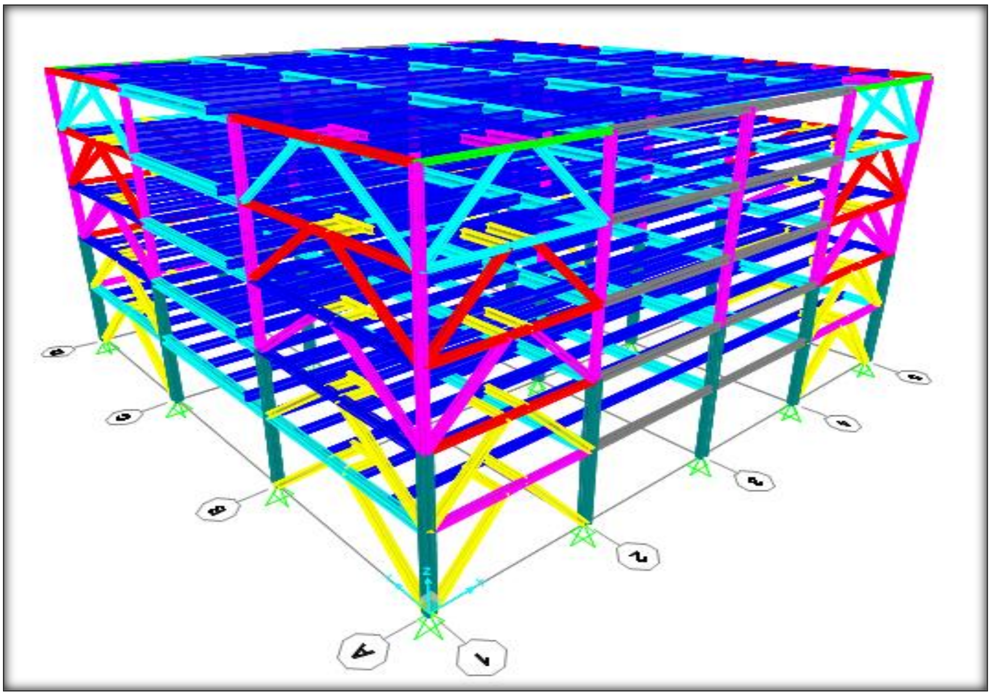

Figure3.7. General system view (EBF)

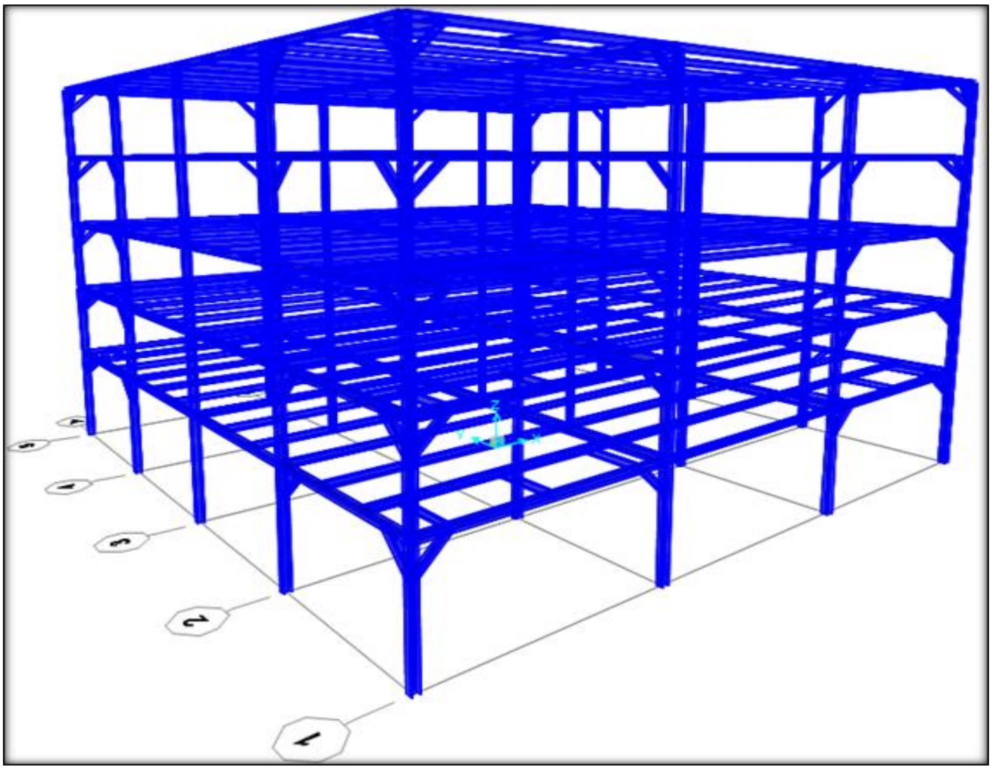

Figure 3.8. General system view (KBF)

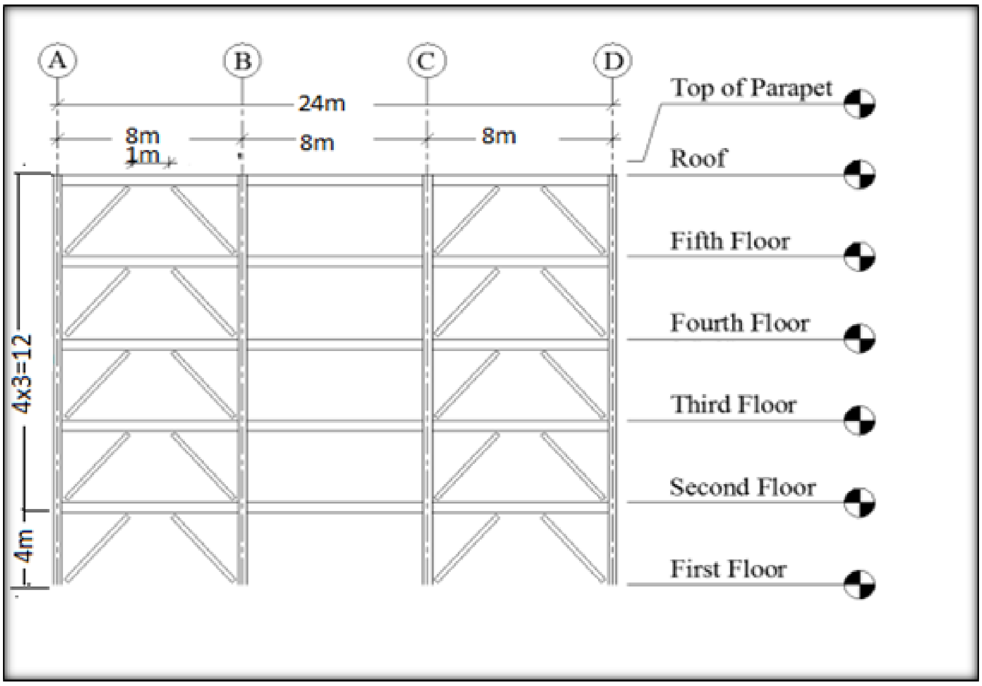

Figure 3.9 Schematic floor heights of the 5-storey steel framed structure (1-1 Section) (EBF).

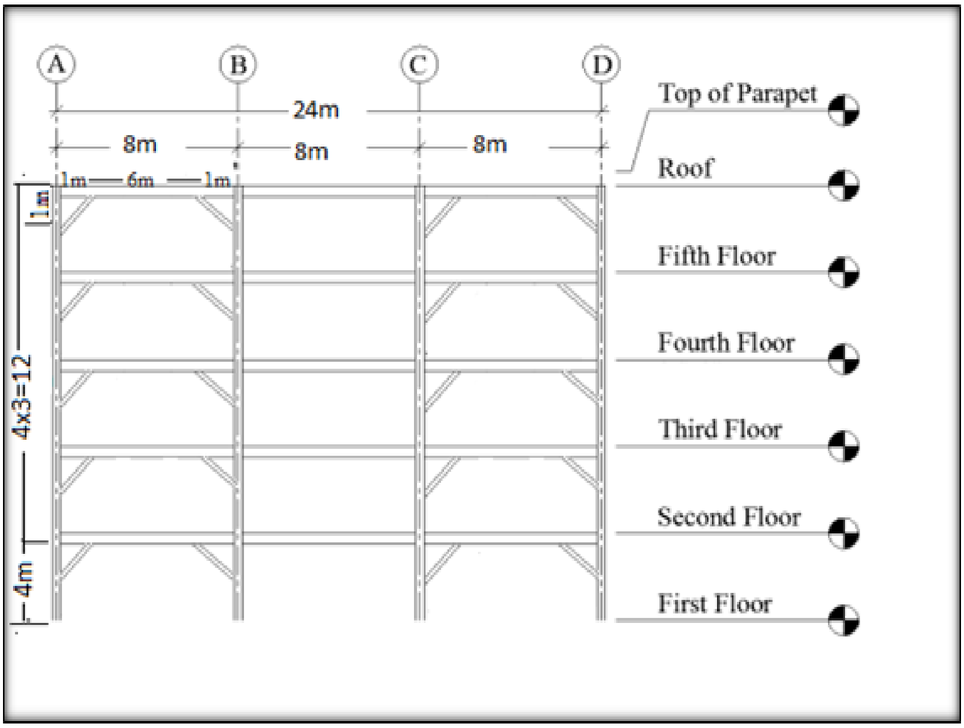

Figure 3.10 Schematic floor heights of the 5-storey steel framed structure (1-1 Section) (KBF)

The floors of eccentric braced and knee braced frame steel buildings, whose geometrical properties are given Table 3.2 and 3.3 respectively, consist of horizontal load-bearing systems in both directions and frames with high ductility level that transmit moments. The primary beams are articulated with secondary intermediate beams that are constructed at two-meter intervals. The connections between the major frame beams on the axles and the columns will be articulated in a similar manner. At an elevation of +0.00, the columns are hinged to the foundation.

For analytical results, a hinged frame with braced and a steel hinge at the end of a single-bay, single-story frame was created. Additionally, the purpose of embedding steel hinge at the end of the brace member is to increase the ductility of the braced when considering buckling control.

Hence, it is necessary for the bearing capacity of the steel hinge at the end of the brace member to be less than the buckling load of the braces. Therefore, it is necessary to determine buckling load of the brace before designing steel hinge dimensions as shown in Table 3.2 and Table 3.3.

Table 3.2. Section properties of frame (EBF)

Member	**Section**	**FLOORS**	**Steel Grade**
Beams	HE280B	1	S275
	HE280B	2	S275
	HE200B	3 and 4 and 5	S275
Columns	HE2400B	1	S275
	HE240B	2	S275
	HE220B	3 and 4 and 5	S275
Braces	HE180B	1 and 2	S275
	HE160B	3 and 4 and 5	S275

Table 3.3. Section properties of frame (KBF)

Member	Section	FLOORS	Steel Grade
Beams	HE240B	1	S275
	HE200B	2 and 3	S275
	HE180B	4 and 5	S275
Columns	HE280B	1 and 2	S275
	HE240B	3	S275
	HE200B	4 and 5	S275
Knee Braces Tub	60x30x5	1 and 2 and 3	S275
	60x30x4	4 and 5	S275

Table 3.4. The beam, column front dimensions, floor area and floor height of the 5-storey building within the scope of the thesis with (EBF) and (KBF).

Floor	Floor Height (m)	Floor Area (m)
Roof	3.0	576
4	3.0	576
3	3.0	576
2	3.0	576
1	4.0	576

3.2.2 Material and building information

All the clamps of the structure belong to the S 275 steel class (F_Y-50 $M_{pa}$and F_u-430M_{pa}). The eccentric braced frames (EBF) and knee braced frames (KBF). The eccentric braced frame profiles of the carrier system elements are given in Table 3.5.

Table 3.5. Coordinate and ground information of buildings

Latitude	41.0300000
Longitude	28.8000000
Local Ground	Grade Z_c

3.2.3 Earthquake data

The building is located in the province of Istanbul and the coordinates and ground class information are given in Figure 3.11.

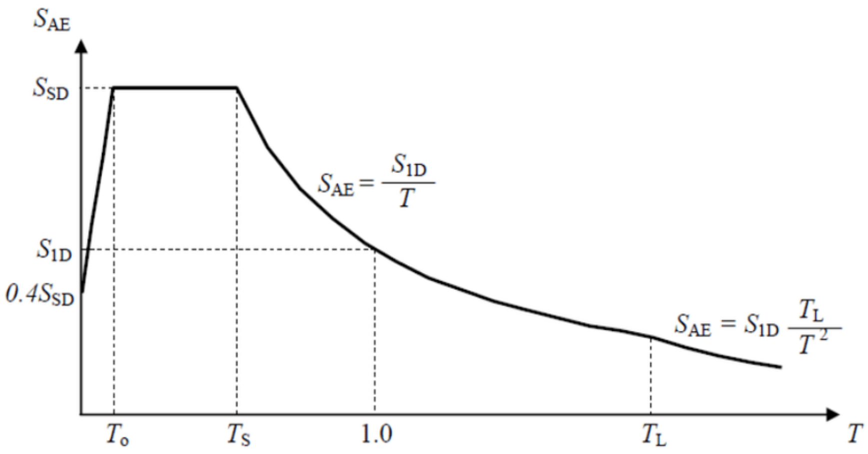

Figure 3.11. Earthquake horizontal elastic design spectrum of the coordinates of the structures designed.

Spectral acceleration coefficients and design spectral acceleration coefficients obtained using Turkey Earthquake hazard maps are summarized. Standard earthquake ground motion (DD-2) and frequent earthquake ground motion (DD-3) for eccentrically braced frame only Table 3.6.

Table 3.6. Earthquake parameters of the coordinates of the structures

	DD-2	DD-3
S_s	0.988	0.379
S_1	0.274	0.106
S_{DS}	1.186	0.493
S_{D1}	0.411	0.159
T_A	0.07s	0.06s
T_B	0.35s	0.32s

3.2.4. Load combinations

Fixed and moving vertical loads acting on 5-storey (KBF) and 5-storey (EBF) structures are summarized as follows

Vertical loads and load combinations

Vertical loads acting on the structure are calculated and summarized in table 3.7

Table 3.7. vertical loads of $(\boldsymbol{EBF})$ $\boldsymbol{and}$ $(\boldsymbol{KBF})$

Roof Floor	Roofing	1,0	kN/m^2
	İnsulation	2,0	kN/m^2
	Suspended ceiling plumbing	0,5	kN/m^2
	Roof Dead Load(G)	4,3	kN/m^2
	Roof live load(Q)	2,0	kN/m^2
Normal Floor	Covering	0,5	kN/m^2
	Suspended ceiling plumbing	0,5	kN/m^2
	Normal floor Dead Load(G)	4,9	kN/m^2
	Normal floor live load(Q)	2,0	kN/m^2
	wall load(Q)	3,0	kN/m^2

3.2.5. Determination of earthquake loads

3.2.5.1. Seismic Loads and Related Load Combinations

Unless a more precise calculation is made, the dominant natural vibration periods in each direction of the building is calculated according to Turkish Building Earthquake Code 4.7.3. In this equation, represents the total story masses and can be obtained using Turkish Building Earthquake Regulation Equation (4.16) as detailed in Table 3.7, the live load participation coefficient can be determined as N=0.3 by using to Turkish Building Earthquake Code Table 3.8.

$$(n = 0.3)\ \ \mathrm{m} = {}^{w}/_{g} \tag{3.6}$$

$$M_{I=\frac{Wi}{g}=\frac{1}{g}[g_i+nq_i]} \tag{3.7}$$

Table 3.8. Story weights and story masses with (EBF) and (KBF).

Floor	**Floor (m^2)**	**G (kN/M^2)**	**Q (kN/M^2)**	**Weight (kN)**	**Mass ($kNs^{2/m}$)**
Roof	576	4.3	2	287.71	2822.4
4	576	4.9	2	322.94	3168
3	576	4.9	2	322.94	3168
2	576	4.9	2	322.94	3168
1	576	4.9	2	322.94	3168
Total				**15494.4**	**1579.45**

3.2.5.2. Determination of earthquake loads acting on floors

According to Turkish Building Earthquake Code 4.7.2, the total equivalent earthquake load is expressed as the sum of the equivalent earthquake loads acting on the building floors. The additional equivalent earthquake load acting on the floor (top) of the building is calculated as follows for both directions, Turkish Building Earthquake Code Equation (4.22).

$$\Delta F_{NE}^{(x)} = 0.0075 * 5 * 1070 = 40.12K_N \tag{3.8}$$

$$\Delta F_{NE}^{(y)} = 0.0075 * 5 * 740.99 = 27.78K_N \tag{3.9}$$

$$\Delta F_{NE} = 0.0075NV_{IE} \tag{3.10}$$

The rest of the total equivalent earthquake load, except for the ΔF_{NE} force, will be distributed to the floors of the building, including the floor, according to Turkish Building Earthquake Code Equation (4.23).

$$F_{IE}{}^{(X)} = \left(V_{IE}{}^{(X)} - \Delta F_{NE}{}^{(X)}\right) \frac{M_{\prime} H_{\prime}}{\sum_{J-1}^{N} M_{\prime} H_{j}} \tag{3.11}$$

Table 3.9. Equivalent earthquake loads acting on the floors of the 5-storey EBF building

Floor	$\boldsymbol{w_i H_I / \sum W_I H_I}$	$\boldsymbol{F_{IE}{}^{(X)} kN}$	$\boldsymbol{F_{IE}{}^{(Y)} kN}$
Roof	0.2954	32.2	32.18
4	0.2694	25.9	25.93
3	0.2072	19.9	19.95
2	0.1451	14.0	13.96
1	0.0829	8.0	7.98
$\sum$	**1.00**	**100**	**100**

Table 3.10. Equivalent earthquake loads acting on the floors of the 5-storey KBF building

Floor	$\boldsymbol{w_i H_I / \sum W_I H_I}$	$\boldsymbol{F_{IE}{}^{(X)} kN}$	$\boldsymbol{F_{IE}{}^{(Y)} kN}$
Roof	0.2954	108.0	112.52
4	0.2694	195.0	203.18
3	0.2072	262.0	272.92
2	0.1451	308.8	321.74
1	0.0829	335.6	349.64
$\sum$	**1.00**	**1209.5**	**1260.1**

According to Turkish Building Earthquake Code 4.5.10, by assuming there is no torsional irregularity, the equivalent seismic loads acting on the floors are determined by shifting the building width by +5% and 5% in the direction perpendicular to the earthquake direction considered, in order to take into account, the additional eccentricity effect. İt is also fore seen to be applied to the floor mass center. Excessive eccentricities in the (X) direction is given e_x = ±0.05x24.0=±1.2m e_y= ±0.05x24.0=±1.2m

3.3. Methods Used

3.3.1. Calculation of the relative floor offsets.

The control of the relative story drifts will be made according to Turkish Building Earthquake 4.9. For any column, the reduced relative story drift, which expresses the difference in horizontal displacement between two consecutive stories is calculated with the following Equation (3.12).

$$\Delta_1^X = \mu_I^X - \mu_{I-1}^{(X)} \tag{3.12}$$

The highest horizontal displacements at the ends of any column on the two subsequent levels of the building for each earthquake direction are represented by μ_I and μ_{I-1} in this equation due to lower earthquake loads. The $\mu_I$values for each earthquake direction in this example are the displacements largest values. Reduced seismic loads are applied with an additional 5% eccentricity, and numerical values are reported in Table 3.11, and 3.12. In each earthquake direction, the effective relative story drift for the column on the floor of the building will be calculated with the following relation, Turkish building earthquake code equation (4.33).

$$\delta_I^{(X)} = \frac{R}{I}\Delta_i^{(X)} \tag{3.13}$$

Under reduced earthquake loads applied with 5% additional eccentricity in (X) and (Y) directions, the values of μ_I and μ_{I-1} horizontal displacements at each floor obtained by analysis of the structural system are in the third column of Table 3.11, and Table 3.12, consecutively. The reduced relative sway between floors is given in the fourth column of the tables. In the calculations, the effect of the displacements perpendicular to this direction on the resultant displacement is abandoned due to the earthquake loads in the main earthquake direction. Due to the symmetry in both directions, the effect of the aforementioned assumption on the displacements is less than 1% in this building, which has no torsional irregularity.

Table 3.11. Relative story drift control in the (X) direction of the 5-Storey prototype building (EBF).

Floor	$\boldsymbol{H_I (m)}$	$\mu_{ix}(m)$	$\Delta_{ix}(m)$	$\boldsymbol{\delta_{ix} = R/_{I.}\Delta_{ix}\ (m)}$	$\boldsymbol{\delta_{ix}}$**/h** $\boldsymbol{(m)}$
Roof	3.0	15.68	1.96	15.67	0.0052
4	3.0	13.72	2.79	22.32	0.0074
3	3.0	10.93	3.34	26.74	0.0089
2	3.0	7.59	3.01	24.10	0.0080
1	4.0	4.57	4.57	36.58	0.0091

Table 3.12. Relative story drift control in the (X) direction of the 5-Storey prototype building (KBF).

Floor	$\boldsymbol{H_I}$ $\boldsymbol{(m)}$	μ_{ix} $\boldsymbol{(m)}$	Δ_{ix} $\boldsymbol{(m)}$	$\boldsymbol{\delta_{ix} = R/_{I.}\Delta_{ix}}$ $\boldsymbol{(m)}$	$\boldsymbol{\delta_{ix}}$**/h** $\boldsymbol{(m)}$
Roof	3.0	29.93	3.39	27.09	0.0090
4	3.0	26.55	5.44	43.49	0.0145
3	3.0	21.11	6.92	55.35	0.0185
2	3.0	14.19	7.13	57.02	0.0190
1	4.0	7.06	7.06	56.52	0.0141

As can be seen from the tables, the largest values of the $\delta i/_{hi}$ ratios are in the (x) and (y) directions where $\left(\delta_{ix}/_{hi}\right)_{maks}$ = 0.0091 and $\left(\delta_{iy}/_{hi}\right)_{maks}$ = 0.0190

According to Turkish Building Earthquake Code 4.9.1.3, if the infill walls are made of brittle material and the frame members have flexible joints between them, the facade elements are connected to the outer frames with flexible connections or the infill wall element is independent from the frame. Based on these restrictions, the following condition must be met Turkish Building Earthquake Code Equation (4.34b) given in Equation (3.14)

$$\lambda \frac{\delta^{(x)}{}_{imaks}}{h_i} \leq 0.016\text{k} \quad (3.14)$$

The coefficient λ is the ratio of the elastic design spectral acceleration of the DD-3 earthquake ground motion, the dominant vibration period of the building in the earthquake direction, to the elastic design spectral acceleration of the DD-2 earthquake ground motion. The coefficient (k) will be taken as 0.5 for steel buildings. Accordingly, the coefficient λ was obtained for the X direction as follows, Turkish Building Earthquake Equation (2.2) for DD-2 direction $(5 - \text{storey building})$

$$T_B = 0.35s < T = 0.6s < T_L = 6s \; for \; S_{ae}(T) = \frac{S_{D1}}{T} = \frac{0{,}411}{0{,}6} = 0{,}685 \quad (3.15)$$

DD-3 for earthquake ground motion

$$T_B = 0.35s < T = 0.6s < T_L = 6s \; for \; S_{ae}(T) = \frac{S_{D1}}{T} = \frac{0.159}{0{,}6} = 0.265 \quad (3.16)$$

$$\lambda = \frac{0.265}{0.685} = 0.386 \quad (3.17)$$

$$\lambda \frac{\delta^{(x)}{}_{imaks}}{h_i} \leq 0.016 \frac{0.5}{0.386} = 0.0207 \quad (3.18)$$

$$\left({}^{\delta_{ix}}/_{hi}\right) maks = 0.0092 < 0.0207 \quad (3.19)$$

Since the condition is met the relative story drifts satisfy the Turkish Building Earthquake Code condition

For the (Y) direction

for DD-2 direction $(5 - \text{storey building})$

$$T_B = 0.35s < T = 0.7s < T_L = 6s \; for \; S_{ae}(T) = \frac{S_{D1}}{T} = \frac{0.411}{0.7} = 0.587 \quad (320)$$

DD-3 for earthquake ground motion

$$T_B = 0.35s < T = 0.7s < T_L = 6s \; for \; S_{ae}(T) = \frac{S_{D1}}{T} = \frac{0.159}{0.7} = 0.227 \qquad (3.21)$$

$$\lambda = \frac{0.277}{0.587} = 0.386 \qquad (3.22)$$

$$\lambda \frac{\delta^{(x)}{}_{imaks}}{h_i} \leq 0.016 \frac{0.5}{0.386} = 0.0207 \qquad (3.23)$$

$$\left({}^{\delta_{ix}}/_{hi} \right) maks = 0.0088 < 0.0207 \qquad (3.24)$$

Since the condition is met the relative story drifts satisfy the Turkish Building Earthquake condition.

3.3.2. Calculation of the second order effects

The second order indicator value representing the second order effects on each floor for the earthquake direction taken into consideration in accordance with Turkish Building Earthquake Equation 4.9.2 is calculated as follows.

$$\emptyset^{(x)}_{(1)} = \frac{\left(\Delta_i{}^{(x)}\right) \Sigma^{N}_{K=\mathrm{i}} WK}{V^{(X)}{}_i h_i} \qquad (3.25)$$

- For each direction, second order indicator value $\emptyset^{(x)}{}_{(1)}$, i is calculated according to Turkish Building Design Regulation 4.9.2 for all floors. If maximum $\boldsymbol{\emptyset_{imax}}^{(\boldsymbol{x})}$, satisfies the following condition; Second order effects are not taken into account in determining the internal forces based on design.
- C $_{\mathrm{h}}$ is a coefficient calculated based on the nonlinear hysteretic behavior of the structural system and 0.5 is taken for steel structure as per Turkish Building Design Regulation 2018 4.9.2.2.
- If the condition given in Equation 3.25 is not fulfilled, only the internal forces obtained from the horizontal earthquake effects are increased with the $\boldsymbol{\emptyset_{imax}}$ coefficient determined by the following equation.

Table 3.13. Second order indicator values with (E B F).

floor	$h_i(Mm)$	$V_{ix}(kN)$	$V_{iy}(kN)$	$\sum_{k=1}^{N} wk$	(Δ_{ix}) (mm)	(Δ_{iy}) (mm)	$\emptyset_1^{(x)}$	$\emptyset_1^{(x)}$
Roof	3,0	32.2	32.18	288	1.929	1.703	0.005	0.0051
4	3,0	58.1	58.11	611	3.037	2.929	0.0106	0.0103
3	3,0	78.1	78.06	934	3.801	3.782	0.0152	0.0151
2	3,0	92.0	92.02	1257	3.764	3.373	0.0171	0.0154
1	3,0	100	100	1579	4.402	4.377	0.0174	0.0173

Table 3.14. Second order indicator values with (K B F).

Floor	$h_i(Mm)$	V_{ix} (kN)	V_{iy} (kN)	$\sum_{k=1}^{N} wk$	(Δ_{ix}) (mm)	(Δ_{iy}) (mm)	$\emptyset_1^{(x)}$	$\emptyset_1^{(x)}$
Roof	3,0	32.2	32.18	288	1.929	1.703	0.0017	0.0015
4	3,0	58.1	58.11	611	3.037	2.929	0.0032	0.0029
3	3,0	78.1	78.06	934	3.801	3.782	0.0045	0.0043
2	3,0	92.0	92.02	1257	3.764	3.373	0.0051	0.0044
1	3,0	100	100	1579	4.402	4.377	0.0052	0.0049

If the maximum value of the second order indicator values calculated for all floors according to Turkish Building Earthquake Equation 4.9.2.2, satisfies the condition given in Turkish Building Earthquake Equation (4.36), the second order effects do not need to be taken into account in the calculation of the internal forces based on the design.

$$\emptyset_{imax}{}^{(x)} \leq 0.12\frac{D}{C_h R} \tag{3.26}$$

In this case, local second order effects can be taken into account in the element design according to the steel regulations in force. C_h=1 will be taken for steel buildings Where .$\boldsymbol{\emptyset_{imax}}{}^{(x)} = 0.009 \leq 0.12\frac{2}{1X5}0.048$ and $\emptyset_{\mathrm{imax}}{}^{(\mathrm{x})} = 0.0173 \leq 0.12\frac{2.5}{1X8}0.038$

Since it satisfies the condition, it is sufficient to evaluate the second-order effects according to design of steel structures. It has been demonstrated by computation that braced frames are rigid frames and that second-order effects, aside from the earthquake effective insignificant.

4. ANALYSES and DISCUSIONS

In this section several nonlinear static pushover analyses have been performed on the 5-storey EBF and KBF prototype building using SAP 2000 in order to evaluate the seismic performance of these structures. Analyses cover the determination of the system performance, the shear capacity curves, the performance points, the plastic hinges and the stabilities of the both type of braced systems. Results obtained from these analyses are compared to discuss the seismic performance of both the type of braced systems in order to determine which the type of braced system has a better seismic performance.

4.1 Pushover Analysis

Pushover analysis can be described as a static nonlinear analysis performed to develop capacity curve of the building. It is based on the nonlinear static analysis that determines the progressive yielding of the structures subjected to a lateral load. The magnitude of the lateral load increases up until the building reaches the target displacement. This target displacement is determined by the top displacement when the building is subjected to design level ground excitation.

4.1.1 Determination of target displacement

Target displacement determines building performance criteria. Three main approaches are generally used to determine the target displacement (maximum inelastic displacement of the structure) of the steel framed structure. They are namely Capacity spectrum method (ATC-40), displacement method (FEMA-356) and displacement modification (FEMA 440). In this study the Capacity spectrum method is chosen to determine the target displacement of the both the type of braced systems.

Capacity spectrum method started with conversion of the capacity curve and demand response into capacity spectrum (Sa vs Sd). The results are plotted in acceleration-displacement response spectrum (ADRS) format (ATC-40)

The general process for converting the capacity curve to capacity spectrum depends on calculating the modal participation factor (MPF1) and the modal mass coefficient(α1), using the following equations:

$$\mathrm{MP}F_1 = \frac{\Sigma_{M_I \theta_{I1}}}{\Sigma_{M_I \theta^2{}_{I1}}} \tag{4.1}$$

$$\alpha = \frac{\left[\Sigma_{M_I \theta_{I1}}\right]}{\left[\sum_{I=1}^{N} m_i\right]\left[\Sigma_{M_I \theta^2{}_{I1}}\right]} \tag{4.2}$$

Where $m_i$mass assigned to level i, $\emptyset_{i1}$amplitude of mode 1 at level i, N the number of stories in building Then, Sa and Sd are calculated for every point on the capacity curve using the following equations:

$$\frac{S_a}{g} = \frac{V_b}{\omega}\frac{1}{\alpha} \tag{4.3}$$

$$S_d = \frac{\Delta_{roof}}{MPF_1 \emptyset_{roof1}} \tag{4.4}$$

where V_b base shear, w building total weight, Δ_{roof} roof displacement

To convert a demand spectrum from S_a and T format to ADRS format, it is required to calculate the value of S_a for each point of the curve using the following equation:(Hakim et al., 2014)

$$S_D = \frac{T^2 S_a}{4\pi^2} \tag{4.5}$$

4.1.2 Determination of system performance with pushover analysis

In order to determine system performance of the both type of braced systems first, shear force joint of the plastic hinges is defined in SAP2000 (Figures 4.2 and 4.5) In eccentric braced and knee braced frame systems, they are introduced as the first elements where damage will be observed in the tie beams as a requirement of the capacity design principle. In these designed systems, it is aimed that the inelastic deformation behavior of the tie beam by plasticizing with the earthquake load and the other elements remain in the elastic region.

For this reason, the definitions of plastic hinges in the link beams, which are the weakest elements of the building, are made separately for each floor in the SAP2000 program. Since the tie beams are not designed as short tie beams, the plastic hinges are defined as plastic shear joints since the yielding situation will be due to the shear force (Figure 4.1).

While defining plastic hinges, damage limits are defined according to the performance levels given in the regulation Turkish Building Earthquake Regulation-2018. The limit values of the rotation angles of the short link beams are used in this definition. In the analyses, the damage limits of the plastic cutting joints are defined as displacement. For this reason, the link beam rotation angles given in the regulation were converted into horizontal displacements of the link beam by multiplying the link beam length and defined in the program (Figure 4.2). Short link beam rotation angle limit values for performance levels are given in Table 3.1 in the previous sections.

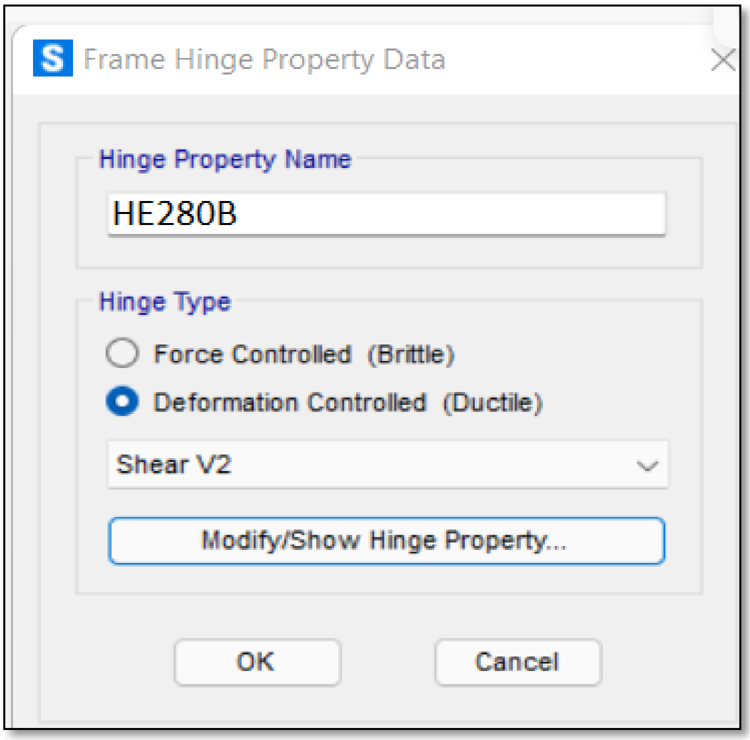

Figure 4.1. Plastic shear force joint defined in the 1st floor tie beams in the SAP2000 program

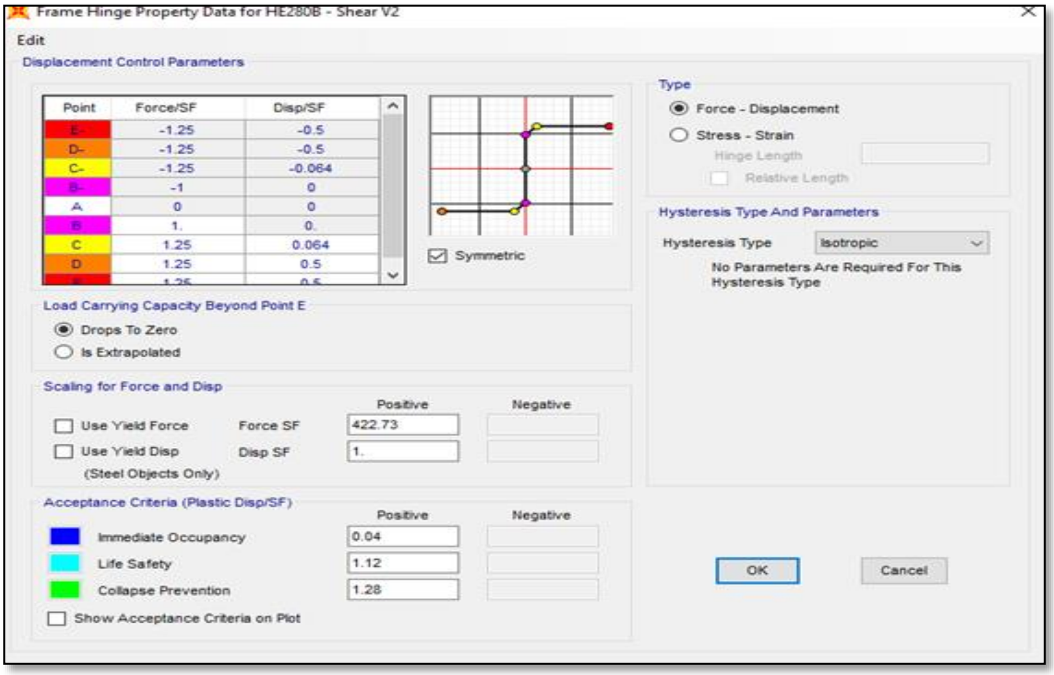

Figure 4.2. Five floors of eccentric baraced frame with plastic hinges

HE280B plastic shear force joint definition of the plastic hinge defined in the 1st and 2nd floor beams of the 5-storey Eccentrically Brace Frame prototype building in the SAP 2000 program.

After the defining the shear force joint definition of the plastic hinge in SAP2000 the pushover analyses for both the 5-storey eccentric Braced frame and knee braced frame prototype buildings are performed. Axis transformations were made to fit the earthquake horizontal elastic design spectrum of the coordinates of the 5-storey prototype buildings.

Pushover curves for the 5-storey of eccentric braced frame and knee braced frame are presented in Figures 4.3 and 4.4. These curves represent the global behavior of the frame with stiffness and ductility. For these curves the yield strength values at the interface used when defining the plastic shear joint are entered in 'kN' and the displacement values in 'm'.

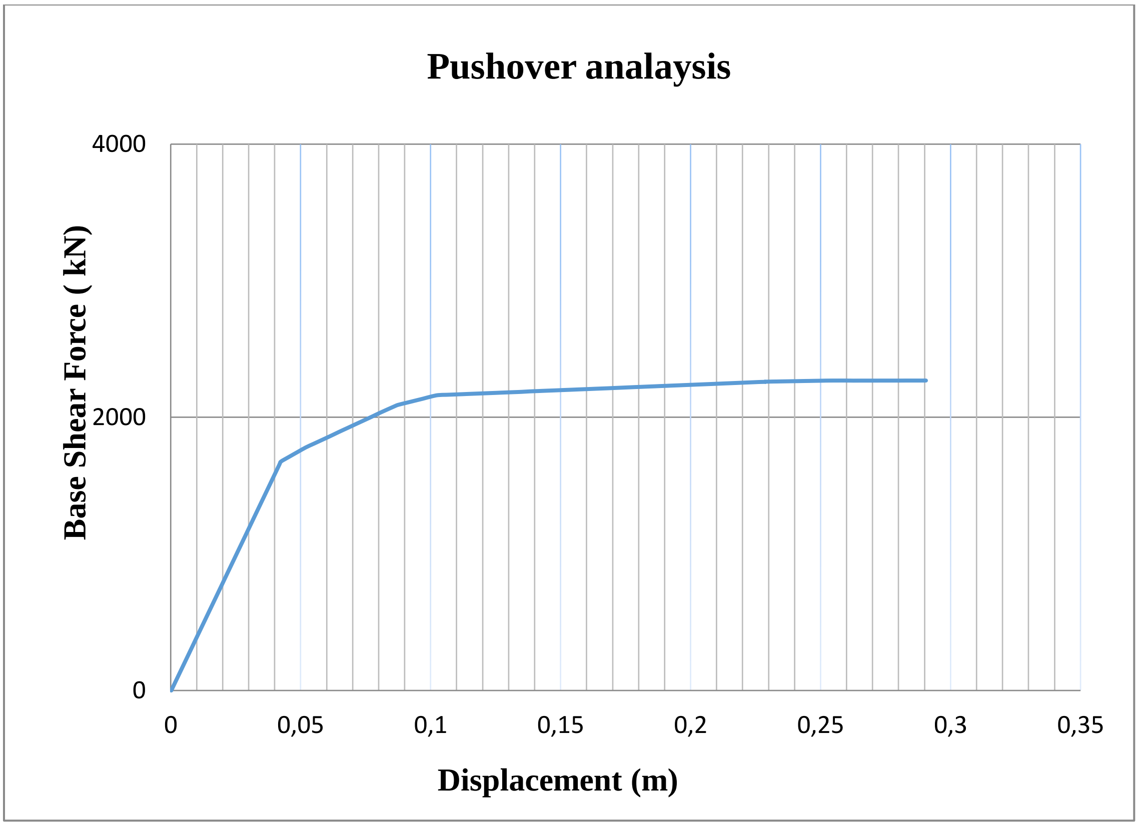

Figure 4.3. The pushover analysis of 5-storey eccentric brace frame prototype building

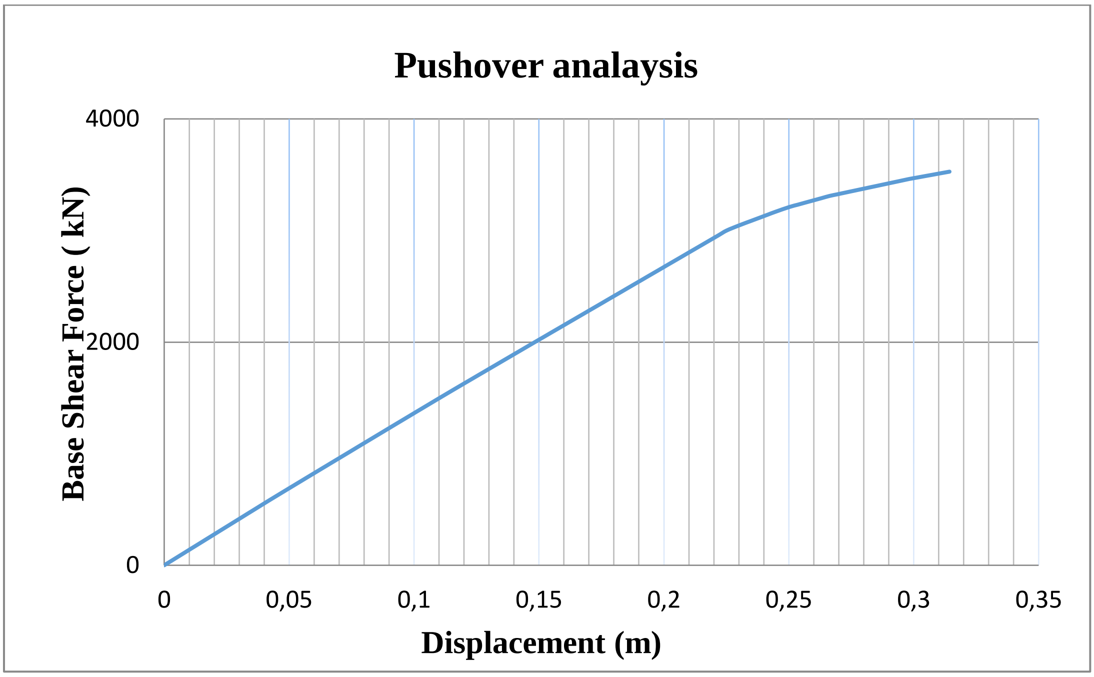

Figure 4.4. The pushover analysis of 5-storey knee brace frame prototype building

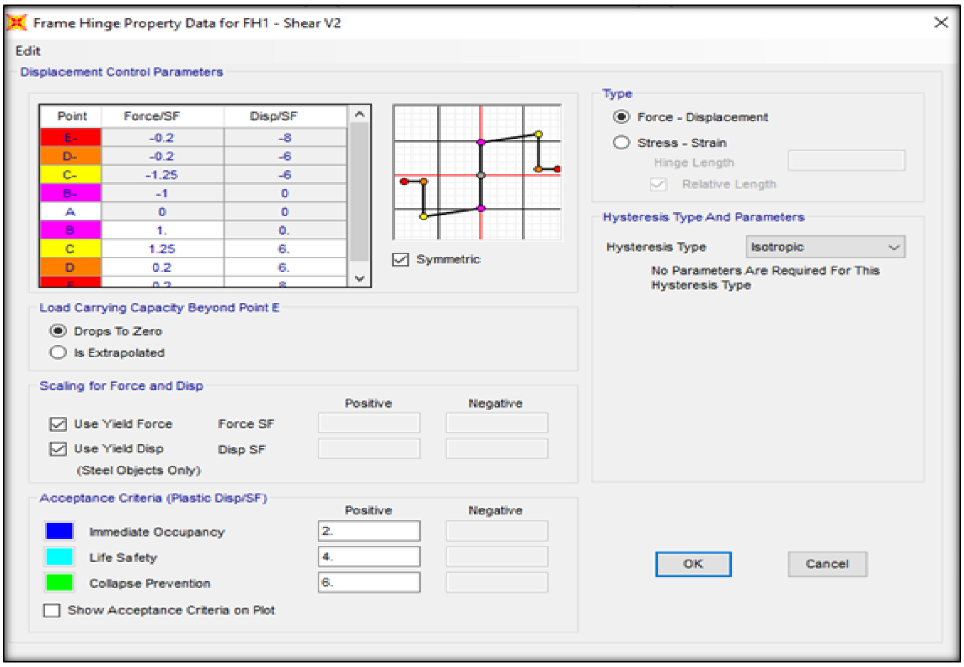

Figure 4.5. Five floors of knee braced frame with plastic hinges

The plastic hinge defined in the floor braces of the 5-storey knee braced frame prototype building in the SAP 2000 program. As one can see that the results obtained from pushover curve for the eccentrically braced frame shows the maximum displacement of 0.28 m and maximum base shear force of 2719 kN. While as pushover curve of knee braced frame shows the maximum displacement of 0.30 m and maximum shear force of 2301 kN.

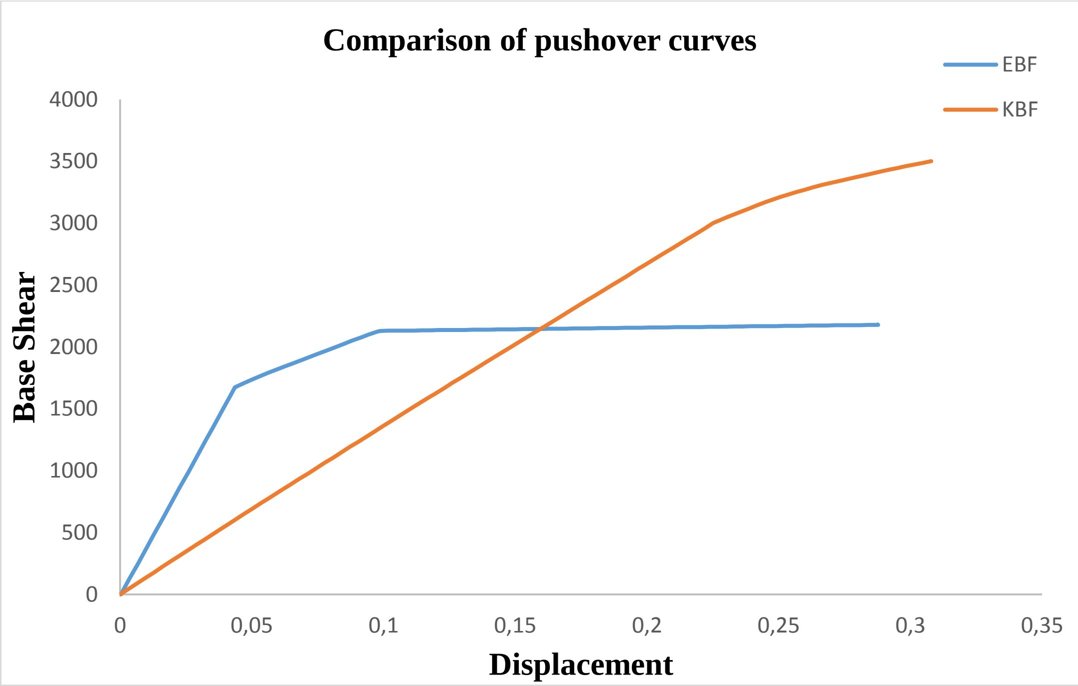

Figure 4.6. Comparison of KBF and EBF pushover curves.

As clearly seen from these two curves that the base shear capacity of knee braced frame is higher than that of the eccentric braced frame. There is almost 65% difference between the peak load calculated for EBF and KBF.

One of the main differences observed between two frames is in the initial stiffness. The eccentric braced frame has a large value of initial stiffness as compared to the knee braced frame. Moreover, the eccentric braced frame is seen to produce the post peak behavior as a constant line with practically zero stiffness after the yield (almost elastic - perfect plastic behavior), whereas in case of the knee braced frame the stiffness gradually decreases after the yield (almost elastic - linear plastic hardening behavior).

4.1.3. Determination of the capacity curve with pushover analysis

The capacity spectrum curves are obtained by using SAP2000 analyses performed for EBF and KBF. Capacity spectrum method is used to convert of the capacity curve and demand response into capacity spectrum (Sa vs Sd). The results are plotted in acceleration- displacement response spectrum (ADRS) format (ATC-40) (Figures 4.7 and 4.8)

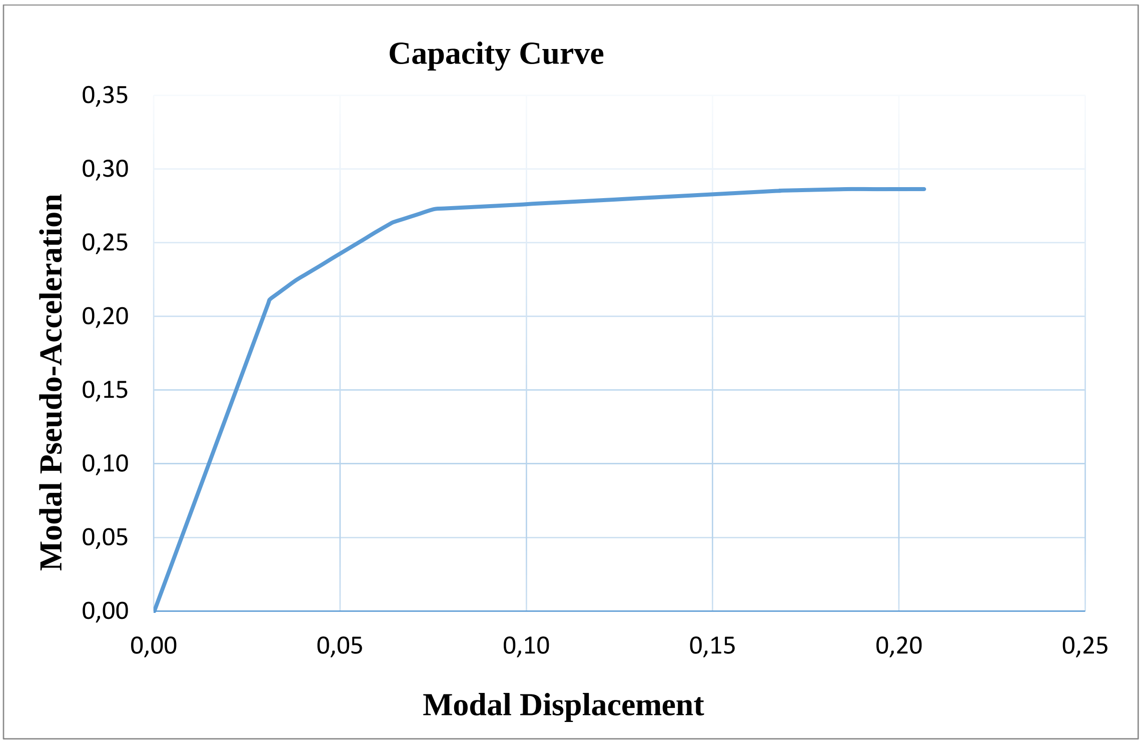

Figure 4.7. Capacity curve for the EBF.

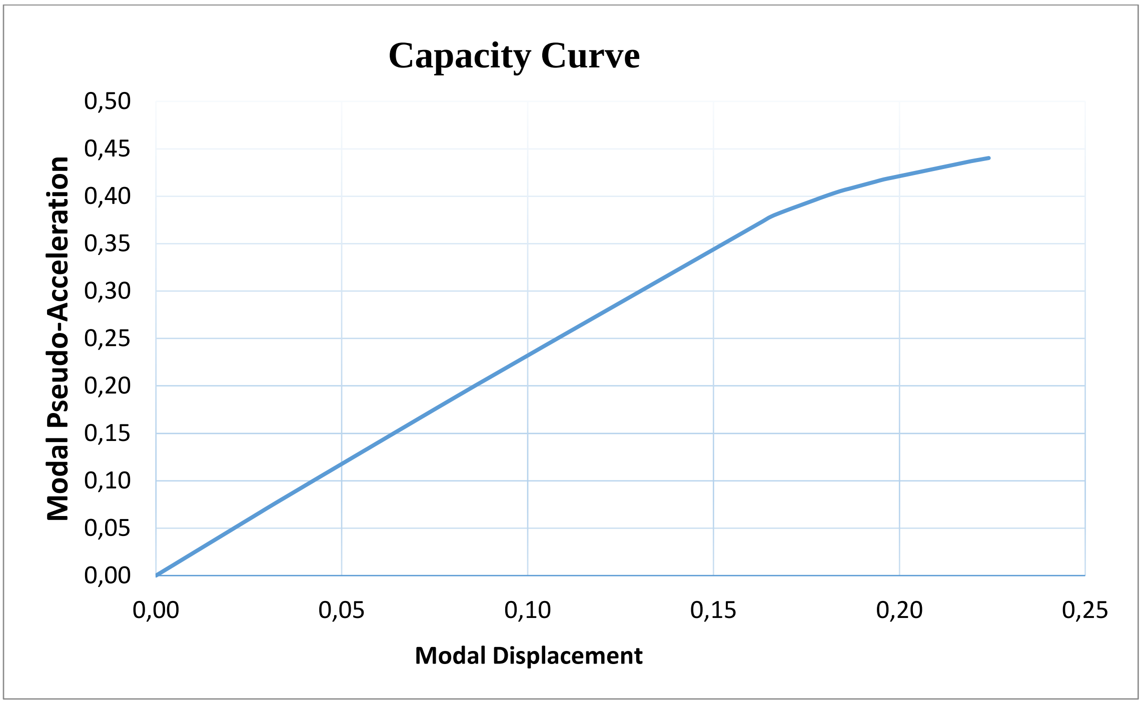

Figure 4.8. Capacity curve for the EBF.

The difference of two figures with model displacement in figure 4.7 and figure 4.8 related in maximum model displacement for eccentric braced frame is 0.21m and maximum model displacement for a knee braced frame is 0.22m. The maximum acceleration for eccentric braced frame is 0.28m and maximum acceleration for a knee braced frame is 0.44m.

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4.1.4. Determination of the performance points with pushover analysis.

The method (ATC-40) used to determine performance points that is defined as the intersection of the capacity curve of the building with the response spectrum of the ground motion which represents the demand curve. Figures 4.9 and 4.10 shows demand spectrum curves, capacity curves and performance points for both type of the braced systems.

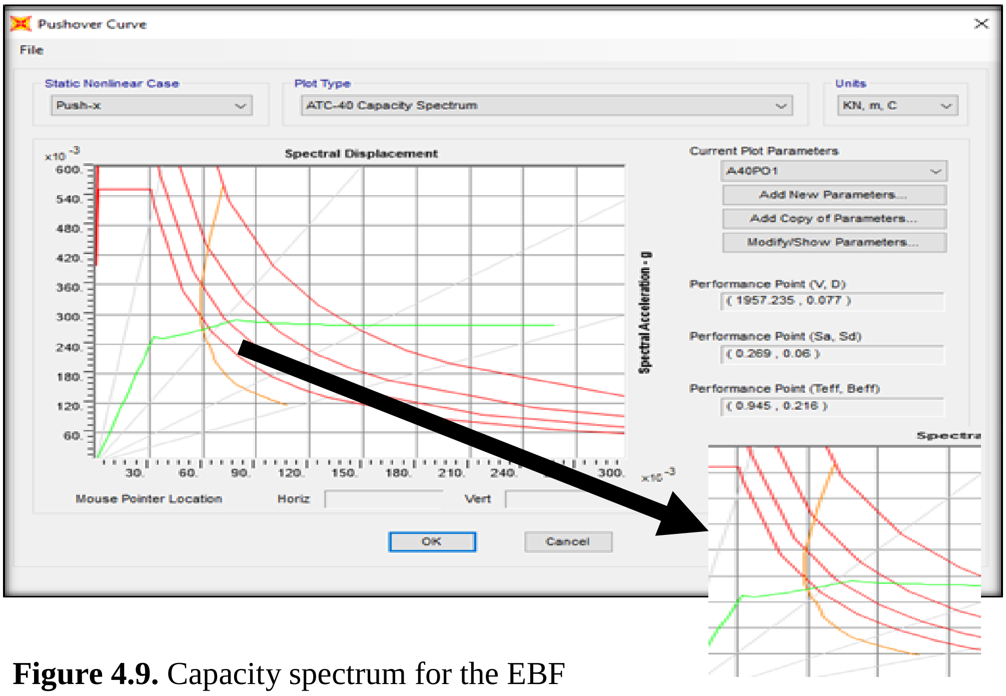

Figure 4.9. Capacity spectrum for the EBF

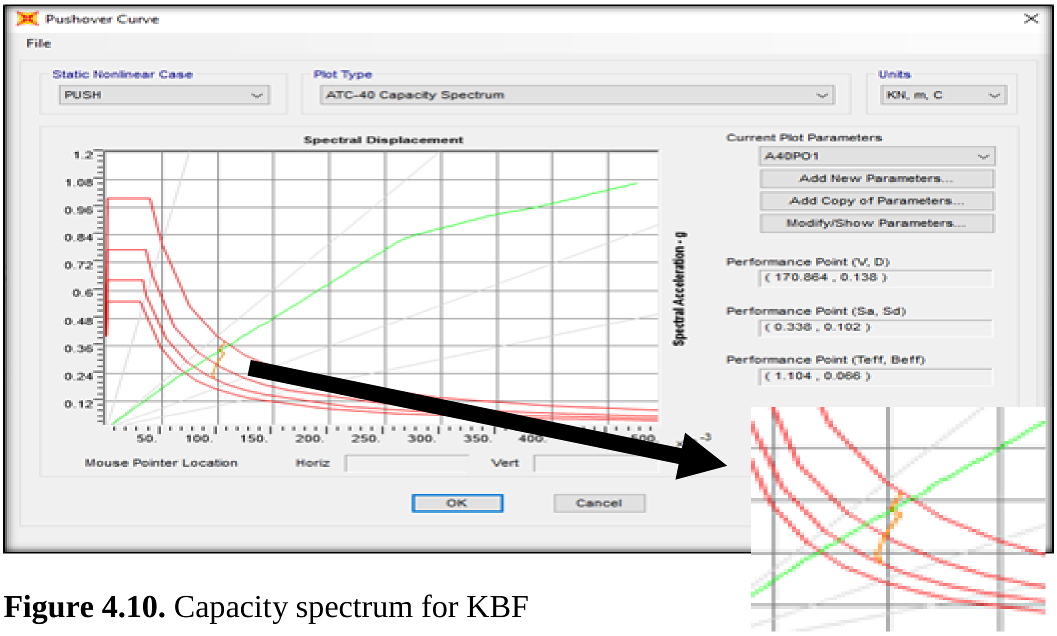

Figure 4.10. Capacity spectrum for KBF

In these figures red colored curves represent the elastic demand spectrum, while the green curves obtained from SAP2000 analyses represent capacity spectrum and the yellow line is used to locate position of the performance point. The intersection of the yellow line (demand) and the green curve (capacity) is the performance point.

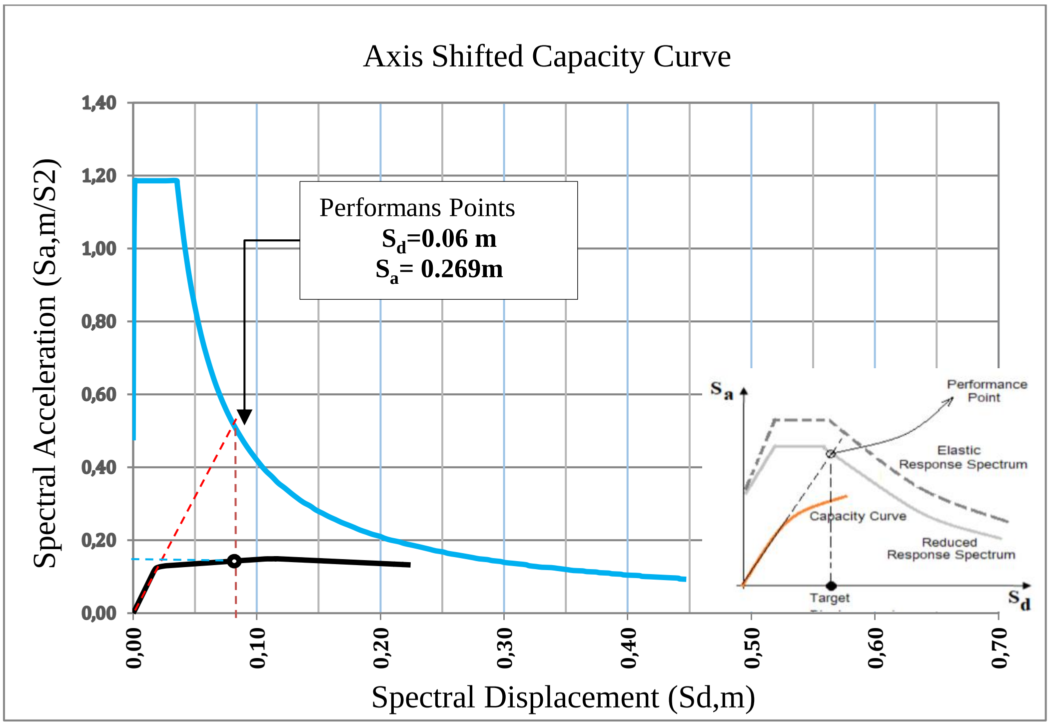

Figure 4.11. The performance point of the prototype building designed according to the 5-storey deformation design-evaluation according to shape change method (EBF).

As seen in Figure 4.11, eccentric braced frame the performance point of the structure was found as spectral acceleration Sa = 0.269 m/s2 and the peak displacement S_d = 0.06 m. The damage conditions of the plastic hinges formed in this structure are shown in Figure 4.13.

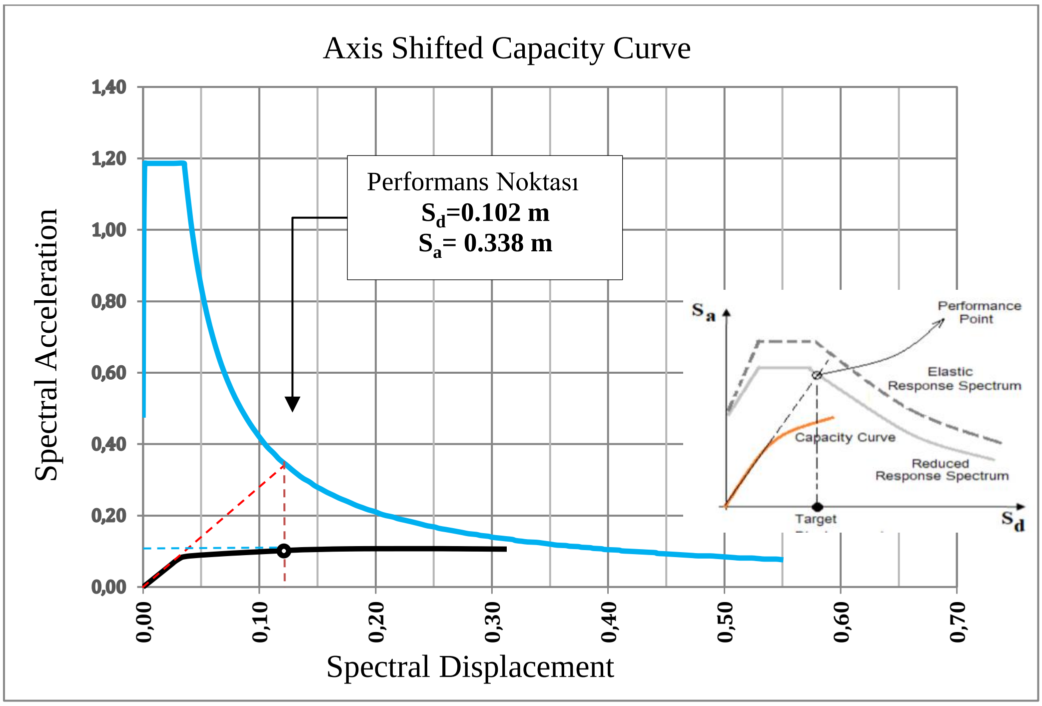

Figure 4.12. The performance point of the building designed according to the 5-storey (KBF).

As seen in Figure 4.12, a knee braced frame the performance point of the structure was found as spectral acceleration Sa = 0.338 m/s2 and the peak displacement S_d = 0.102 m. The damage conditions of the plastic hinges formed in this structure are shown in Figure 4.14.

Pushover analysis is used to derive capacity curves, which are then turned into spectral displacement and spectral acceleration (Sa & Sd) curves from base shear and top displacement coordinates. Graphical representations of the expected seismic performance of the models are shown in Figures 4.11 and 4.12 by the intersection of a capacity curve and an elastic response spectrum.

$$a_1^{(X,K)} = \frac{V_{[X]}^{(X,K)}}{V_{[X]}^{(X,1)}} \tag{4.6}$$

$$a_1^{(X,K)} = \frac{u_{Nx1}^{(X,K)}}{\emptyset_{Nx1}^{(1)} \Gamma_1^{(x,1)}} \tag{4.7}$$

To find the performance point, Turkish Building Earthquake Regulation Equation 5B.3 is used in the coordinate transformation to the base shear force. The $V_{tx1}^{(X,K)}$ term of these curves is transformed into Turkish Building Earthquake Regulation Equation 5B.3 and the $u_{NX1^{(X,K)}}$ term is transformed into Turkish Building Earthquake Regulation Equation 5B.4.

4.1.5 Determination of the plastic hinges with pushover analysis

Plastic hinge distributions for the KBF and EBF 5-story structure resulted from nonlinear static nonlinear pushover analyses are shown in Figure 4.13, and Figure 4.14

In all the configurations of EBF, it is seen that nonlinear hinges are formed only in the link elements which are assigned with shear hinges and all other elements are in the elastic range (Figure 4.13). This behavior of frames validates the capacity design principles.

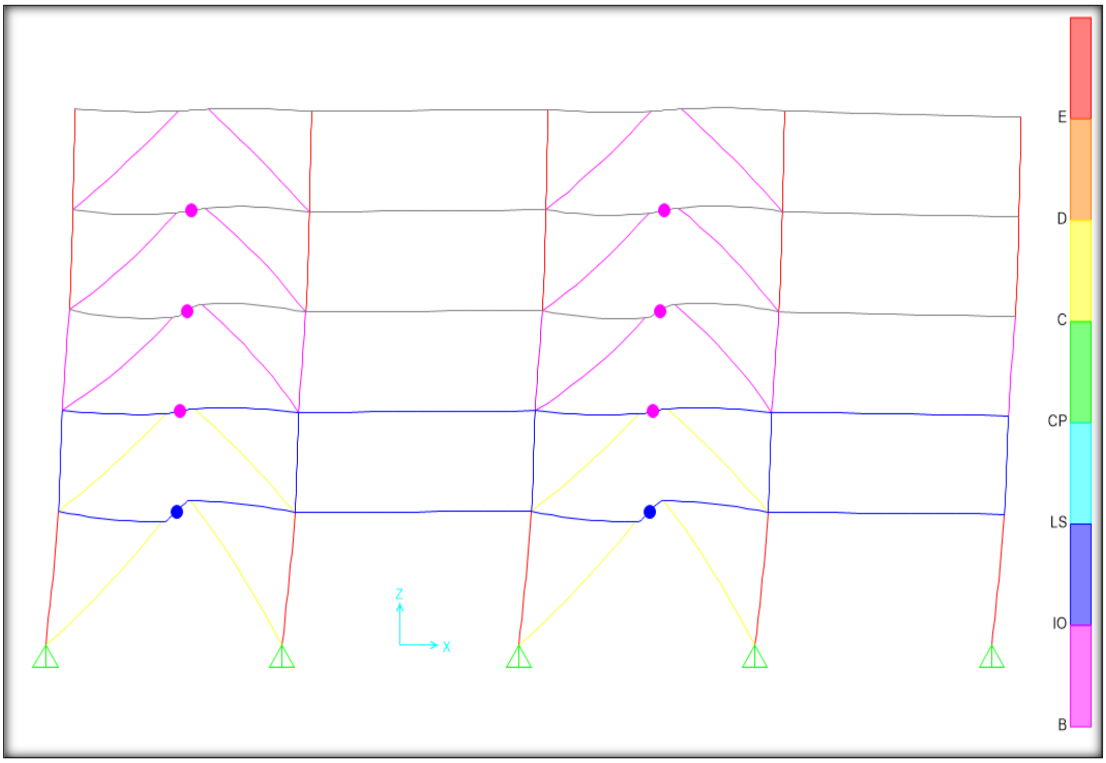

Figure 4.13. Plastic hinge distribution for EBF

The EBF plastic hinges generally started to from the beam between the beams, the produced damages in the links of EBF systems can be very expensive to repair. On the other hand, the KBF plastic hinges generally started to from in links between the braces, which are the main structural element to dissipate energy in KBF systems. It is observed that the first hinge was formed in the knee elements for KBF system (Figure 4.14). Knee element is considered a secondary or fuse element in the lateral resistant system, which can be easily repaired or replaced easily after a server earthquake.

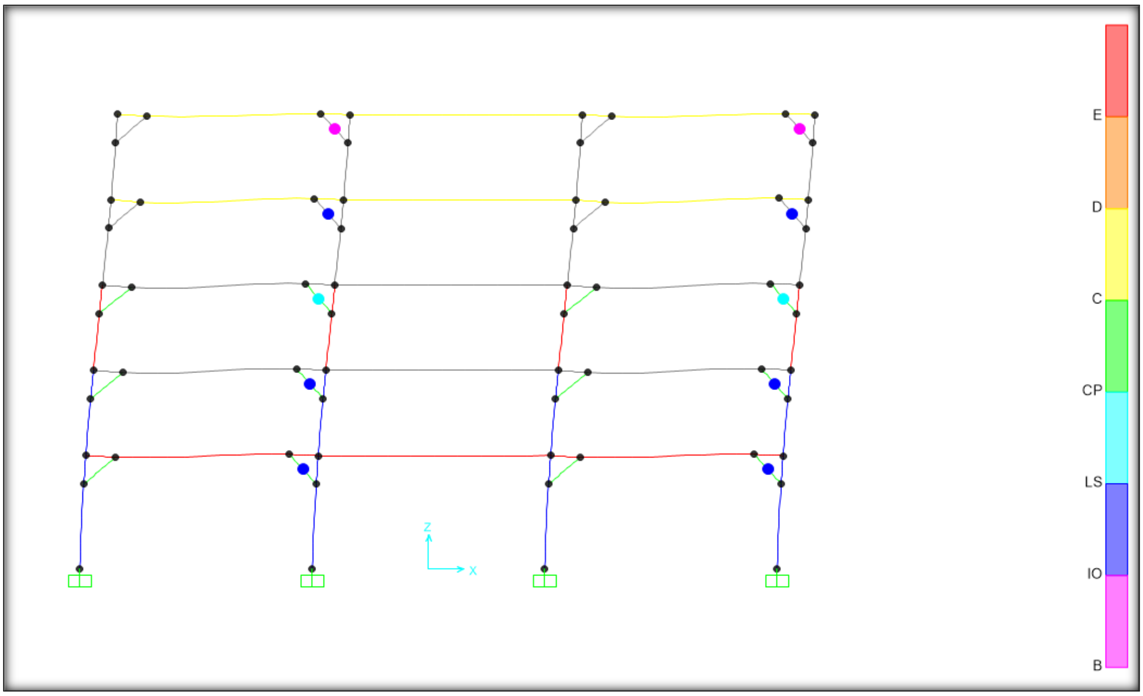

Figure 4.14. Plastic hinge distribution for KBF

As one can see in Figure 4.13-, and Figure 4.14 that the horizontal load-carrying capacity of the vertical carrier system has slightly increased in the horizontal load performance analysis. EBF frames enter into the inelastic range after the yielding much earlier than KBF. In static pushover analysis, the displacement of the 5-story eccentric braced frame building was 0.06, while the 5-story knee braced frame building was 0.102. The obvious difference was observed in the linear region of the load-displacement curve.

5. CONCLUSION

Within the scope of this study, prototype buildings with steel structure frame system, consisting of 5-EBF and 5-KBF floors, are analyzed by designed according to Turkish building earthquake code. In this section, the results obtained from the performance analyzes of the structures are summarized.

- It was found that the shear capacity of knee braced frame is higher than that of eccentrically braced frame. There is almost 65% difference between the peak load calculated in EBF and KBF.
- The area under force-displacement diagram of KBF is much large as compared to EBF. Which means the energy dissipating capacity of KBF is better than EBF.
- As an energy dissipating system, the KBF combines excellent ductility and lateral stiffness and is easy for application to rehabilitation if earthquake damaged the buildings.
- The pushover graph of structures with KBF is higher than that of EBF. Thus, indicating that the KBF enters the nonlinear range later than EBF. Graphs also indicate that EBF braces have more plasticity than KBF.
- As results of plastic hinge distributions analyses it is found that the horizontal load-carrying capacity of the vertical carrier system has slightly increased in the horizontal load performance of KBF systems. On the other hand, EBF system enter into the inelastic range after the yielding much earlier than KBF.

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