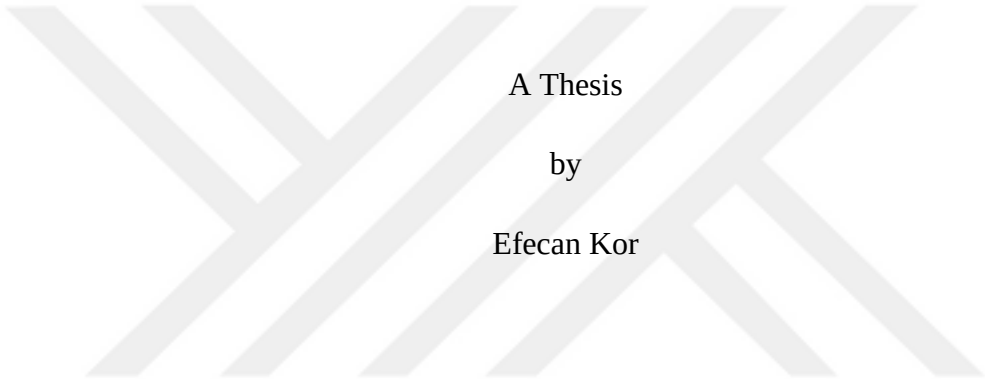


**SEISMIC PERFORMANCE EVALUATION OF
CONCENTRIC X-BRACED STEEL FRAMES
DESIGNED TO THE TURKISH BUILDING
EARTHQUAKE CODE 2018**



A Thesis

by

Efecan Kor

Submitted to the
Graduate School of Sciences and Engineering
In Partial Fulfillment of the Requirements for
the Degree of

Master of Science

in the
Department of Civil Engineering

Özyeğin University
December 2021

Copyright © 2021 by Efecan Kor

**SEISMIC PERFORMANCE EVALUATION OF
CONCENTRIC X-BRACED STEEL FRAMES
DESIGNED TO THE TURKISH BUILDING
EARTHQUAKE CODE 2018**



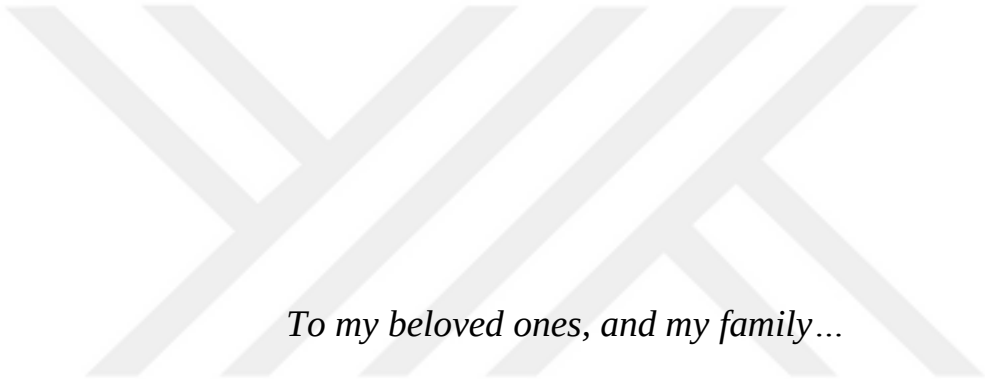
Approved by:

Asst. Prof. Dr. Yiğit Özçelik, Advisor,
Department of Civil Engineering
Özyeğin University

Prof. Dr. Gülay Altay
Department of Civil Engineering
Özyeğin University

Asst. Prof. Dr. Ziya Müderrisoğlu
Department of Civil Engineering
Beykent University

Date Approved: 27 December 2021



To my beloved ones, and my family...

ABSTRACT

Concentrically braced steel frame systems are frequently used lateral load resisting frame systems, especially in highly seismic regions. Those systems are preferred due to their significant contribution of lateral stiffness and strength of the structure. Other advantages of CBFs can be expressed as their elastic stiffness that provides to satisfy the serviceability deformation limits compared to other lateral load resisting frames, as well as their contribution to the ductility of the structure by allowing inelastic deformations within the system. However, they must be designed and detailed carefully to maintain the general stability of the system and to avoid a brittle and sudden local failure. These details and related design approaches have been specified by various design regulations and guidelines, including the Turkish Building Earthquake Code 2018 (TBEC 2018) which came to force in January 2019. With this code, not only a number of new design approaches that did not take place in the previous version were introduced, but also various definitions and approaches were revised. These changes can be expected to influence the dynamic behavior and seismic performance of CBFs designed per TBEC 2018. From this perspective, in this research it is aimed to investigate the lateral behavior and seismic performance of CBFs. A set of CBFs were designed to TBEC 2018 and a nonlinear numerical model was developed to simulate the designed CBFs. Afterwards, several nonlinear static and dynamic analysis procedures were followed to obtain and evaluate the dynamic behavior and seismic performance of CBF systems. Results and findings are reported at the end of each study and summarized in the conclusion.

ÖZET

Merkezi çaprazlı çelik çerçeve tipi yanal yük taşıyan sistemler özellikle sismik hareketliliği ve riski yoğun olan bölgelerde sıkça karşılaşılan çerçevelerdir. Bu sistemler, sağladığı yüksek seviye rijitlik ve dayanım nedeniyle tercih edilirler. Aynı zamanda elastik rijitlikleri sayesinde özellikle servis yükleri altında deformasyon limitlerini diğer yanal yük taşıyıcı sistemlere kıyasla daha kolay sağlamaları, aynı zamanda da plastik yerdeğiştirmelere izin verip yapının sünek davranmasını sağlamaları bu sistemlerin diğer avantajlarından. Fakat yapının genel stabilitesini korumak ve yerel burkulmalardan doğabilecek gevrek ve ani burkulma veya yırtılmaları engellemek amacıyla bu sistemlerin sismik tasarım ve detaylandırması dikkatli bir şekilde yapılmalıdır. Bu detaylar ve ilgili tasarım yaklaşımları, Türkiye Bina Deprem Yönetmeliği 2018'in de dahil olduğu (TBDY 2018) çeşitli tasarım yönetmelikleri ve yönergelerince belirlenmiştir. TBDY 2018, Ocak 2019'da yayınlanmış ve bu yönetmelikle birlikte önceki yönetmelikte yer almayan bir dizi yeni tasarım yaklaşımları getirilmiş, aynı zamanda bazı kısımlar da revize edilmiştir. Bu revizyonların, TBDY 2018'e göre tasarlanan merkezi çaprazlı çelik çerçevelerin dinamik davranışlarını ve sismik performanslarını etkilemesi beklenebilir. Buradan yola çıkarak, bu araştırmada/tezde merkezi çaprazlı çelik çerçevelerin yanal davranışları ve deprem performanslarının incelenmesi hedeflenmiştir. Çeşitli bir dizi merkezi çaprazlı çelik çerçeve tasarımları TBDY 2018 göz önünde bulundurularak gerçekleştirilmiş ve bu tasarımlar doğrusal olmayan bir numerik model ile modellenmiştir. Sonrasında çeşitli statik ve dinamik analiz prosedürleri takip edilerek bu modellerin dinamik davranışları ve deprem performansları ölçülüp değerlendirilmiştir.

ACKNOWLEDGMENTS

First of all, I would like to thank to my supervisor, Dr. Yiğit Özçelik, for his contribution in this research and his patience throughout my master's education. Also, all professors who I had the distinct honor of participating their classes and have a great impact on me, deserve another appreciation. And it is not possible to forget the support of the staff of Özyeğin University.

Special thanks go to the esteemed members of the thesis committee.

I have always felt the most precious support and never-ending love from my friends and my family, who are ready to dedicate themselves for me unconditionally. They will always be in my heart, and I will always be grateful...

TABLE OF CONTENTS

ABSTRACT.....	iv
ÖZET	v
ACKNOWLEDGMENTS	vi
LIST OF TABLES	ix
LIST OF FIGURES	x
I INTRODUCTION	1
1.1 Overview	5
II REVIEW OF THE RESEARCH AVAILABLE ON CONCENTRICALLY BRACED FRAMES IN THE LITERATURE	7
2.1 Brace Buckling.....	7
2.2 Balanced Design Procedure (BDP).....	10
2.3 Brace Fracture	14
2.4 Incremental Dynamic Analysis (IDA)	15
2.5 Overview on FEMA P-695 Quantification Methodology.....	18
2.6 Experimental Studies on Concentrically Braced Frame Systems.....	19
III DESIGN PROCEDURE.....	21
3.1 Design Excel	24
IV NONLINEAR ANALYTICAL MODEL	36
4.1 Model Development on OpenSees.....	36
4.2 Parallel Analyses on DesignSAFE.....	43
V SEISMIC PERFORMANCE ASSESSMENT OF CONCENTRICALLY BRACED FRAMES DESIGNED TO TBEC 2018	45
5.1 Assessment per TBEC 2018 Performance Evaluation Method	45
5.2 IDA of a Concentric X-braced Frame Designed to TBEC 2018	48
5.3 Collapse Assessment of Concentrically Braced Frames Designed to	

	TBEC 2018	50
VI	CONCLUSIONS	57
VII	APPENDIX A	60
VIII	APPENDIX B	80
IX	APPENDIX C	103
X	APPENDIX D	140
XI	BIBLIOGRAPHY	147
XII	VITA	151

LIST OF TABLES

1	Table- 1. A comparison between several seismic parameter definitions and design approaches of TBEC 2018 and TEC 2007	4
2	Table- 2. I section table used for the beams and columns	140
3	Table- 3. SHS section table used for the braces.....	141
4	Table- 4. Required plate thicknesses vs. plate thicknesses that are available in the market and used for the gusset plate connections	143
5	Table- 5. Gusset plate dimensions of the SDC D_{max} archetypes (<i>see Figure 21 for dimensions a, b and c</i>).....	144
6	Table- 6. Gusset plate dimensions of the SDC C_{max} archetypes (<i>see Figure 21 for dimensions a, b and c</i>)	145

LIST OF FIGURES

1	Figure 1. Concentrically braced frame configurations (a) x-braced, (b) inverted V (chevron) and (c) v-braced.....	2
2	Figure 2. Brace model proposed by Uriz et al. (2008), in which the bracing element is separated into 4 segments	8
3	Figure 3. Nonlinear rotational spring model retrieved from Hsiao et al. (2012)	10
4	Figure 4. Gusset plate geometry and $8t_p$ elliptical clearance model, retrieved from Roeder et al. (2011).....	13
5	Figure 5. Single-IDA and static pushover curves, retrieved from Vamvatsikos et al. (2002).....	16
6	Figure 6. Multi-IDA curve, retrieved from Vamvatsikos et al. (2002).....	17
7	Figure 7. Softening (a), hardening (b, c) and weaving (d) behaviors observed in IDA procedure, retrieved from Vamvatsikos et al. (2002)	18
8	Figure 8. CBF test specimen in the experiment conducted by Palmer et al. (2012).....	20
9	Figure 9. A flowchart on design procedure	23
10	Figure 10. Input parameters to begin the design procedure.....	25
11	Figure 11. ELF load distribution retrieved from TBEC 2018	26
12	Figure 12. Spreadsheet to calculate the equivalent lateral earthquake loads and distribute the total base shear among each story based on the ELF method of TBEC 2018	27
13	Figure 13. Spreadsheet for designing the braces in accordance with the story shear that is imposed in accordance with the ELF procedure (a-b).....	28

14	Figure 14. Spreadsheet for designing the beams with respect to the plastic capacity of the braces that are calculated by considering the expected material strengths (a-b)	30
15	Figure 15. Spreadsheet for sizing the columns by following a similar approach that is adopted for the beam design (a-b).....	31
16	Figure 16. Mass and stiffness matrices with period and story drift calculations	32
17	Figure 17. Eigenvector iterations	33
18	Figure 18. Spreadsheet developed to design G.P. connections using BDP (a-c)	34
19	Figure 19. A flowchart briefly illustrating the modeling process.....	38
20	Figure 20. An illustration of the nonlinear analytical model	43
21	Figure 21. An illustration regarding the gusset plate dimensions where a and b are the horizontal and vertical dimensions on beam and column faces, respectively, and c is the distance from the work point of the connection to the brace end	143



This page is intentionally left blank.

CHAPTER I

INTRODUCTION

Lateral load resisting systems are the most important aspect of the design of steel structures especially in high seismicity regions. Braced frames are common types among various lateral load resisting systems. The elastic behavior of those systems provides a significant amount of strength and stiffness which reduces the displacements developed within the system that might possibly affect the unity of non-structural elements, too. Efficiency of braced frames is not only limited with the elastic response, but also they can satisfy the energy demand and provide a safeguard against plastic deformations arising from severe seismic actions.

Major framing configurations of CBFs can be named as X-braced, multi-story X-braced, V-braced, inverted V-braced (chevron) etc., which are illustrated in Figure 1. One of the most common CBF configurations is the X-braced frame which is widely utilized in both industrial and residential multi-story buildings. Typically, braces of this system are placed diagonally between the work points of the beam-column joints, and intersect at the midspan. Primary load distribution of XBFs is based on truss action in which the story shear developed due to seismic actions is assumed to be shared between braces as axial loading. The energy demand of a severe earthquake event is satisfied by the buckling and yielding of the bracing members under axial compression and tension, respectively. However, this inelasticity must be accounted carefully to avoid a potential unintended failure mode. These large amounts of inelasticity must be tolerated by the

system. To overcome that, the seismic design codes suggest applying the capacity design principles which dictates that the members except the braces of a CBF system must be capacity designed. This approach is based on sizing the beam, column and connection elements to resist the maximum load that can be transferred from the braces to the other components of the frame which can also be described as the expected capacity of the braces.

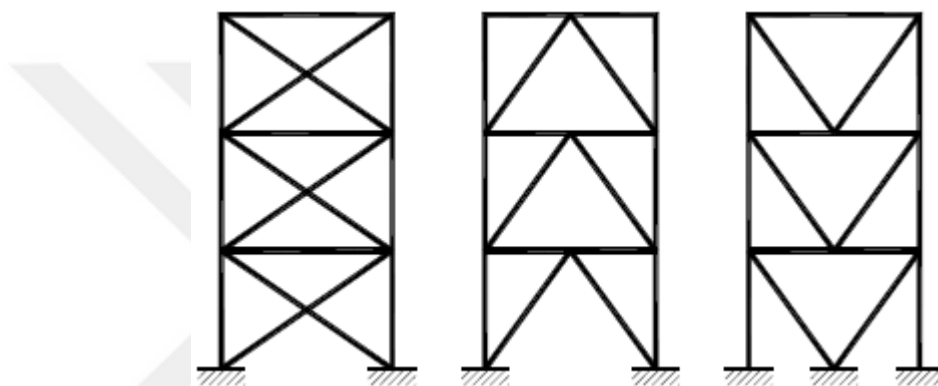


Figure 1. Concentrically braced frame configurations (a) x-braced, (b) inverted V (chevron) and (c) v-braced

There are numerous experimental studies that are mainly focused on the seismic behavior of concentrically braced frame systems [1–6] as well as the analytical studies [7,8]. Structural response of CBF systems under cyclic loadings are widely investigated to achieve a deeper understanding. Effects of stiffness degradation and ductility on the general behavior are challenging phenomenon to be predicted. Similarly, experimental programs are also available in the literature that provide a better understanding on the effect of the gusset plate connections on braced frame response [9–12]. These studies have also developed and suggested new design approaches and methods regarding the capacity design of braced frame connections, in order to enhance the seismic

performance of CBFs and result in more ductile designs while keeping the economical perspective in mind.

The Turkish Building Earthquake Code 2018 [13] (TBEC 2018) was published in January 2019, and it is currently in-use instead of the previous version, Turkish Earthquake Code 2007 [14] (TEC 2007). Various design principles are introduced by the publication of the new code such as yield mechanisms, usage of expected material strength, performance-based design, seismic hazard levels, ground motion intensities etc., which have similarities with several design specifications and guides of AISC and ASCE [15–17]. Also, several revisions were made in the definitions and applications of TEC 2007 such as definition of the design response spectrums, definition of the earthquake load reduction factor, calculation of design spectral accelerations etc. (please see Table- 1). For instance, a yield mechanism similar to the AISC is employed to account for the effect of post-buckling behavior of braces, in which the load transferred from braces to the other components are separated into 2 cases as (case 1) 1.14 times the expected compressive strength of braces and (case 2) 0.3 times the case 1. In addition to the yield mechanisms, response spectrum definitions are also revised in TBEC 2018. The previous definition was based on tabulated seismic zone parameters while the current code utilizes the mapped spectral coefficients depending on the ground motion intensity level and the local site class. To obtain the mapped spectral acceleration coefficients, a new version of Turkish Seismic Hazard Maps [18] was published. Here it should be noted that the local site class was only affecting the corner periods of the response spectrum in TEC 2007, however, it affects the amplitude of the spectrum's spectral intensity as well as the corner periods in TBEC 2018. Another revised definition is the earthquake load reduction factor, which was previously assumed to be

constant after the first corner period of TEC 2007 response spectrum while the beginning point of this region was shifted towards the second corner period of TBEC 2018 response spectrum. All aforementioned revisions and newly introduced requirements and design approaches are expected to influence the seismic design and performance of the CBFs designed to the new code, TBEC 2018.

Table- 1. A comparison between several seismic parameter definitions and design approaches of TBEC 2018 and TEC 2007

Case	TBEC 2018	TEC 2007
Response Spectrum Definition	Depends on design-basis earthquake (DBE) spectral response acceleration parameters at short periods, S_{DS} , and at 1-s periods, S_{D1} , adjusted for local site effects	Depends on peak ground acceleration coefficient (A_o) and require no adjustment for site class effects
Corner Periods of the Spectrum	Function of DBE spectral response acceleration parameters and local site class	Tabulated as constant values based on local site class
Earthquake Load Reduction Factor	Linear up to the second corner period followed by a constant region	Linear up to the first corner period followed by a constant region
Design Basis	Plastic analysis based on the yield mechanisms	Elastic analysis including the system overstrength

Case	TBEC 2018	TEC 2007
Ground Motion Intensity	4 levels are introduced to represent the earthquakes according to their probabilities of exceedance	Only DBE level intensity is given
Earthquake Hazard Maps	Interactive seismic maps are developed to produce the short and 1-second mapped spectral accelerations, S_s and S_1 , based on the location	Turkey is separated into 4 categories, and seismic hazard map is employed to specify a location's seismic risk category to obtain A_o

In this thesis, the major aim is to determine and evaluate the seismic performance of concentrically braced frames designed per TBEC 2018. As it was discussed above, to the author's knowledge, the literature is not fulfilled yet by such research that has a purpose on performance evaluation of TBEC 2018 CBF designs.

1.1 Overview

The overview chapter provides the outline of this thesis in order to make the document easier to follow. The beginning of the document, Chapter 1, is the introduction part which consists of the brief explanation regarding the background, motivation and the primary focus of this study. In the next chapter, a review on the literature and the design and modeling concepts of CBFs takes place. This part is also a brief description of the studies that are referred for the nonlinear model development,

selection of analysis methods and performance assessment technique used in this study. After the literature review, design approaches and the developed nonlinear numerical model are described in detail through chapter 3 to 4. In the 5th chapter, the seismic performance assessment of concentrically braced frames is provided in compliance with the previous publications of the author. The assessment is performed using three different approaches as (1) a performance evaluation approach given in TBEC 2018 and (2) a collapse assessment and (3) a probabilistic approach with a wider range of data. And the final part, chapter 6, concludes the thesis with a brief summary of the background, methodology and results of the assessments.

CHAPTER II

REVIEW OF THE RESEARCH AVAILABLE ON CONCENTRICALLY BRACED FRAMES IN THE LITERATURE

Braced frames are widely preferred lateral load resisting systems. So that there are various experimental and analytical studies available in the literature. A number of these studies aim to enhance the seismic design and detailing while some focus on improving the accuracy of the analytical models by introducing several modeling techniques. This chapter represents the brief summaries of those studies and discusses the background and application of them.

2.1 Brace Buckling

As it is discussed previously, braces are the main components of concentrically braced steel frames. In brief, they are employed to theoretically resist the whole lateral load acting on the frame by truss actions. This load distribution yields the braces to axially deform under compressive and tensile forces during cyclic loading such as seismic actions. Also, inelastic yielding and buckling of braces satisfy the energy demand of a major earthquake event and braces are called ‘fuse elements’ of the system due to allowance of plastic deformations. Accordingly, all members except braces of CBF systems must be capacity designed based on the capacity of bracing members to avoid a potential undesired mechanism formation.

The main challenge of brace modeling is simulating the buckling initiation and post buckling behavior of braces. It is not accurate to model the braces as single line (frame) elements since the buckling behavior cannot be caught by such an approach. To overcome that, Uriz et al. [8] conducted a study on cyclic buckling of the braces in which a modeling approach is proposed. This approach is based on the buckled shape of compression members and suggests introducing an initial camber to the element. Recommendations of this study are (1) dividing the brace elements into 2 subparts and (2) assigning initial imperfections of 0.1% of the brace length. The first one is to provide the distribution of plasticity along the brace, while the second one is based on considering the small deflections in the analysis to capture the buckling initiation. Also, the initial camber value is limited to a range of 0.05% to 0.1% to avoid an unintended stiffness reduction in the system during elastic behavior. In general the purpose of this approach is to simulate the braces such that the buckling initiation and post buckling behavior is captured while preserving the elastic stiffness to a large extent. The proposed brace model in this study is illustrated in Figure 2.

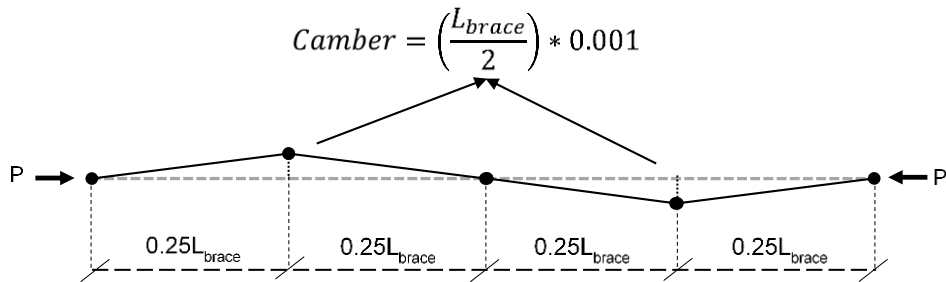


Figure 2. Brace model proposed by Uriz et al. (2008), in which the bracing element is separated into 4 segments

Major improvements to the brace modeling approach proposed by Uriz et al. were introduced by Hsiao et al. [7] with a set of both experimental and computational studies. Effect of the gusset plates on the global system was included in this technique,

as well as suggestions on brace element subdivision approach. Connections are assumed to be pinned in the model proposed by Uriz et al., which yields to neglect the restraining effect of gusset plates of brace-to-beam-column joints since the gusset plate connections are neither fully fixed nor pinned actually and provide remarkable stiffness to the global system. Pursuant to that, Hsiao et al. recommended to place nonlinear rotational springs to brace ends which extend from the working point of the joint as modeled as rigid end zones. Rigid offsets enable to simulate the in-plane stiffness of gusset plates. Rotational stiffness of the springs was derived as given in Eq. 1, as a function of geometrical and material properties of the gusset plate.

$$K_{gusset}^{rotational} = \frac{E}{L_{avg}} \left(\frac{W_w t_p^3}{12} \right) \quad (1)$$

Here, E is the modulus of elasticity, L_{avg} , W_w and t_p are average buckling length, Withmore width and thickness of the gusset plate, respectively. Post yielding stiffness of the spring model is suggested as 1% of the initial stiffness. Rigid end zones are proportional with the gusset plate dimensions and assigned as the plate height from the beam face and 75% of the plate width from the column face to allow a limited deformation within the connection, as experiments showed a small amount of in-plane deformations can be expected in gusset plate connections. Brace end rigid offsets provide accurately modeling the effective buckling length of braces compared to the length between work points of brace-to-beam-column joints, which directly affects the capacity of the member. On the other hand, the previously mentioned study suggested to divide braces into 2 sub elements while Hsiao et al. revealed that modeling the braces as 10 or more segments to form a sine curve shape with an apex of similar initial imperfection provides more accurate results in terms of buckling behavior. Nonlinear

force-based beam-column elements consisting of fiber sections are suggested to be assigned to all elements of CBF systems to simulate distributed plasticity. A detailed illustration regarding the spring model proposed by Hsiao et al. is provided in Figure 3.

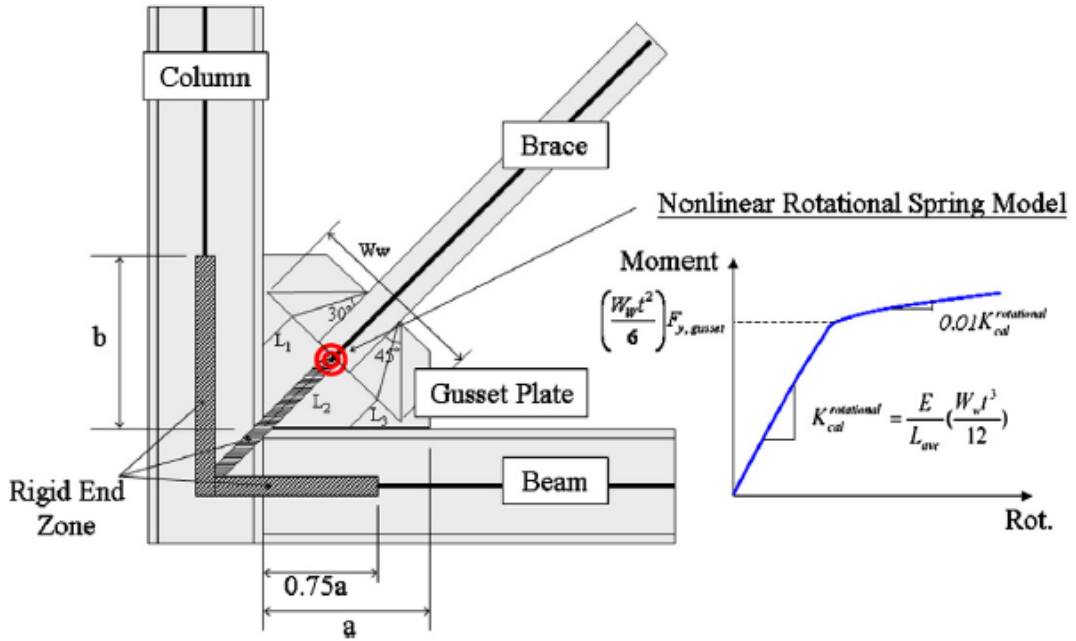


Figure 3. Nonlinear rotational spring model retrieved from Hsiao et al. (2012)

2.2 Balanced Design Procedure (BDP)

Connections of CBFs must be designed carefully to provide required strength and stiffness while ensuring rotation demands. It is important to avoid a sudden failure within the connection under large deformations. Capacity design principles require designing the corner connections of CBFs to be stronger than braces. There are numerous design approaches that are currently in-use to size the gusset plate connections. For instance, the $2t_p$ linear clearance is still a popular approach for sizing the gusset plates to allow the required out-of-plane rotations that are expected to develop at the brace ends due to the buckling behavior of the member. However, this clearance model usually requires thick and large gusset plates according to the [19].

Lehman et al. [11] proposed an alternative method, namely $8t_p$ elliptical clearance model, for the linear model, which aims to result in more economical connection design and enhance the efficiency of the connection's seismic response. As a result of the same program, a balanced design procedure (BDP) concept was also proposed for ensuring a better performance while avoiding a development of premature failure mode within the connection. This program provided a wide range of experimental data regarding several components of braced frame systems.

The Balanced Design Procedure, BDP, is proposed by Roeder et al. [19] and aims to provide the required strength and deformation capacity. BDP employs balance factors, β , to amplify each failure (yield) mode that should be considered for design. This amplification is based on the priority of failure modes. Design of the connection is based on following the expression below (Eq. 2), in which β is the balance factor, P_u is the critical of the expected compressive and tensile plastic strength of the braces, R_{fail} is the nominal resistance of the yield mechanism:

$$P_u \leq \beta_{yield,1} R_y R_{fail,1} \leq \beta_{yield,2} R_y R_{fail,2} \leq \beta_{yield,i} R_y R_{fail,i} \quad (2)$$

The limit states, or namely secondary yield mechanisms, that are considered for the connection design using the BDP approach are as following:

- Whitmore yielding ($\beta=1.00$),
- Brace net section fracture ($\beta=0.95$),
- Gusset plate buckling ($\beta=0.90$),
- Block shear ($\beta=0.85$),
- Whitmore fracture ($\beta=0.85$),

- Brace-to-gusset weld ($\beta=0.75$),
- Brace-to-gusset base metal ($\beta=0.75$) and
- Interface welds ($\beta=0.75$).

Undesired failures are highlighted as critical with a lower β factor to prevent the related modes to occur while the selected yield mechanisms are allowed in a controlled manner. For instance, the most critical limit states were assumed to be brace-to-gusset weld, brace-to-gusset base metal and interface welds. All those mentioned states can be expected to develop sudden and brittle failures and accordingly their β factor was employed as 0.75. On the other hand, Whitmore yielding limit state was accepted as the most desired failure mode, with a β factor of 1.0, among all considered limit states. This means Whitmore yielding is the primary yield mechanism which is intended to develop first in the connection. Also since the connection is not expected to fail during Whitmore yielding state, the desired rotations to be formed within the connection is allowed by this way. Conventional CBF connection designs per current design codes such as the Turkish Specification for the Design, Calculation and Construction of Steel Structures 2016 [20] (SDCCSS 2016) or some AISC guides do not ensure the connection to accommodate the required level of rotations under large story drifts demanded by cyclic loads because of the excessive strength of the connections.

The experimental program showed failures of the system occurred due to interface weld fractures which is an undesired brittle mode and may cause a sudden failure. Major goals of BDP are to ensure the connection is sufficiently ductile and strong and avoids brittle failures. Similarly, the $2t_p$ linear clearance approach of design codes usually leads to thicker gusset plates which reduces the rotational capacity of the connection. To that end, the authors performed several tests on CBF connections to be

able to provide a comparative study on the effect of different clearance models on cyclic behavior of CBFs. According to the results, they recommended a new model based on $8t_p$ elliptical clearance which was deemed to be an effective approach that results in more compact plates while satisfying the strength and rotation demands. This clearance model is given in Figure 4.

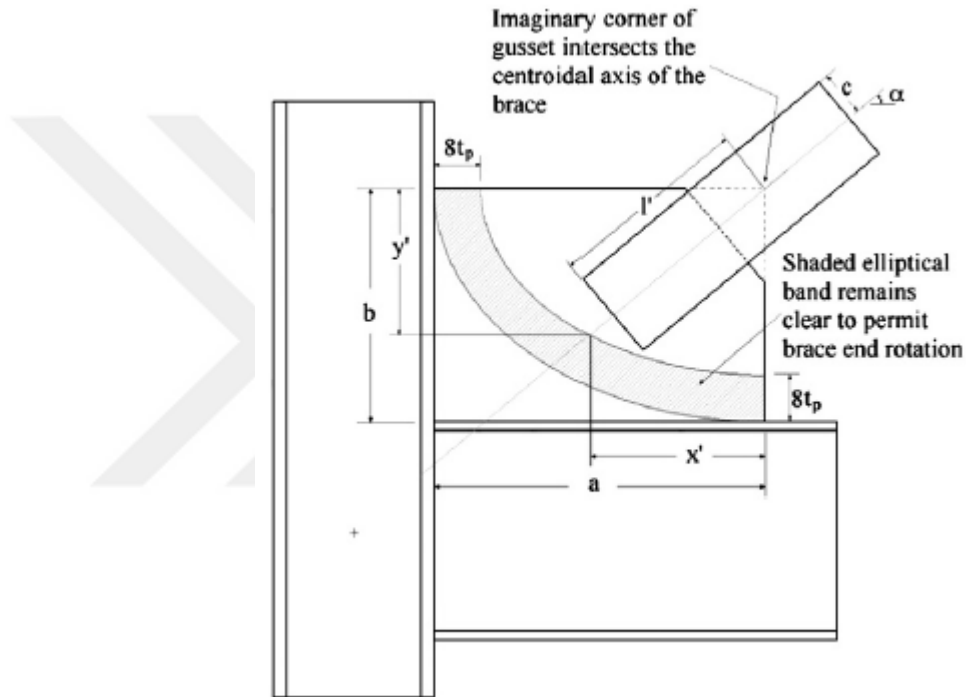


Figure 4. Gusset plate geometry and $8t_p$ elliptical clearance model, retrieved from Roeder et al. (2011)

Conventional gusset plate designs are not capable of accommodating story drifts as large as the gusset plates designed using BDP are capable of, according to the tests conducted by the authors. BDP provides 20% more deformation capability compared to the other design philosophies, which means the system becomes more ductile with BDP connection design while maintaining the strength and avoiding undesired brittle failure

modes. Please refer to the study to find a comprehensive table and illustration regarding the comparison of the results of various design approaches.

2.3 Brace Fracture

Brace fracture is another challenging behavior to simulate in computational models and analyses. Buckling initiated in bracing elements will eventually result in a fracture within the cross section of the brace due to the local strains developed under cyclic loading such as earthquake or wind loadings. As it is difficult to accurately simulate the buckling and post buckling behavior of compression members, it is also hard to estimate the strain at which a brace will face fractures. However, approximated approaches were developed to overcome this challenge by several studies such as Hsiao et al. [21] and Lignos et al. and Karamanci et al. [22,23]. Tests conducted by Lignos et al. and Karamanci et al. showed that it is possible to predict the strain accumulated within the brace at fracture state. According to the experimental results, they have suggested power-based equations for different structural steel sections, which enable to calculate the strain at fracture, ε_0 , approximately. Those expressions are defined as a function of geometrical and material properties of the section used as a bracing member. The equation suggested for HSS profiles is given in (Eq. 3), in which (1) kL is the effective length and r is the radius of gyration, (2) E is elasticity modulus and F_y is yield strength and (3) w is the width and t is the wall thickness of the member.

$$\varepsilon_0 = 0.291 \left(\frac{kL}{r} \right)^{-0.484} \left(\frac{w}{t} \right)^{-0.613} \left(\frac{E}{F_y} \right)^{0.3} \quad (3)$$

Using the predictive equations for calculating the maximum strain at fracture, a material model can be improved to capture the damage accumulated within a cross section and the fracture itself. For instance, there is a material model named ‘fatigue material’ available on OpenSees [24] library, which enables the inclusion of the maximum strain that can be accumulated within a section into the model by wrapping onto the base material.

2.4 Incremental Dynamic Analysis (IDA)

Structural analysis methods have been continuously shifting from simplified static analysis towards more complex nonlinear dynamic analysis methods. Enhancement of computer processing ability provided to perform more sophisticated and faster analyses. One of the most common nonlinear dynamic analysis methods is named as Incremental Dynamic Analysis method, which was mentioned in the late 20th century by several studies such as [25–27], and adopted by FEMA guidelines that are published at that time (e.g. [28]). Vamvatsikos and Cornell [29] unified the studies regarding the IDA concept to develop a methodology and terminology for the IDA method. IDA is briefly based on increasing the scale factor of a single time history analysis under a ground motion record, by a specific increment at each step, until a collapse state has been observed. IDA is not limited to a single time history analysis but also can be employed with a set of GM records, which are named single IDA and multi-IDA, respectively. The primary result of this method is the IDA curve, in which the vertical axis commonly consists of the spectral intensity while the horizontal axis includes a damage or intensity measure. Intensity measure, IM, is defined as the scalable parameter that indicates the level of the related IDA step depending on the

monotonically increasing scale factor. Various quantities might be employed as an IM parameter such as peak ground velocity, spectral acceleration etc., which are being typically used. On the other hand, damage measure is defined as the level of structural response to the applied seismic loading. Common parameters that are utilized to quantify the damage measure are base shear, interstory drift, rood drift etc. Accordingly, IDA curves are produced as a function of both selected damage and intensity measures as a result of the IDA method. The most common types of DMs and IMs are maximum interstory drift ratio (ISDR) and spectral acceleration, respectively, and IDA curves consist of those parameters in general.

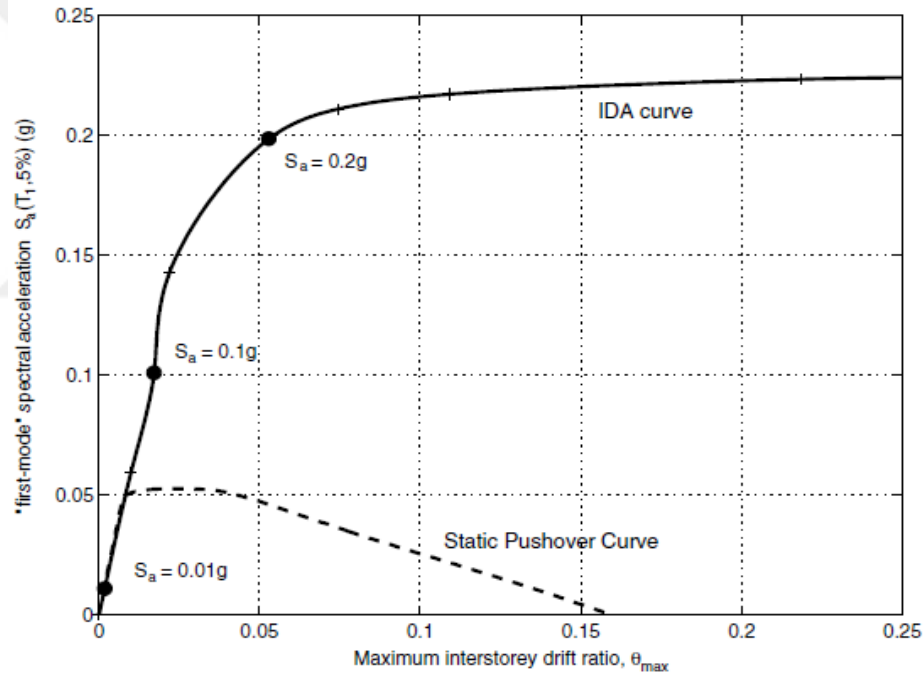


Figure 5. Single-IDA and static pushover curves, retrieved from Vamvatsikos et al. (2002)

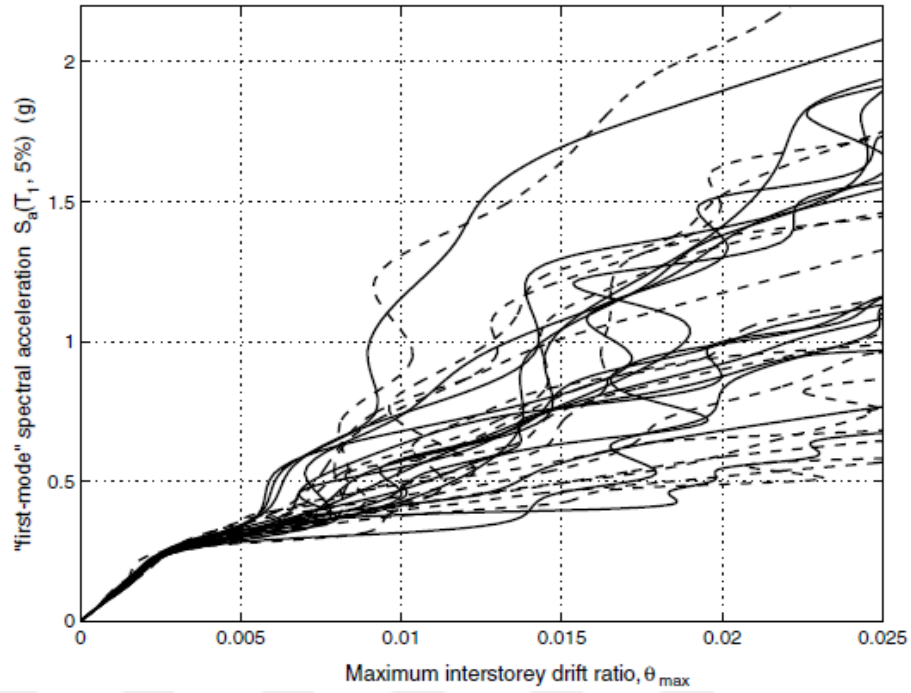


Figure 6. Multi-IDA curve, retrieved from Vamvatsikos et al. (2002)

Examples of single- and multi-IDA curves are illustrated in Figure 5 and Figure 6, respectively. The curves show how the structure responds to the demands imposed by the applied ground motion records depending on the level of intensity. Typically, there is an elastic region upon a specific point similar to the aforementioned analysis methods, and following the elastic region, slope of the IDA curve starts to diverge from the elastic tangent and the structural system begins to soften. However, hardening and weaving (Figure 7) can be observed in an IDA curve, which means unexpected changes in the slope of the curve, even the slope of an individual IDA step, might be greater than the elastic behavior.

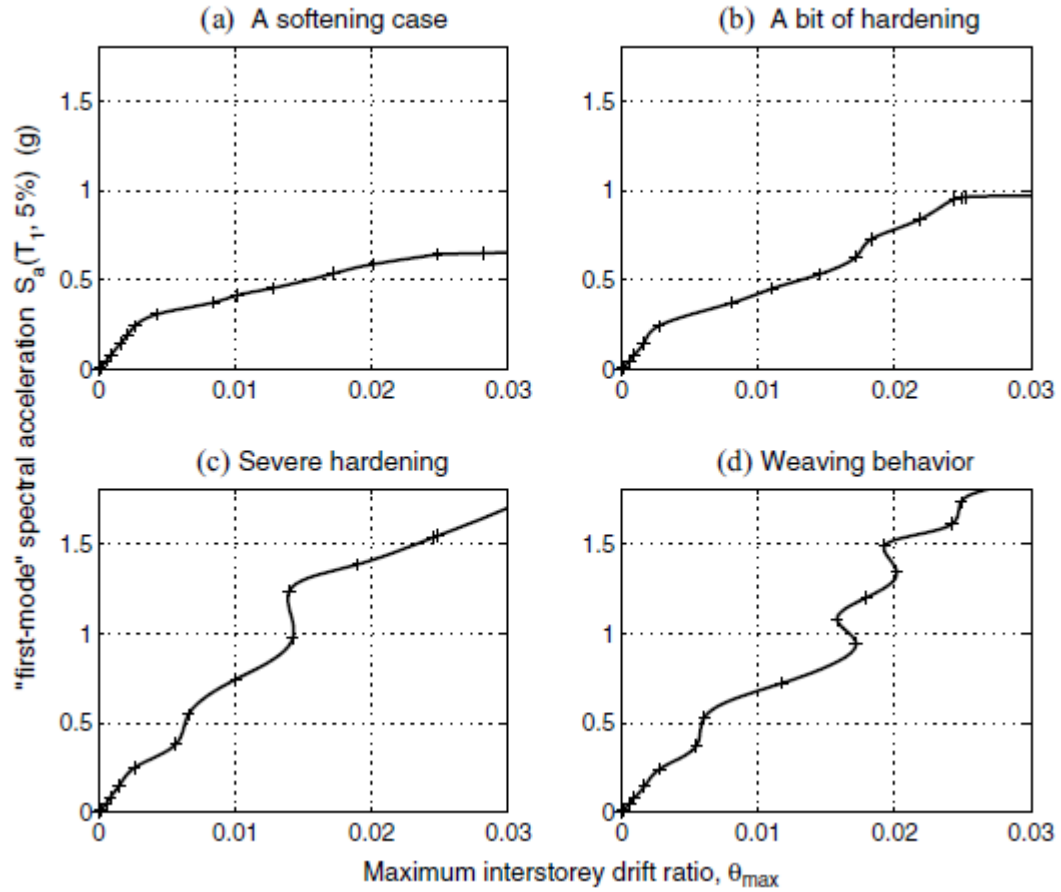


Figure 7. Softening (a), hardening (b, c) and weaving (d) behaviors observed in IDA procedure, retrieved from Vamvatsikos et al. (2002)

2.5 Overview on FEMA P-695 Quantification Methodology

FEMA P-695 [30] is a guideline that is published by the U.S. Federal Emergency Management Agency aiming to quantify the seismic performance of lateral load resisting systems and to verify the usage of seismic design parameters such as response modification coefficients, deflection amplification factors or system overstrength factors etc. The process starts with archetype selection and design. It is followed by nonlinear model development. The nonlinear model is then used to perform nonlinear static analysis. Base shear, ultimate roof drift and effective roof yield drift are

observed from the pushover curves, to be used later on the calculations of system overstrength and period-based ductility. These parameters are employed to obtain system uncertainties, which are one of the primary input variables to define the collapse probabilities, namely collapse margins. After the pushover analyses are completed, a set of nonlinear dynamic analyses are performed based on the IDA method to obtain the spectral acceleration at 50% of the GM records causing collapse of the system, which is called median collapse capacity. This parameter is used to calculate the collapse margin ratio (CMR) that is defined as the ratio of median collapse capacity to the spectral acceleration of maximum considered earthquake. CMR is transformed into adjusted collapse margin ratio (ACMR) by amplifying with the spectral shape factor, which has a purpose of accounting for the variety in the frequency content among the GM record set. ACMR values are compared with the target ACMRs which are tabulated depending on system uncertainties, in order to verify the archetypes have passed or failed the performance evaluation. If the performance is deemed to be unsatisfactory, the procedure is reiterated until an ACMR larger than the target value is achieved.

2.6 Experimental Studies on Concentrically Braced Frame Systems

Besides the analytical studies, there are a considerable number of experimental studies on seismic behavior and performance of both global systems and individual members of CBFs, in the literature. To name some of them among many others, Tremblay et al. (2003) [1], Fell et al. (2009) [4], Palmer et al. (2012) [31] and Palmer et al. (2013) [2] investigate hysteretic behavior of CBFs under cyclic loads and individual behavior of braces under earthquake loads. For instance, in Palmer et al. (2013) two types of braced frames were tested under cyclic loading to observe the damage

accumulation in both the system and individual elements. Similarly in Palmer et al. (2012) an experiment was conducted to investigate the effect of brace splice connections and the out-of-plane behavior of braces. The single-story single bay concentrically braced frame test of this study, which is shown in Figure 8, was adopted to verify the models used in this research.

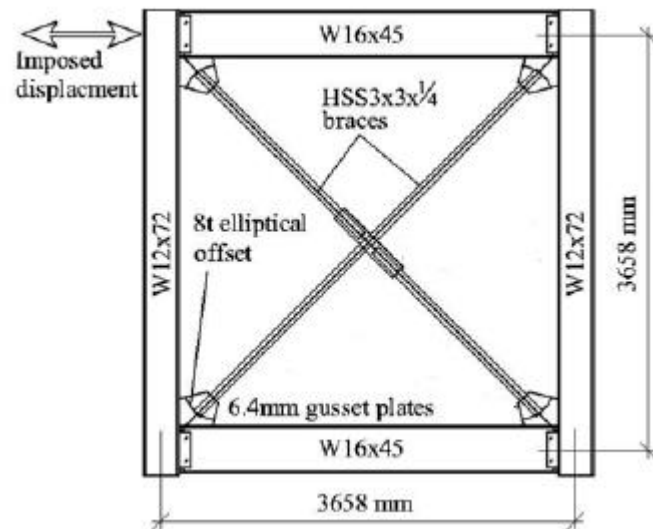


Figure 8. CBF test specimen in the experiment conducted by Palmer et al. (2012)

All these works provide a better understanding on performance of CBF systems and inelastic behavior of braces, which is a sophisticated behavior to predict, and also, they are effective tools to validate the analytical models to be used in CBF modeling.

CHAPTER III

DESIGN PROCEDURE

All CBF design procedures typically start with sizing the bracing member first, just after the lateral load distribution among the system is determined. Similarly, concentrically X-braced frames are commonly designed by adopting the capacity design method to ensure the structure is adequately ductile while being stiff enough to resist the lateral earthquake loads. Since these loads are assumed to be resisted by the bracing members as compressive and tensile load components, braces are the beginning elements per the capacity design principles. After the braces are sized with respect to the distribution of the lateral load, their expected plastic capacities for tension (expected yield stress(T)) and 2 cases of compression ((1)expected buckling strength($C1$) and (2)post-buckling strength($C2$)) are calculated considering the expected material strengths. Then the earthquake load that is transferred to the other members of the system is set to be all 3 components of expected plastic capacities of the braces. Depending on the considered earthquake direction, $C1$, $C2$ and T loads are acted to the columns, beams and connections. In addition to the earthquake loads, both seismic and structural steel design codes employ well known load combinations that include typical loading cases such as dead and live loads, by amplifying each case with specific coefficients depending on the design approach, ASD or LRFD. In this study, the LRFD approach was used for CBF member designs. Afterwards, the load combination that results in the most critical case was selected for each member to determine the member size. Please note that the primary objective for member selection was minimizing the

structural weight so that the lightest section that is available within the section table (Table- 2) was employed. The section table was created to represent the commonly used sections in both Turkish and European markets which consisted of HEA, HEB, HEM, IPE, IPN and SHS profiles. Section catalogues of several local manufacturers were also investigated to validate the availability of the tabulated sections. S355 grade structural steel was selected for all members of the CBF systems including the gusset plates of connections, with a yield strength of 355 MPa and a modulus of elasticity of 200 GPa. Figure 9 provides a flowchart that represents the design steps and summarizes the design procedure in general.

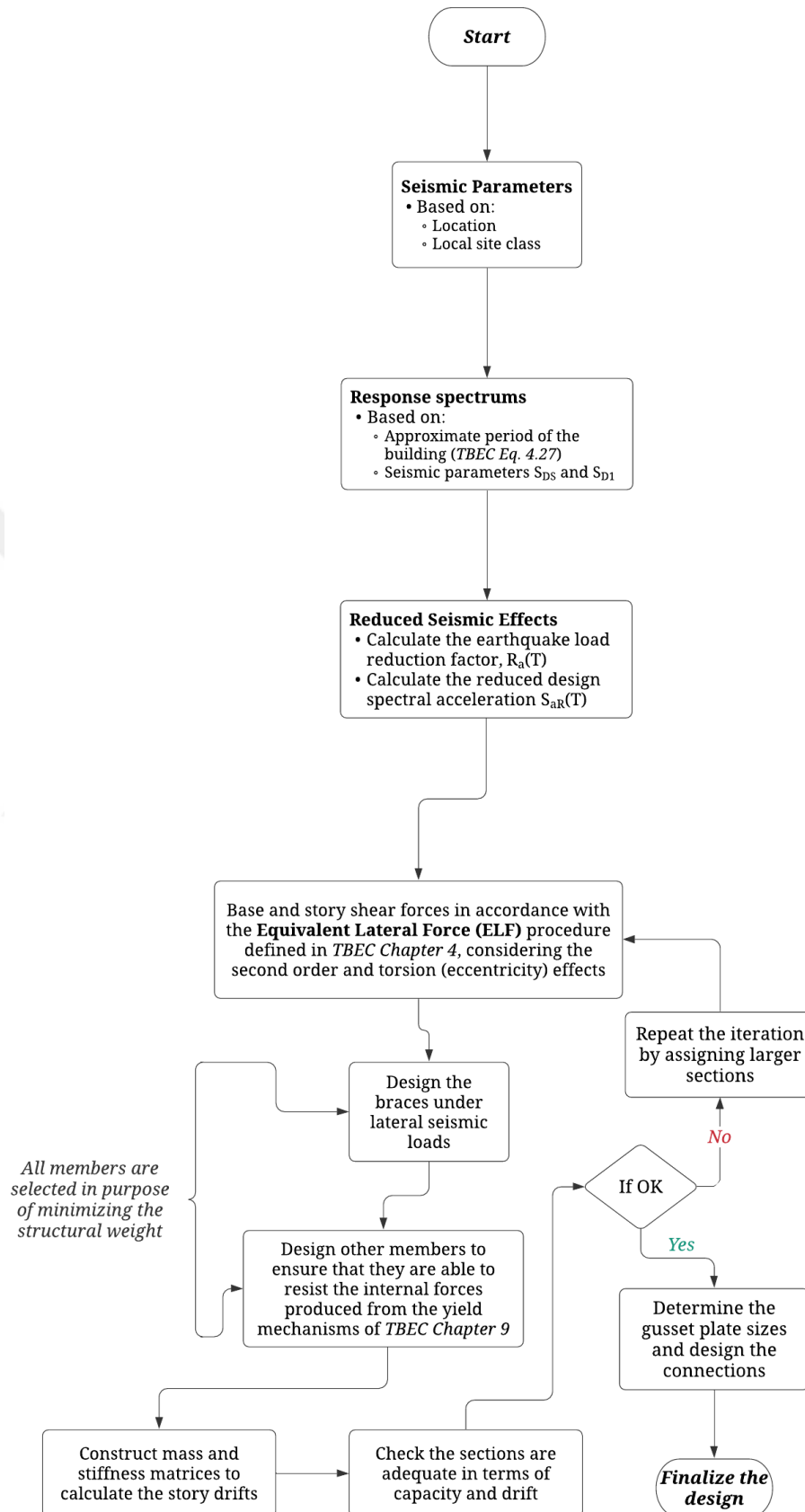


Figure 9. A flowchart on design procedure

3.1 Design Excel

Structural design and member sizing of all archetypes were performed on a MS Excel spreadsheet which was developed for the purposes of this study. The procedure will be described in detail herein, within this chapter. The first sheet includes the input cells that are required to define the building. The data regarding the building's geometrical and material as well as the loading and design parameters are provided here. As it can be seen from the Figure 10, (1) yield and ultimate strength with elasticity modulus and strain hardening of the members, (2) number of bays, span lengths, number of braced bays and the type of section to be used as brace (square or tube), (3) number and height of each stories, (4) mapped spectral coefficients for several ground motion intensity levels and site class, (5) design loading values and, (6) response modification coefficient, overstrength, building importance and live load mass participation factors are required as input parameters. Number of stories is limited to 20. After these cells are filled, the design button is clicked to start the macro code that performs the section selection for each member.

shear formulation m_t is the total seismic weight, I is the building importance factor and g is the gravitational constant, and in the second formulation; $F_{iE}^{(X)}$ is the load to be acted on i^{th} story towards X direction, $\Delta F_{NE}^{(X)}$ is the additional equivalent lateral force to be acted at the top (roof level) of the building towards X direction, m_i is the weight and H_i is the height of the i^{th} story.

$$V_{tE}^{(X)} = m_t S_{aR}(T_p^{(X)}) \geq 0.04 m_t I S_{DS} g \quad (4)$$

$$F_{iE}^{(X)} = (V_{tE}^{(X)} - \Delta F_{NE}^{(X)}) \frac{m_i H_i}{\sum_{j=1}^N m_j H_j} \quad ; \text{ where} \quad (5)$$

$$\Delta F_{NE}^{(X)} = 0.0075 N V_{tE}^{(X)} \quad (6)$$

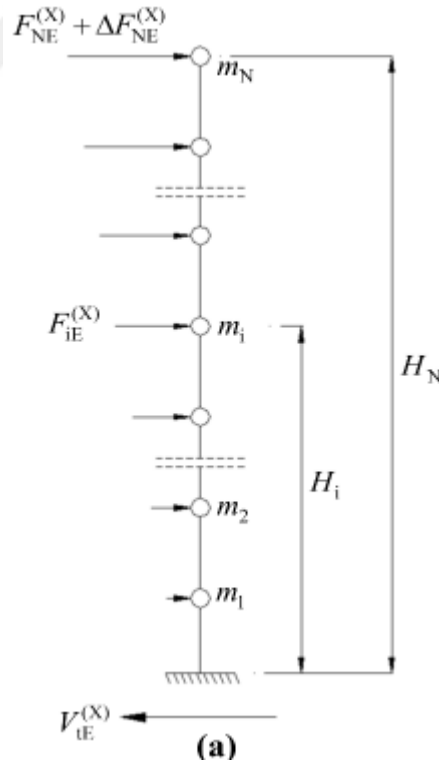


Figure 11. ELF load distribution retrieved from TBEC 2018

The spreadsheet developed to calculate the seismic loads to be imposed to the system based on ELF procedure is given in Figure 12.

Nstory	2	#	LLMPF (n)	0.3
Area	900	m ²	Ta	0.344
g	9.80665	m/s ²	Mtotal	1027.9
Ct	0.08		Htotal	7
I	1		SaR (T)	0.2687

TBDY 2019
Ta: approximate period

W (kN)	Dead	Live
Mstory	4500	540
Mroof	4500	540
Gtotal	10080.0	

Vte (kN)	ΔFn (kN)	m ² *Hj
2708.7	40.6	5396.3

If theta does not satisfy the TBEC limits, the second order effects defined in TSSS are utilized

Story	ith	Hi (m)	Ht (m)	M (ton)	Fi (kN)	$\frac{m}{\Delta}$	V (kN)	w_{tot}	θ_{li}	TBEC Ch.4 β	FEMA Φ_{Lx}	m_x	C_o
1	1	3.5	3.5	513.937	889.3	0.006693	2708.7	1027.9	0.007116	0.90	1.00	0.53377	513.937
2	2	3.5	7	513.9	1819.3	0.006023	1819.3	513.9	0.004767	0.89	1.00	1	513.937
3	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
4	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
5	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
6	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
7	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
8	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
9	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
10	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
11	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
12	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
13	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
14	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
15	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
16	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
17	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
18	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
19	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000
20	0	0	7	0.0	0.0	0	0.0	0.0	0	0.00	0.00	1	0.000

Figure 12. Spreadsheet to calculate the equivalent lateral earthquake loads and distribute the total base shear among each story based on the ELF method of TBEC 2018

Braces are sized with respect to the axial loads calculated as a result of the lateral loads distributed between each story. Then the expected capacities of selected sections are obtained in order to find the maximum force that can be developed in the braces and transferred to other components of the frame. Two coefficients, R_y and R_t are utilized in TBEC 2018 to transform the characteristic material strengths into expected material strengths. For S355 grade SHS sections these are defined as 1.4 and 1.3, respectively. Using those values, expected material strength of selected brace sections are determined for each story. All three cases defined in the yield mechanisms of TBEC 2018 are considered in the calculations of maximum load that can be developed in braces. Please note all I, box and tube sections and plates used for all members are tabulated considering the commonly used sections in practice and highly available in

[illegible]

F_y= 355
F_u= 510

[illegible]

28

After the braces are sized, beams and columns of the frame are designed under the effect of the load combinations that consist of vertical dead and live and horizontal earthquake loads. Due to the lateral loads, beams are designed under combined effect of flexure and axial load. Besides the capacity check according to the P-M ratio of the sections, the compactness criteria of TBEC 2018 is also checked and all sections are selected among the compact sections that satisfy the criterion. The selected sections as a result of this procedure indicate the final design of the building. After that, mass and stiffness matrices are formed to calculate the drifts resulting from reduced lateral loads. Drifts are amplified with the R factor used in the design to transform the elastic drifts (reduced) into inelastic drifts (amplified) according to the superposition approach suggested in TBEC 2018, and are checked with respect to the maximum allowable drift values calculated per TBEC 2018. The spreadsheets that are created to design the beams and columns of the CBF, and the mass and stiffness matrices constructed for the drift calculations and eigenvalue analysis are provided in Figure 14, Figure 15, Figure 16 and Figure 17.

History	2	Lns (m)	5.00	DL (kPa)	5.00	DLr (kPa)	0.00	SL (kPa)	2.00
Ry	1.25	Lew (m)	5.00	LL (kPa)	2.00	LLr (kPa)	2.00		
Fy	355								
0.6*5ds/3	0.2								

NS		Rd	m	kN															
Story		Angle	Length	T	C	Cpb	Tension Side		Compression Side		Tributary (kN)		1.2D+1.6L	D+L+0.25+E1	0.9D+E1	D+L+0.25+E2	0.9D+E2		
							E1	E2	E1	E2	DL	LL	P	P	T	P	T	P	T
1	0.611	3.5	955.88	462.92	138.88	442.43	516.44	1117.06	909.40	125	30	200.8	1315.1	-316.8	1221.3	-359.9	1104.6	-391.8	1010.8
2	0.611	3.5	713.50	278.55	83.56	159.74	47.92	409.17	409.17	62.5	5	83.5	497.2	-100.4	455.7	-116.7	497.2	12.2	455.7
3	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

(a)

(a)

K= 1
Cm= 1

Section	KL/r	Fe	Fcr	Pn	Ca	λ_r	S.rat.	Check	TBEC		TSSS		β	λ_r	λ	P/C Ratio	TSSS			Type	TypeCheck
									B_{II}	B2	2nd order										
											Pel	B1 Epb									
13.0	76.59	336.53	228.29	1489.56	0.57	43.17	14.35	1	1.00	1.01	1.00	7.121	6.429	0.883	77571.1	1.01	2	1			
13.0	76.59	336.53	228.29	1489.56	0.21	49.63	14.35	1	1.00	1.00	1.00	7.121	6.429	0.334	77571.1	1.01	2	1			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!	0.0	0.00	10	---			
---	0.00	0.00	0.00	0.00	#N/A	#N/A	#N/A	---	0.00	1.00	0.00	0	#N/A	#DIV/0!							

(b)

Figure 15. Spreadsheet for sizing the columns by following a similar approach that is adopted for the beam design (a-b)

Elastic	maxDrift	maxDisp
	0.0019	0.0067
Inelastic	maxDrift	maxDisp
	0.0096	0.0335

[illegible][illegible]

Figure 16. Mass and stiffness matrices with period and story drift calculations

Figure 17. Eigenvector iterations

33

When the whole design process is completed, the intended data is recorded to be used for analyses and assessment purposes. A flowchart that briefly represents the design procedure is provided at the beginning of this chapter, in Figure 9.



CHAPTER IV

NONLINEAR ANALYTICAL MODEL

The nonlinear analytical model developed for this study will be explained throughout this chapter. Modeling approaches, assumptions and methods used during the model development process will be presented. Detailed information and discussion on these methods with applicability and background is provided in the previous chapters.

CBF behavior is a challenging case to simulate in the computational models. Buckling initiation, post buckling stiffness, in-plane rigidity and out-of-plane flexibility of the connections are sophisticated structural responses to be captured. Over-simplified models result in inaccurate simulations especially for the inelastic behavior since the nonlinearity cannot be sufficiently accounted for in the model. However, there are various studies regarding the CBF behavior as well as the individual element response, as prescribed in Chapter 2. The nonlinear numerical model used for this research was constructed by adopting the modeling approaches proposed by those studies.

4.1 Model Development on OpenSees

Since a wide range of CBF archetypes were selected and designed for the purposes of this research, an analysis software that is able to enable the user to develop a parametric analysis file was used. OpenSees [24] provides this to its users, and it is

able to handle complex models including the nonlinear material definitions, geometrical nonlinearities, several types of analyses etc. The model was developed on this program. OpenSees is a code-based program which utilizes the Tool Command Language (Tcl) as the fundamental coding language. Accordingly, it is possible to link it with another software such as Matlab. The main code that defines the materials, sections, nodes, elements and analysis parameters can be called as a source file from Matlab. As it is prescribed, the Design excel records the design data for each archetype. The input parameters to define the archetypes are retrieved from this data regarding the following structural properties and design;

- the number of stories,
- total floor area of each story,
- story heights,
- dimensions, material and geometrical properties of the sections that are assigned to the CBF components as well as the connections.

A brief summary regarding the modeling procedure is illustrated in the flowchart given in Figure 19.

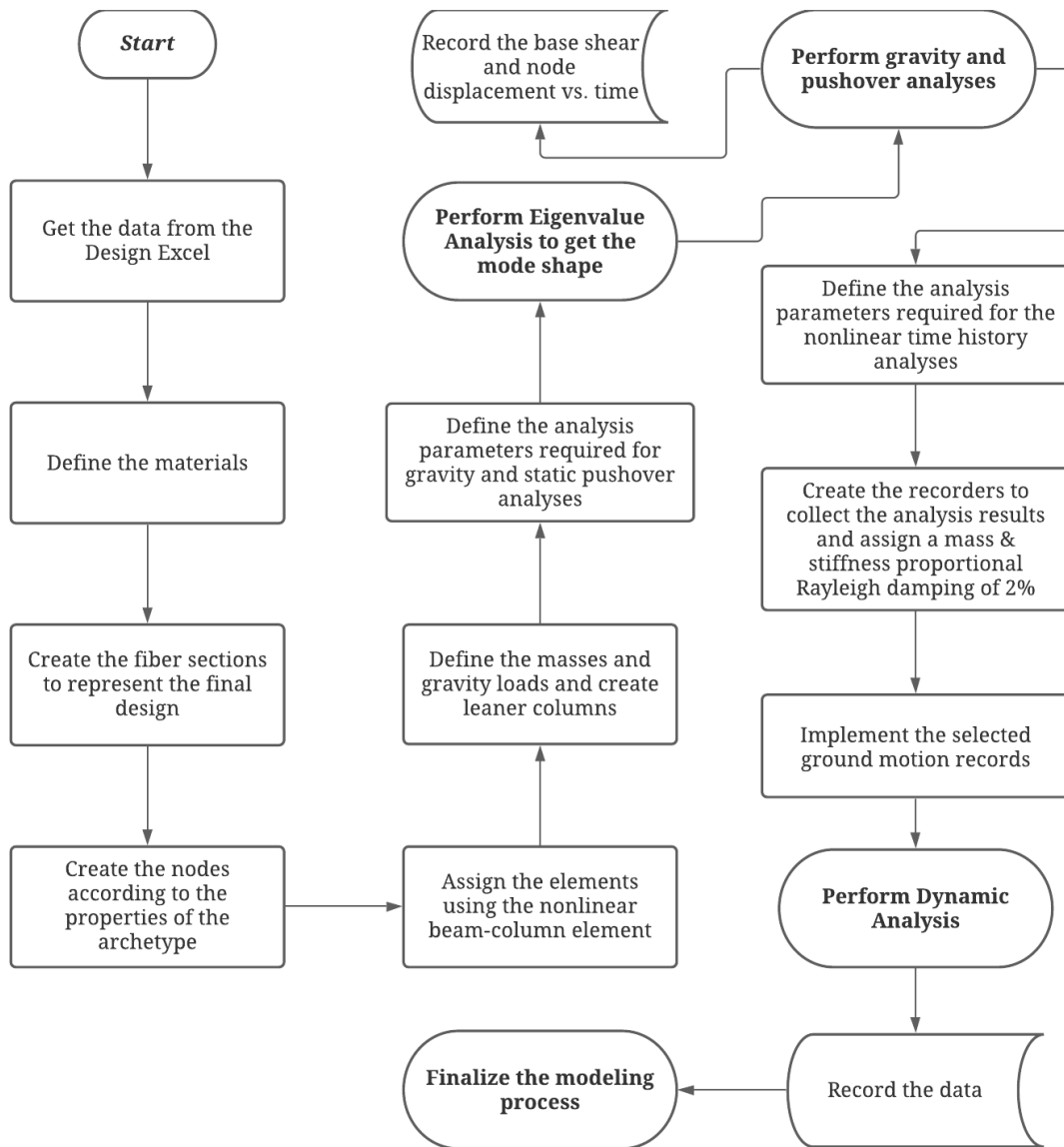


Figure 19. A flowchart briefly illustrating the modeling process

Modeling procedure starts with the definitions of number of dimensions and degree of freedoms and material and sections. All models were created and analyzed assuming a 2 dimensional-3 degrees of freedom space. A commonly used material model, Steel 02 - Giuffré-Menegotto-Pinto Model with isotropic strain hardening, which is available on the OpenSees Library, was adopted for all elements, to simulate the S355 grade structural steel selected for design. This material model contains an

accurate representation of both monotonic and hysteretic material behavior. However, fracture behavior is not included in Steel 02, which might significantly affect the general stability of the system due to a sudden development of a fracture within the cross section of a brace. To account for this phenomenon, another material model should be implemented, namely the fatigue material, which is also available on OpenSees Library. This approach was proposed by Karamanci and Lignos [22], with the predictive equations as prescribed in Chapter 2. According to this study, the strain at fracture is named as ϵ_0 and calculated using the equation derived for the HSS bracing members (Eq. 3). The fatigue material is employed to improve the model to capture the fractures of the braces by assigning the corresponding input parameters depending on the section's geometrical and material properties. On the other hand, the same material, Steel 02, is also separately defined for the gusset plates. The nonlinear rotational spring model proposed by Hsiao et al. [7] is defined at the brace ends with rigid end offsets at both ends, using this gusset plate material; however, the input parameters of this material model were assumed different from the one defined for the framing components. Hsiao et al. [7] proposed two separate expressions to calculate the strength and stiffness to be used to define the material of the nonlinear rotational spring model. The background of these approaches is also described previously.

At the next step, sections of the framing components are modeled with the fiber sections to simulate the material nonlinearities within the cross section. In other words, the distributed plasticity approach is adopted. I and SHS sections were defined to be used for the beam-column and bracing elements. To create the frame, nodes are defined in accordance with the data retrieved from the Design Excel. After that, all elements are defined between the related nodes with the related boundary conditions starting from

the columns. Rigid elastic elements are placed at column ends. The same approach is also adopted for the beams. The lengths of column and beam end rigid offsets are taken as the exact length of the vertical dimension and 0.75 of the horizontal dimension of the gusset plate, respectively, in accordance with the recommendations of Hsiao et al. Braces are defined by dividing them into 12 segments to follow a sine curve with an apex of 0.2% of the brace buckling length. Similar to the beam-column elements, rigid end offsets are placed at brace ends considering the gusset plate dimensions. These rigid zones are the representations of the in-plane rigidity effect of gusset plates. As a result, the elastic behavior of the system is simulated more accurately compared to the work point to work point modeling, since the effective buckling length of braces are modeled closer to the realistic buckling length. One of the braces is modeled continuous while the counterpart is separated into two equal parts to simulate an applicable frame due to the intersection zone of two braces in X-braced frames. Also in this zone, it is required to provide a connection to attach the separated half brace parts to the continuous brace. Therefore a plate might be placed at the intersection zone. To account for the effect of this zone, rigid elements are also defined at the end zones of half brace segments that are connected to the continuous brace. Column bases and rigid offset-to-half brace segment joints at brace intersection zones are assumed to be simple connections. As previously discussed, rigid elements are placed at the brace ends and intersections to account for the effect of gusset plates. Afterwards, the nonlinear rotational springs are placed at brace end-to-rigid element nodes, in order to allow the connection to accommodate rotations while providing both rotational and translational stiffness of the gusset plate. When all framing components are modeled, gravity loads acting on the system are assigned. FEMA P-695 suggests considering the load combination of

1.0D+0.25L for the gravity loads. The loading values used for the design were assumed in the numerical model, too. 4 braced bays are assumed to be placed towards each direction. Since 4 braced bays contribute to the lateral stiffness of the system approximately equal, each braced bay is assumed to take the $\frac{1}{4}$ of the total seismic mass of each story. In brief, this approach can be imagined as the building is divided into 4 subparts by means of gravity loads and seismic mass. Based on that, the total gravity load of each story will be shared between those subparts. However, gravity loads acting on the braced frame do not include the whole loading that is acting on the related portion, they consist of only the design loads acting on the frame. It is not possible to account for the second order (P-D) effects due to this situation. To overcome that, leaner columns are implemented into the model to represent the P-D effects. Remaining gravity loads are assigned to the leaner columns at each story level. Leaner columns are modeled as elastic beam column elements with very large axial stiffness and infinitely small flexural stiffness to neglect the axial shortening effect and lateral stiffness contribution, respectively. Bases of leaner columns are also assumed to be simple connections, similar to the braced frame columns. To provide the link between the frame and leaner column elements, the equalDOF command was employed in the 1st degree of freedom. This approach simply assigns a constraint between the associated nodes of the frame and leaner columns so that their displacement in the linked directions will be equal. By utilizing the leaner columns, the second order effect due to the gravity loads and the frame's displacement in the 1st translational degree of freedom is simulated in the model. Modeling process of the frame elements is completed by the leaner column implementation.

The analysis procedure on the OpenSees can be created by using various types of integration methods, constraint handling approaches, solution algorithms and convergence tests etc. This enables the user to freely arrange the analysis parameters depending on the requirements and needs. For instance, a linear algorithm provides faster solutions however it takes one iteration to get a solution which possibly results in convergence errors, while a Newton with Line Search algorithm utilizes iterations to solve the equations and seeks for a solution at each step. So it is important to select a combination of analysis parameters to avoid a time-consuming analysis while obtaining accurate solutions without getting errors due to convergence issues. Detailed descriptions of each parameter are available at the OpenSees Command Manual. A common approach to avoid convergence errors, the analysis is divided into a number of steps as (1) time increments for dynamic analysis and (2) roof displacement increments for pushover analysis. During the analysis, it is intended to seek a solution at the end of each step and to achieve the analysis to converge successfully. The approach is based on making iterations to capture a successful convergence if the analysis fails to converge at the end of a step during the analysis. First the unsuccessful iteration is remade by decreasing the size of the step. If it still fails to converge, different types of solution algorithms are employed to reach a successful solution. Apart from those, a damping parameter is defined for the model. For instance, it is a typical approach to assume a 2% of mass and stiffness proportional Rayleigh damping, which is also adopted for this research. Before stating the pushover or dynamic analyses, it is required to perform a gravity analysis. Afterwards those analyses can take place. Also recorders are defined to collect the data regarding the analysis results. Base shear and nodal displacements, and peak ISDR are recorded in this research during pushover and dynamic analyses,

respectively. The nonlinear analytical model developed for the purposes of this research is illustrated in Figure 20.

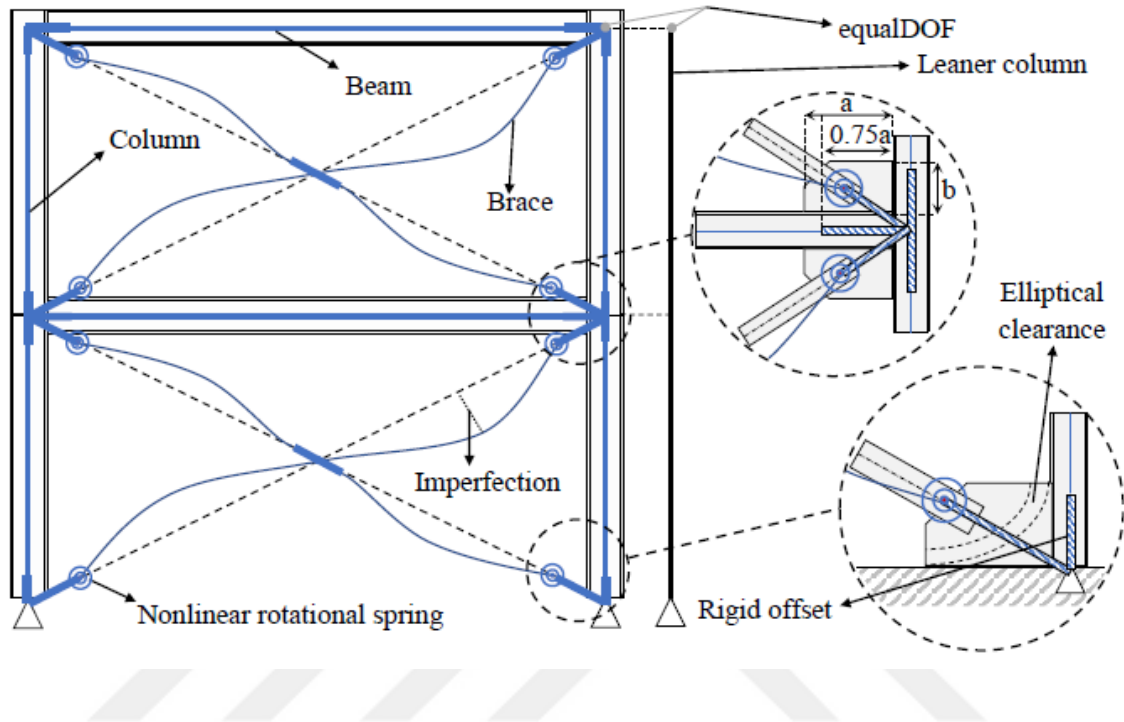


Figure 20. An illustration of the nonlinear analytical model

4.2 Parallel Analyses on DesignSAFE

Another benefit of using OpenSees is this program allows the users to perform multi-processor analyses, which are held at the same time. There are various supercomputer networks that provide such computational environments, such as DesignSAFE platform [32]. In this platform a number of nodes and processors are employed to perform the parallel operations. This provides a significant amount of time saving while it is required to conduct large amounts of analyses such as IDA. For instance, a single archetype requires 352 separate analyses under 44 ground motion records and 8 scale factors. Significant amount of time is required to complete an IDA

procedure even for a single model. So this kind of a solution is a helpful tool for researchers and enables them to produce a larger amount of data in a limited time. In this research, a separate code was used to perform parallel analyses to use the assigned processors effectively. Since there are 36 archetypes which gives a total number of 12672 analyses, this process was utilized to be able to investigate a wider range of CBF systems.

Model development process is completed with the definitions of analysis parameters. Afterwards, analysis procedures can begin with creating the models of the previously designed archetypes based on the design data retrieved from Design Excel.

CHAPTER V

SEISMIC PERFORMANCE ASSESSMENT OF CONCENTRICALLY BRACED FRAMES DESIGNED TO TBEC 2018

In this chapter, seismic assessment of concentric X-braced frames in accordance with various evaluation methods are presented based on the author's studies. Brief explanations regarding the backgrounds, methodologies and findings are provided on each method used in the research.

5.1 Assessment per TBEC 2018 Performance Evaluation Method

One of the major revisions that comes with publication of the Turkish Building Earthquake Code 2018 is the performance evaluation approach, besides the revisions in design approaches. 4 performance levels are introduced with the new code similar to the ones in ASCE/SEI 41-17 [33] as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). IO, LS and CP indicate the performance targets starting from the highest level through the lowest, respectively. Those targets are defined for DD-3, DD-2 and DD-1 level ground motion intensities, respectively. On the other hand, nonlinear time history analysis is required for the evaluation as indicated in TBEC 2018, with using a minimum 11 ground motion records. Analyses results are compared with the calculated performance targets in order to perform the evaluation.

In the study by Kor and Ozcelik [34], the performance evaluation method of TBEC 2018 was adopted to carry out an assessment on seismic performance of CBFs. In this regard, a 2-story concentric X-braced frame was designed per TBEC 2018. A different design philosophy was adopted in this study, in which the approximate period approach of TBEC 2018 was used initially to perform the seismic design of the structure. At the next step, eigenvalue analysis is performed, and the eigenvalue and approximate periods compared. If they do not match, in other words the design varies for each period, design is performed again. This iteration continues until designs at the previous and current step match. Design at the last step of the iteration is accepted as the final design. The nonlinear analytical model used for the evaluation was a simplified version that is prescribed in Chapter 4. In this model, braces were divided into 4 pieces to form an “S” shape and modeled as work point-to-work point of the beam-column joints. Also rigid end offsets with nonlinear rotational hinges were not implemented to this model. Apart from these exceptions, remaining modeling approaches that are described in the Chapter 4 were valid for this model, too. An illustration regarding this model is provided in the study. Thereafter, 11 ground motions were selected to be compliant with the considered location’s local site properties, expected magnitude of a severe earthquake and fault type for the nonlinear time history analysis. Selected ground motions were scaled in accordance with the simple scaling method specified in the code which aims to amplify the ground motions until every point on the mean response spectrum of the selected ground motions are greater than or equal to the DD-1, DD-2 and DD-3 response spectrums. In TBEC 2018, there are inelastic limits defined to specify the transitions between each performance level depending on the ground motion intensity level. For CBF systems, the limits are specified in terms of plastic rotations for

beam and column elements, and axial shortening and elongation for bracing elements. IO, LS and CP performance targets are specified considering the DD-3, DD-2 and DD-1 level intensities. Beams are the members that are typically designed for flexure; however, the case might differ in CBF systems due to a potential axial load development in beams. According to this philosophy, beams with axial load to capacity ratio higher than 0.2, which is specified by the code, were assumed to be acting like a column and the performance targets for beams were determined accordingly. But this ratio was found to be less than 0.2 for the beams in most of the cases. Independent from this threshold, beams of the considered archetype were found adequate since there are no plastic rotations observed. Plastic rotations of the columns were also deemed to be within the IO performance target region. This is an expected situation since the CBF system was designed by adopting the capacity design principles. It is intended to avoid a plasticity within the columns and beams of the system. This target was achieved for all ground motion intensity levels for the CBF design. However, it is not possible to observe the similar case for the braces. LS and IO performance targets are satisfied for the braces under DD-3 level scaled ground motions. For DD-2, or design earthquake level intensity, the inelastic deformations were found to be within the LS and CP limits generally, although there were a couple of analysis points beyond the CP region. Especially the compressive forces developed in braces caused significant amounts of plastic deformation. But on average, the performance level of the braces was observed as CP for the design earthquake level. Most of the ground motions scaled to the maximum considered earthquake level resulted in potential total collapses of the CBF, since there were dramatically large inelastic axial shortenings obtained in the braces of both stories. Regardless of evaluating as individual or average, except for a number of

analyses, all GMs caused deformations up to 5 times of the CP target and the performance of the CBF was far beyond the CP limits. In brief, a potential failure was observed especially in compression under the effect of maximum considered earthquake.

5.2 IDA of a Concentric X-braced Frame Designed to TBEC 2018

The Incremental Dynamic Analysis (IDA) method is an effective tool for measuring the structural response under seismic actions. By utilizing the IDA method, the effect of infrequent ground motions on different lateral load resisting systems can be understood. An elaborate description regarding the IDA approach is given in Chapter 2. To briefly give a summary on application of this method, the archetype is subjected to a set of ground motions that are scaled incrementally until a collapse is observed for each analysis. Incrementing the ground motions provides an insight into the dynamic behavior of the considered system at different intensities of ground motion records. In general, it can be observed from the IDA how the response of the structure varies under increasing ground motion intensity.

In this study [35], a 2-story concentric X-braced frame was designed in accordance with the requirements of TBEC 2018. The same nonlinear analytical model used for the previous study was also utilized for this study. 12 ground motions were selected based on the approach in which the expected magnitude, local site class and fault characteristics at the building's location to be parallel with the selections. The selected ground motion records are scaled to intersect the points that correspond to the structural period on response spectrums of each ground motion record and the design

response spectrum. Collapse limit states were specified in terms of damage and intensity measures. A damage measure can be expressed as the physical deformation or a substantial parameter such as interstory drift ratio or peak base shear, respectively while an intensity measure can be defined as a function of the level of loadings imposed to the system. In this study, maximum interstory drift ratio and spectral acceleration were employed as damage and intensity measures, respectively. Accordingly, the system was assumed to achieve a collapse state if the maximum interstory drift ratio exceeds 5% (DM rule) or if the tangent slope between consecutive IDA steps is less than 20% of the initial elastic slope (IM rule). After the collapse states were specified, the IDA procedure was started with initial scale factors and increasing them at the end of each step, until a collapse state is triggered.

Based on the results of the IDA procedure, the collapse data of the system was obtained and recorded. Collapse fragility curve of the system was constructed as a function of the recorded collapse data, by applying a lognormal cumulative distribution function with a dispersion of 0.6. The background of this approach lies on a curve fitting that aims to accurately match the collapse probability at each scale factor of the IDA procedure. The median collapse spectral intensity, $\hat{S}_{CT}$, was obtained from the fitted curve, namely fragility curve, as the spectral acceleration that corresponds to the point on the curve where the collapse probability of the CBF is 50%. This value reflects the spectral acceleration at which 50% of the ground motions cause failure in other words. $\hat{S}_{CT}$ value of the system was observed to be 2.28g from the fragility curve. To make the evaluation, the median collapse spectral intensity is divided by the spectral acceleration on the maximum considered earthquake response spectrum for the corresponding structural period, S_{MT} , which results in the collapse margin ratio (CMR).

CMR of the CBF was calculated as 1.37, as a function of the ratio of $\hat{S}_{CT}$ to S_{MT} . The collapse margin reveals that the median collapse spectral intensity of the system is almost 1.4 times greater than the spectral acceleration of a major earthquake expected. In conclusion, the seismic performance of the CBF designed per TBEC 2018 was deemed adequate in terms of collapse behavior.

5.3 Collapse Assessment of Concentrically Braced Frames Designed to TBEC 2018

During infrequent major earthquakes, lateral load resisting systems are subjected to intensely large loads that are expected to cause plasticity in various members depending on the load distribution mechanism within the system. For instance, beams are the typical members that are desired to absorb the energy due to seismicity for moment resisting frames while links are employed to be the fuse elements of eccentrically braced frames. Likewise, the inelasticity is desired to be developed along the bracing member to avoid a potential instability arising from an unintended yield mechanism. These design philosophies are employed to allow inelastic deformations of the lateral load resisting systems which provide the energy absorption and ductility during a large earthquake, while maintaining the global stability of the structure. However, due to complexity of the plastic behavior, the seismic response of the fuse elements must be well understood to create safe, efficient and economic designs. This situation is even more sophisticated for X-braced frames as well as the other concentrically braced frame configurations since the buckling initiation and post buckling behavior of braces are challenging phenomenon to estimate and represent in an

analytical model. There are various experimental and analytical studies that are aimed to investigate the seismic behavior of both individual bracing members and CBF systems in order to provide a better understanding on CBF inelastic response and better estimate on brace buckling behavior. Both national or international seismic design codes and specifications on structural steel design are also utilized to provide design principles and requirements, in order to result in more efficient designs by means of seismic performance and to provide life safety during an infrequent major seismic event. To name one of those seismic design codes among many of them, TBEC 2018 is the latest version of the Turkish national seismic design codes, which was published in 2019 with a remarkable number of revisions compared to the previous versions. An elaborative discussion on the revisions and new design approaches introduced with TBEC 2018 are presented in previous chapters and appendices. Despite the revisions introduced in the new code, which are expected to affect the seismic design and performance, there is a limited number of studies that focus on this topic [34,35]. Therefore a study was conducted to provide a better insight into the seismic performance of CBFs designed per TBEC 2018.

The final study regarding the seismic performance of CBFs designed per TBEC 2018, [36], is presented herein. A more sophisticated and accurate but also efficient nonlinear numerical model was developed on OpenSees, based on the previously used model. However, various improvements were made to increase the accuracy of the model. To name those differences, rigid end offsets were implemented for the purpose of taking the additional translational and rotational stiffness due to the gusset plate connections into account. Similarly, nonlinear rotational springs were placed between the rigid end offsets and brace elements. Besides that the braces were divided into 12

segments to better represent the second mode buckled shape of the compression member. Also a material model that is wrapped onto the primary material of braces and is capable of measuring and recording the damage accumulation within the wrapped section was included to the model, in order to capture the brace fracture following the large inelasticity due to brace buckling. A comprehensive discussion on those improvements is provided in Chapter 2 and Appendix C.

The number of archetypes is increased in this study to better represent and investigate the effect of differences in structural geometry. For instance,

- number of stories,
- bay widths (span length) and
- seismic design categories

were considered during the archetype selection, which gives a total number of 36 archetypes. The purpose behind that is to be capable of observing the seismic response and collapse performance of CBFs with varying structural period, aspect ratio and seismicity. Performing the assessment with a wider range of archetypes also provides a larger data to investigate, which can increase the accuracy of the assessment by reducing the effect of potential “outliers”. All archetypes were designed with regards to the requirements of TBEC 2018, including the design limitations. For instance, the code allows a maximum total building height of 42 meters for concentrically braced frames, so the tallest archetype was designed to be 12 stories with 3.5 meters constant story height which sums up to 42 meters in total. On the other hand, different from the designs of the previous studies, gusset plate connections were implemented to the CBF design. Widely preferred plates that are highly available in the market were selected for

the connection design and the gusset plate connection dimensions are tabulated in Table- 5 and Table- 6. The balanced design procedure (BDP) with elliptical clearance model was adopted for the gusset plate design. As suggested by this approach, all limit states including plate buckling and yielding and weld fracture, were taken into account to avoid an undesired premature, brittle or sudden failure. Background, verification and application of those design approaches as well as the other design principles that are adopted for the structural design are provided in Chapter 2 and Chapter 3.

FEMA P-695 utilizes a quantification methodology based on a probabilistic approach, in which a set of archetypes selected and designed to be analyzed using nonlinear static and nonlinear dynamic analysis procedures. Here, a brief summary on the Methodology is provided. The nonlinear static analysis procedure is performed to obtain the structural lateral response by establishing the system overstrength and the period-based ductility. On the other hand, the nonlinear dynamic analysis procedure is required to collect the collapse data. The Incremental Dynamic Analysis method (Chapter 2) is adopted as the primary analysis method of NDA. After both analysis procedures are completed, the collapse data is processed for the purpose of assessment. First, FEMA P-695 employs a parameter named collapse margin ratio (CMR), which is the ratio of median collapse spectral intensity that is calculated as the intensity where half of the GMs cause collapse of the archetype, to the spectral acceleration at maximum considered earthquake level. CMR is then transformed into adjusted collapse margin ratio (ACMR) by amplifying with the spectral shape factor. At the end, ACMRs are compared with the target ACMRs that are defined separately for individual or group of archetypes to decide if a design variable such as response modification factor or deflection amplification factor fails or passes the quantification. However, this study did

not aim to verify the design variables, but it aims to assess the seismic performance of CBFs designed to TBEC 2018 by means of collapse capacity. Therefore, instead of establishing the CMRs, the collapse data was used to construct fragility curves to obtain the median collapse spectral intensities. For the purpose of acquiring a better estimate on the median collapse spectral intensity, a full IDA approach was adopted in which the analysis continues until all GMs cause collapse instead of stopping the analysis at where 50% of the GMs cause collapse. Based on the results of this approach, collapse fragilities of each system were constructed, and the median collapse capacities were obtained. Then the Methodology requires accounting for various sources of uncertainty which include the variations that can influence the analysis results. The sources of uncertainties are based on the quality, quantity, availability and robustness of design requirements, test data, modeling approach and GM records. A total system uncertainty is then defined as a function of four random variables which are statistically independent, and it gathers all sources of uncertainty together. Using the median collapse spectral intensities and total system uncertainties, collapse fragilities of each system were re-constructed to obtain the median collapse capacities.

Based on the collapse fragilities, collapse probabilities of each archetype were calculated with respect to the design earthquake level (DBE) GM intensity. In order to obtain the collapse probabilities under maximum considered earthquake (MCE) GM intensity, it is required to transform the DBE probabilities to MCE level, MCE/DBE ratio should be specified. Despite this ratio being specified as constant in ASCE/SEI-07 as 1.5, TBEC 2018 does not define such a ratio for the transformation. To that end, mapped spectral acceleration coefficients were retrieved from Turkish national seismic maps, and the MCE to DBE ratios are calculated and referred as τ , which are observed

to be ranging between 1.40 and 1.75. Those minimum and maximum values of τ ratios are considered, namely 1.40 and 1.75. The DBE level collapse probabilities are then transformed to the MCE level by amplifying with the τ ratios, and the calculated collapse probabilities are employed to be the collapse probability under the effect of maximum considered earthquake.

According to the results, the most remarkable difference in collapse performance of the archetypes is observed to be caused by the seismic design category. Collapse probabilities of SDC $D_{\max}$ archetypes are relatively higher than SDC $C_{\max}$. Since the effect of an infrequent earthquake that can be expected for the SDC $D_{\max}$ is intensely large compared to the SDC $C_{\max}$, this is not a surprising situation. However, collapse probabilities of SDC $D_{\max}$ archetypes are found to be larger than 0.20, which is suggested by FEMA P-695 as the maximum allowable collapse probability for individual archetypes. Even in a number of archetypes, the probability is jumped far beyond 0.50 for τ ratio of 1.75. On the other hand, SDC $C_{\max}$ archetypes were able to remain below 0.20 level for both τ ratios, despite 2 exceptions with probabilities above the 0.20 level. However those exceptions do not represent the general behavior of SDC $C_{\max}$ archetypes since their collapse probabilities were found extremely high which are apparently diverged from the group average; and accordingly they were evaluated as outliers. Another difference that affects the collapse performance of CBFs is the number of stories which is directly proportional with the structural period. For both SDC $D_{\max}$ and SDC $C_{\max}$ archetypes, long period archetypes were observed to be more critical by means of collapse performance in comparison with short period archetypes. In particular, long period SDC $D_{\max}$ archetypes clearly exceed the allowable collapse probability, 20%, while almost the entire SDC $C_{\max}$ group and most of the short period

SDC $D_{\max}$ archetypes are able to remain below the limit in general. The aspect ratio of the braced frame is not found to have a significant effect on collapse performance. However, BF1 archetypes have slightly less collapse probabilities on average, compared to BF2 and BF3 with shorter span lengths, although the difference is not as apparent as it is in SDCs and structural periods.

In this study, the major purpose was to perform an assessment on collapse performance of CBFs. Thus, a set of archetypes that are selected considering several design variables that might potentially influence the seismic behavior and designed by adopting several design approaches that are available in the literature and design specifications. By utilizing the Methodology, nonlinear analyses were performed to obtain the structural response and collapse data, which are then used to establish the collapse fragilities and probabilities of failure. Results revealed that the collapse probability is directly proportional with increasing seismicity and structural period, which are considered in the archetype selection in terms of seismic design category and number of stories as it was mentioned previously. Especially the seismicity has the most negative impact on seismic performance of CBFs with respect to the assessment. Collapse probabilities of SDC $D_{\max}$ archetypes under MCE level GMs were found dramatically high in comparison with the SDC $C_{\max}$.

CHAPTER VI

CONCLUSIONS

Concentrically braced frame systems have been widely preferred lateral load resisting systems and commonly employed for the seismic design of steel structures. They provide large stiffness and lateral strength and led to a comparatively economical design. Despite their benefits by means of stiffness, their inelastic behavior during a major earthquake is relatively complex and it must be well investigated. In many years, a significant number of improvements have been employed including the special detailing requirements and design philosophies. Similarly, the latest Turkish seismic design code, TBEC 2018, also introduced such revisions in design and detailing approaches for CBF systems.

In this research, the primary objective was to assess the seismic behavior and collapse performance of CBFs designed per TBEC 2018, which was published with various revisions and new design approaches. The revisions have remarkable effect on CBF designs and they are potentially expected to influence the seismic performance, too. To that end, 3 studies were carried out to better understand the seismic behavior and collapse performance of CBFs. These studies are focused on

- 1) performance evaluation of a CBF based on TBEC 2018,
- 2) IDA of a TBEC 2018 CBF design and
- 3) collapse performance assessment of CBFs designed per TBEC 2018.

In brief, the primary objectives of the studies are seismic performance assessment of CBF systems designed per TBEC 2018. Several archetypes, analytical models and assessment techniques were used for the studies.

- Results of the first study showed that the CBF designed per TBEC 2018 was capable of resisting the seismic effects of two GM intensity levels without a potential collapse, however, the MCE level scaled GMs caused a potential failure of the braces which might possibly result in a total collapse of the frame.
- The incremental dynamic analysis procedure was adopted in the second study in order to reveal the structural response for several intensity levels of the imposed ground motion records. In accordance with the IDA results, collapse fragility of the considered CBF was constructed, and the collapse margin ratio (CMR) was reported with the probability of failure value. Calculated CMR was compared with respect to the allowable margins, which revealed that the system showed satisfactory performance in terms of collapse behavior.
- The last study was relatively more comprehensive because of the wider range of archetypes, an improved analytical model and elaborative assessment method. A total of 36 archetypes were selected and designed considering 3 different design variables including the structural geometry and seismicity. A more accurate nonlinear analytical model was developed that contains several modeling approaches which aim to enhance the accuracy. Nonlinear static and dynamic analyses were performed on CBF designs using the numerical model. Results of these procedures were processed to construct the collapse fragilities, which are defined as a function of median collapse spectral accelerations and system uncertainties. From the collapse fragilities, collapse probabilities of each

archetype were observed and reported. The relation between increasing seismicity and structural period with collapse probability, which are found to be directly proportional, and the effect of other design variables on collapse performances of CBFs were presented and discussed.

Seismic performance of CBFs designed per TBEC 2018 was investigated by adopting several assessment approaches in all 3 studies. The major observation is long period CBFs are highly vulnerable in high-seismicity regions especially. On the other hand, when the CBF systems are employed as lateral load resisting systems, they provide safeguard against large earthquake loads and they are able to resist relatively low, frequent earthquake loads with limited deformations which is an intended response for the serviceability. However, care must be taken for using the CBF systems in high-seismicity regions and longer structures.

APPENDIX A

**CONFERENCE PAPER: SEISMIC PERFORMANCE
EVALUATION OF STEEL X-BRACED FRAMES PER
THE TURKISH BUILDING EARTHQUAKE CODE 2018**

Conference Paper

Efecan Kor, Dr. Yiğit Özçelik

Presented and Published at the 9th Turkish Conference on Earthquake

Engineering, hosted in Istanbul, Turkey

<https://tr.9tcee.org/>

Abstract

Concentrically braced frames are acknowledged to be a reliable lateral force-resisting system with high lateral elastic stiffness and strength. X-braced frames are one of the configurations of concentrically braced frames in which two braces in each story are placed diagonally between the beam-column connections crossing each other in braced bays. Resisting lateral loads with truss action, one of the braces develops axial tension while the counterpart develops axial compression. These braces are designed as the fuse elements of the system such that the tension brace yields and the compression brace buckles to provide energy dissipation. The Turkish Building Earthquake Code 2018 (TBEC) was released in 2018 and superseded the Turkish Earthquake Code 2007 (TEC) as of January 2019. There are major differences in these two codes one of which is the introduction of yield mechanisms for steel frames. Similarly, there are several changes in the code such as the introduction of new seismic maps and change in spectral shape which affect the magnitude of the design earthquake force. As a result, X-braced

steel frame designs per TBEC are likely to yield different member sizes compared to the designs per TEC. In addition to the changes related with design, changes related with performance assessment have been introduced in TBEC such as the deformation limits for steel members. In this study, a two-story X-braced frame was designed per TBEC to assess its seismic performance according to the deformation limits provided in TBEC. The designed X-braced frame was subjected to three sets of ground motions which were scaled to match the seismic hazard levels specified in TBEC for the defined earthquake ground motion intensity levels. The plastic deformations of beams, columns, and braces were obtained from the nonlinear response-history analysis and compared with the deformation limits in the code. The results of this study showed that the beams remain elastic for all seismic hazard levels considered in the study. The columns experienced plastic rotations at floor levels due to nonuniform drift distribution along the building height; however, the plastic rotations are within the limits given in TBEC. The braces underwent significant inelastic action exceeding the deformation limits of TBEC; therefore, collapse might be observed for the frame designed per TBEC based on the deformation limits of TBEC.

Keywords: Steel structures, X-braced frames, seismic behavior, nonlinear response-history analysis, seismic performance evaluation

Introduction

Concentrically braced frames (CBFs) are one of the most common lateral load resisting systems that provides high lateral load resistance and lateral stiffness. In such

systems, lateral loads are mainly resisted by the diagonal elements, namely, braces that primarily work under axial load effects. X-braced frames are a commonly used configuration among several CBF systems where braces are placed between beam-column joints of sequent floors and cross each other. In X-braced frames, one of the braces in the braced bay resists compression while the other resists tension. The energy during an earthquake is dissipated through the inelastic buckling and yielding of the braces under compression and tension, respectively. In other words, the braces are deliberately designed as the fuse elements of the system to dissipate energy. In order to achieve such plastic mechanism, the beams, columns, and connections (i.e., all elements but the braces) must be capacity-designed. In case of plastic deformations in elements such as columns and beams, stability problems may arise that might lead to partial or total collapse of the structure.

The current Turkish seismic code, Turkish Building Earthquake Code 2018 (TBEC) (TBEC, 2018), was came into effect in January 2019 and took the place of the former code, the Turkish Earthquake Code 2007 (TEC) (TEC, 2007) with several remarkable changes in many design and assessment methods. One of the most fundamental changes is the revision in the seismic hazard maps and introduction of seismic hazard levels. In the former seismic hazard map adopted in TEC, Turkey was divided into 4 seismic regions (and one non-seismic region) and a peak ground acceleration coefficient was assigned to each region to define the seismic hazard for a return period of 475 years. In TBEC, however, the seismic hazard is defined for earthquake ground motion intensity levels (namely, DD-1, DD-2, DD-3, and DD-4 with return periods of 43, 72, 475, and 2475 years, respectively.). Using a web-based platform maintained by AFAD (AFAD, 2018), the peak ground acceleration, spectral

acceleration at 0.2s period, and spectral acceleration at 1.0s period can be retrieved for a particular location for each seismic hazard levels. Another major difference is the effect of local soil class on the amplitude of spectral acceleration. The local soil class was considered only in defining the corner period values of response spectrum in TEC. The local site class, however, is used in TBEC to adjust the spectral acceleration at 0.2s period, and spectral acceleration at 1.0s period retrieved from the web-based platform. Similarly, the functions provided for the seismic force reduction factor, $R_a(T)$, are different in TEC and TBEC, that affect the reduced spectral acceleration values. A comparison of the elastic and reduced response spectra of TEC and TBEC is provided for Maltepe, Istanbul in Figure 1.

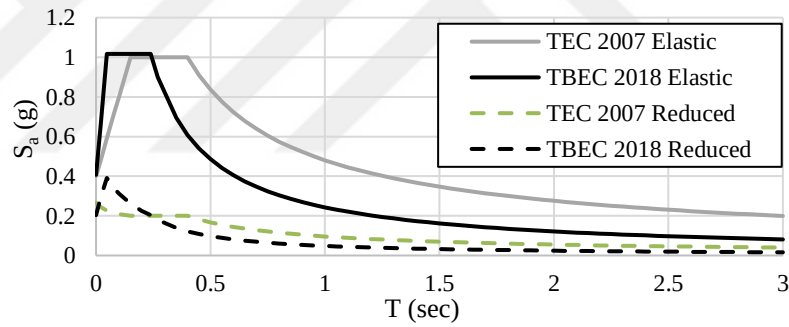


Figure 1. Elastic and reduced spectrums of TEC and TBEC

In addition to the changes mentioned herein affecting the magnitude of the earthquake load, another change in TBEC is the introduction of two yield mechanisms for concentrically braced frames where the internal forces in the columns, beams, and connections are determined considering pre- and post-buckling responses of the braces to capacity-design these elements. These two mechanisms are to be used to determine the maximum possible forces to be resisted by all elements of CBFs but the braces when the braces reach their expected load carrying capacity either in tension or compression. To that end, the expected tension strength of a brace is calculated using the expected

yield stress, $R_y F_y$, in lieu of the nominal yield stress. Similarly, the compression strength is calculated using the expected buckling stress, $1.14 F_{cr}$, instead of nominal buckling stress, F_{cr} . In the first mechanism, all of the tension and compression braces are assumed to develop their expected tension and compression strength, respectively. All CBF members but the braces are designed for the corresponding internal forces. In the second mechanism, the same forces are assumed for tension brace; however, the compression braces are assumed to resist 30% of their expected compression strength which corresponds to the post-buckling strength of the braces. The yield mechanisms are illustrated in Figure 2 in detail. Please note that TEC also has a capacity design approach that requires the columns, beams, and connections to be designed for earthquake forces amplified with a factor called overstrength factor. The same approach is also considered in TBEC; however, TBEC requires designing per the yield mechanisms too.

There are several experimental and analytical studies aiming at understanding the seismic behavior of CBFs. To name a few examples among many others, Tremblay et al. (2003) and Uriz & Mahin (2004) investigate the seismic behavior of CBFs. Similarly, Hsiao et al. (2013) evaluates the collapse performance and response modification factor of special CBFs. Despite several efforts made to understand the seismic behavior of CBFs, there has been no study, to the authors' knowledge, to evaluate the seismic performance of CBFs designed to TBEC. As there have been significant changes in TBEC with respect to TEC that are likely to have a significant impact on member sizes as well as the alterations in the seismic hazard maps, there is an urgent need to quantify the seismic performance of CBFs designed to TBEC and verify that the CBFs designed to TBEC perform under seismic actions as intended. This study

aims to evaluate the seismic performance of X-braced frames. To that end, a two-story CBF was designed per TBEC. Nonlinear time history analyses were performed under three ground motions sets scaled to the earthquake ground motion intensity levels DD-1, DD-2, and DD-3. The ground motions were selected to be compatible with the expected conditions in the design location such as magnitude range, shear-wave velocity ($V_{s,30}$) range, and fault type. The demands in the members were compared with the deformation limits for steel member in TBEC.

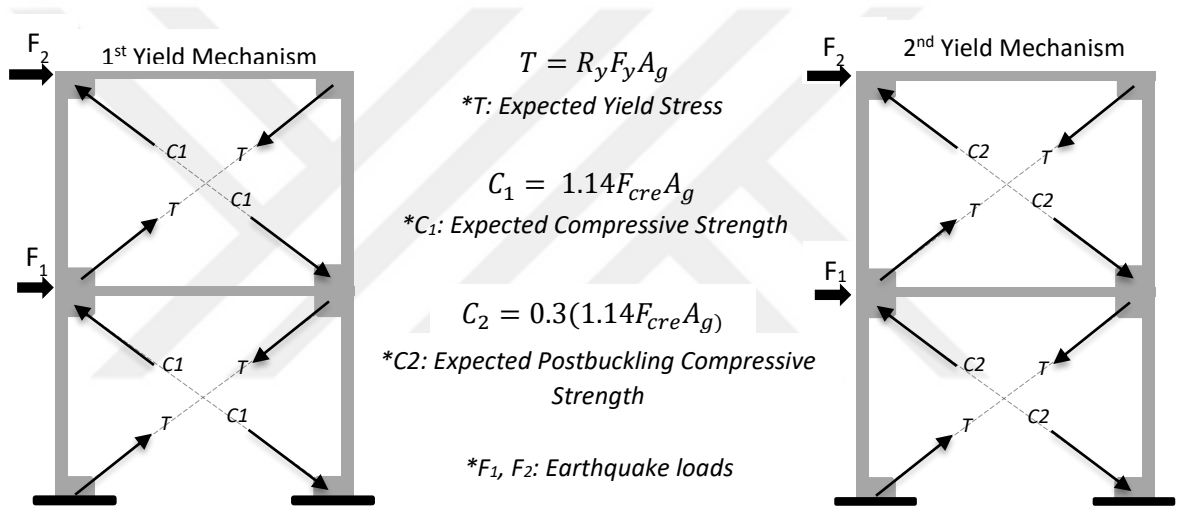


Figure 2. Yield mechanisms defined in TBEC

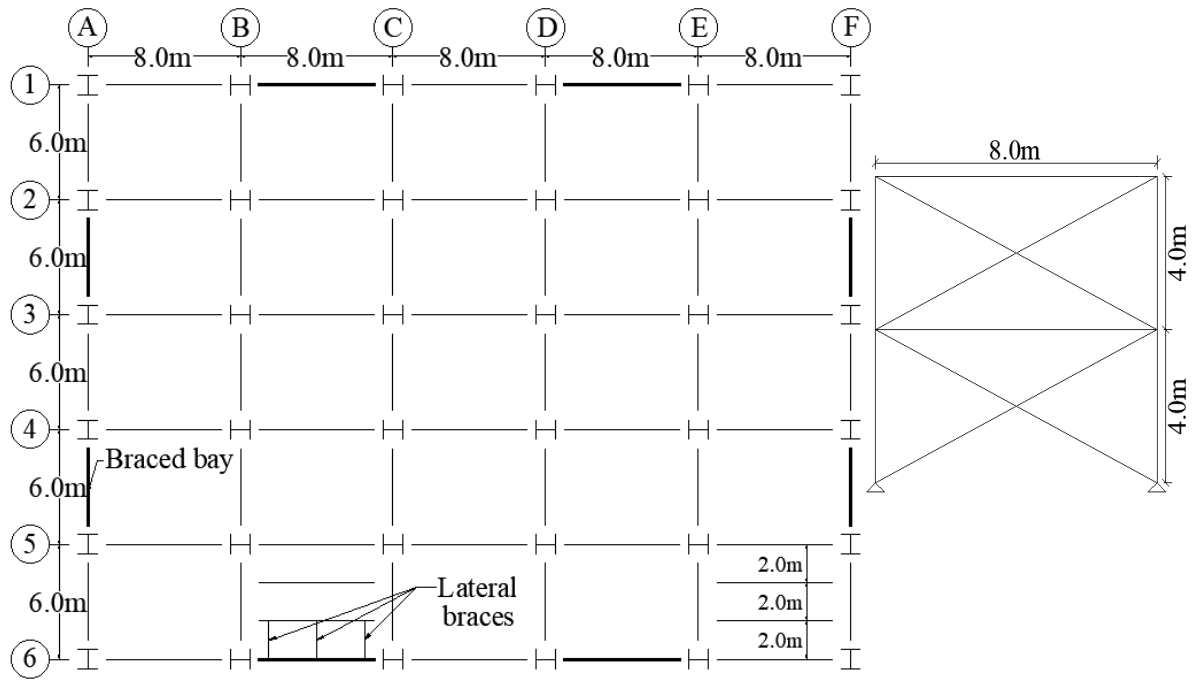


Figure 3. Plan and elevation views of the building

Design of the X-Braced Frame

The structural design of a two-story X-braced office building located in Maltepe, Istanbul (40.912424°, 29.13445°) was carried out in accordance with the methods defined in the Turkish Specification of Steel which consists of 5 bays in each face with 8- and 6-meter bay widths in the NS and EW directions, respectively. 2 out of 5 perimeter bays consist of X-braced frames symmetrically to provide lateral load resistance to the structure and avoid any undesired significant torsional effects. The story height is assumed to be 4 meters and constant for each floor. Plan and elevation views of the building are provided in Figure 3. The seismic design was carried out only for one braced bay in the NS direction assuming that there is rigid diaphragm that distributes to the earthquake forces to the vertical braces equally. Regarding to the occupancy type and dimensional properties of the building, the live load mass participation factor (n), the building occupancy category (BKS), the building height

class (BYS), and the building importance factor (I) were obtained from the related tables of TBEC as 0.3, 3, 7, and 1, respectively. The steel grade of S235 was assumed for the braces while S355 was adopted for beams and columns. The dead, live, and snow load were assumed to be 5 kPa, 2 kPa, and 2 kPa, respectively. P- Δ effects were included using the approximate method given in TSSS. The response spectrum was constructed per TBEC for a short-period spectral acceleration (S_s) of 1.017g and a 1 second period spectral acceleration (S_1) of 0.243g that were retrieved from the Turkish Seismic Hazard Maps of AFAD for the DD-2 earthquake ground motion intensity level for Maltepe. Depending on the assumed local soil class of the location, ZB , response modification factor (R) calculated from the approach defined in TBEC as 5 and I , a reduced design response spectrum was created in terms of reduced design spectral acceleration, S_{aR} . The total seismic mass of the structure was calculated as 1370 tons and the earthquake loads were calculated adopting the equivalent lateral load (ELF) procedure described in TBEC. The fundamental period of the structure was calculated using the approximate method given in TBEC. The fundamental period and the base shear force were calculated as 0.38s and 1700 kN, respectively. A 5% eccentricity was considered while calculating the earthquake force resisted by one braced bay to account for the accidental torsion effects. After performing the preliminary design of the braced bay considering the earthquake forces calculated based on the approximate fundamental period, an iterative approach was adopted such that the structural design was repeated until the period used to determine the earthquake forces and the period obtained from the Eigen Analysis performed at the end of each step match. Consequently, the underestimation of the earthquake forces associated with the differences in the assumed fundamental period and calculated period was eliminated. At the end of the iterations,

the fundamental period was calculated as 0.27s and the base shear was determined as 2400 kN (which yields a base shear coefficient of 0.178). The final member sizes of the braced bay are tabulated in Table 1. A flowchart summarizing the design procedure is provided in Figure 4.

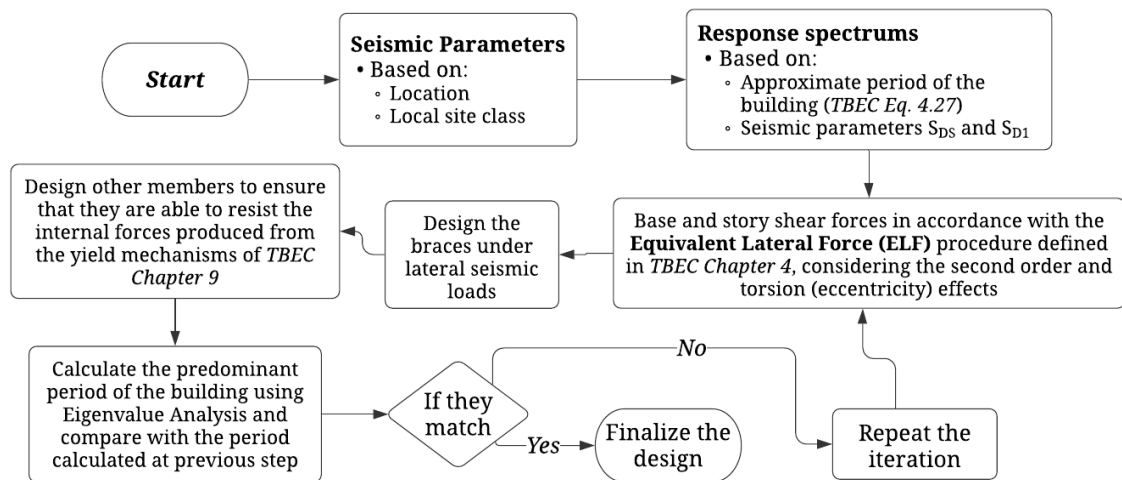


Figure 4. Flowchart of the design procedure

Table 1. Member sizes of the X-braced frame

Story	Column	Beam	Brace
1	W200x86	W250x115	HSS220x220x5
2	W200x86	W250x89	HSS180x180x4

Analytical Model

Modeling and Nonlinear Dynamic Analysis

A two-dimensional model of the designed X-braced frame was constructed in OpenSees, open-source simulation software for earthquake engineering (McKenna et al., 2016). All members such as beams, columns, and braces were modeled using

distributed plasticity force-based nonlinear beam-column elements. The interaction of the flexural and axial demands was achieved through fiber sections, which is one of the modeling approaches defined in TBEC Chapter 5 as distributed plasticity behavior model, with integration at 5 points along the member lengths whose locations were selected in accordance with the Gauss-Lobatto quadrature rule (Scott 2011). The steel material available in the OpenSEES material library, Steel02, was used for all members with a strain hardening ratio of 0.3%. The Steel02 material follows the hysteretic model proposed by Giuffre-Menegotto-Pinto with isotropic strain hardening. The geometric nonlinearities were taken into through corotational transformation. To simulate the buckling of the braces, the model proposed by Uriz et al. (2008) was adopted. In this brace model, the brace is divided into at least two sub-elements with an initial imperfection in the nodes between the end nodes of the brace. In the model considered in this study, each brace was divided into two and an initial imperfection (camber) of 0.1% of the brace length was assigned at the mid node of the brace. To account for the P-D effects, a leaner column was added to the model representing the gravity system. As one braced bay is expected to provide lateral resistance to a quarter of the building, one quarter of the gravity loads acting on each floor was applied to the leaner column at floor levels. The flexural stiffness of the leaner column was assumed to be negligible just to conservatively ignore the possible contribution of the gravity frame to the lateral force resisting system, i.e., braced frame. All connections such as beam-to-column, beam-to-brace, and column bases were assumed to be simple connections. The details of the modeling approaches are illustrated and explained in Figure 5.

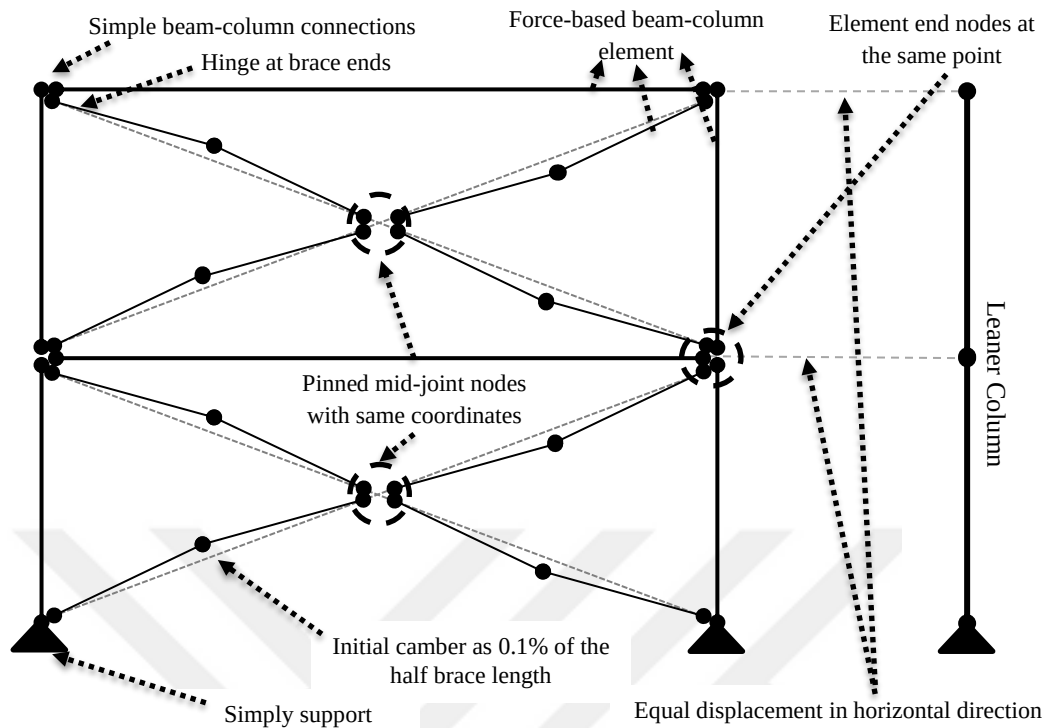


Figure 5. Illustration of the numerical model

To validate the numerical model of the X-braced frames constructed in OpenSEES following the modeling approaches explained herein in detail, a numerical model of a x-braced frame tested by Palmer et al. (2012) was built. The Palmer's specimen was a full-scale one-story one-bay x-braced frame with HSS braces with slotted tube knife plate type connections. The specimen was subjected to a cyclic displacement pseudo-static loading protocol based on the ATC 24 protocol (ATC 1992). Figure 6 shows the comparison of the base shear vs. story drift of the experimental results and the OpenSEES results. It appears to indicate that the numerical model mimics the experimental response satisfactorily. For story drifts less than 0.5%, the numerical model underestimates the lateral strength and stiffness; however, for larger drifts, the numerical model is capable of capturing the strength and stiffness adequately. The difference between the results can be attributed to the fact that the braces were

ideally assumed to have pinned ends, which affects the stiffness for small drifts as limited nonlinear action was observed in the experiments and the gusset plates did not undergo significant inelastic action as expected. However, the difference in the lateral strength becomes insignificant as drift increases because plastic hinges formed at the brace-gusset plate interface that show a similar behavior to the end connection assumed for the brace in the numerical model. Another reason for the deviation might stem from the fact that the members were modeled used their center-to-center length that might lead to more flexible system. In brief, the numerical model was deemed adequate for the purposes of this study despite difference between the experimental and numerical results in the elastic range.

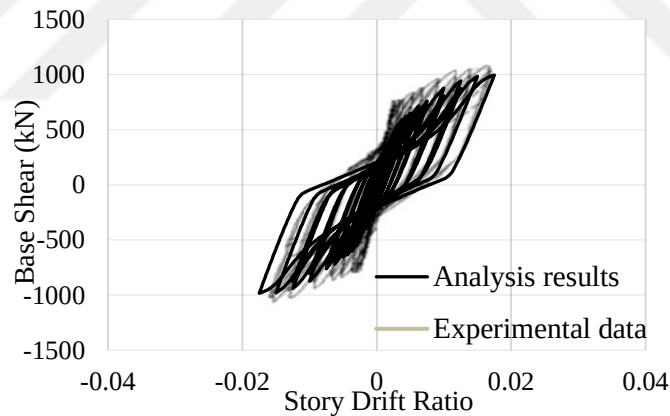


Figure 6. Comparison of experimental and analytical results

To assess the seismic performance of the X-braced frame designed per TBEC, nonlinear dynamic analyses were performed. To that end, eleven ground motions were selected from the PEER NGA West-2 database (PEER, 2020) for the dynamic analysis which are consistent with the fault characteristics at the site. Figure 8 shows the response spectra of the selected ground motions that are named in accordance with their record sequence numbers in the PEER database. The response spectrums are provided

as 5% damped since it is specified as a requirement in TBEC. The ground motions were scaled three times to match the response spectra of TBEC for the earthquake ground motion intensity levels, DD-1, DD-2, and DD-3, adopting the ground motion scaling procedure explained in detail by Fahjan (2008). Note that the simple scaling defined in TBEC requires the mean response spectrum of the selected ground motions to match the elastic response spectrum of TBEC such that the spectral acceleration values of the mean response spectrum shall not be less than the TBEC response spectrum between $0.2T_p$ and $1.5T_p$, where T_p is predominant period of the structure. Although several ground motion records were tried to satisfy this requirement, it was not possible to perfectly match the spectrums as it is described, since there are too many scatters due to the peaks within the constant acceleration region of the spectrums. So the scaling procedure was performed with a conservative approach. The nonlinear dynamic analyses were performed at those three earthquake ground motion intensity levels (namely, DD-1, DD-2, and DD-3). Consequently, the X-braced frame was subjected to eleven ground motions with three scaling factors for each, which added up to 33 nonlinear time-history analyses. In the analysis, a Rayleigh (mass and stiffness proportional) damping was assumed and a damping ratio of 2 % was conservatively assigned to the first two modes of the numerical model, since 2% is a generally accepted and preferred damping ratio for steel structures. The seismic assessment methodology was summarized in a flowchart in Figure 7.

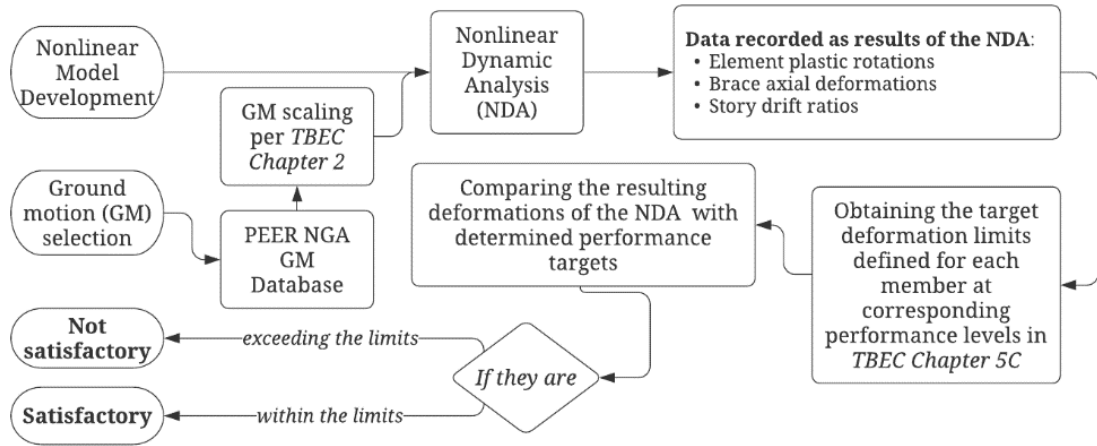


Figure 7. Flowchart scheme of the seismic assessment methodology

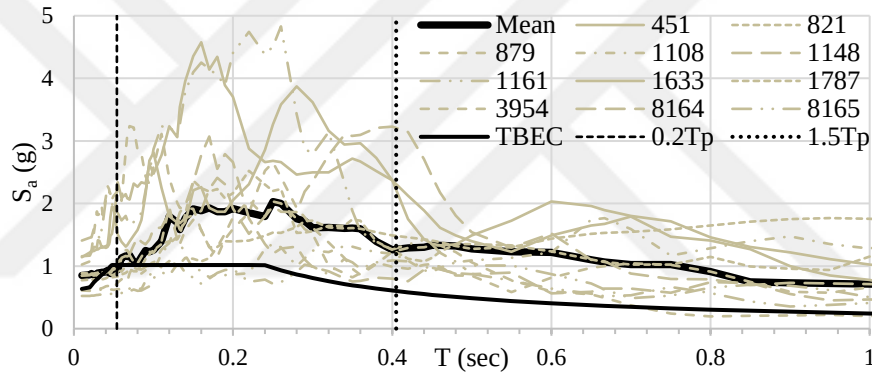


Figure 8. Response spectra of the selected ground motions (DD-2 scaled, 5% damped)

Results and Discussion

TBEC introduces displacement limits for beams and columns in terms of plastic rotations as well as for braces in terms of axial deformations depending on the defined performance levels which are immediate occupancy (IO), life safety (LS), and collapse prevention (CP). Also, TBEC defines inelastic capacities for CBF members in terms of plastic rotation (θ_y) for beams and columns as well as axial shortening (Δ_c) and elongation (Δ_T) for braces under compression and tension, respectively. For columns, it is required to obtain the plastic rotation capacity as a function of axial load/capacity

(P/P_c) ratio. 2 different cases are introduced in TBEC for columns as P/P_c ratio greater or lesser than 0.2. For greater values, a function is provided in the code in terms of plastic rotation capacity and P/P_c , which enables to calculate the limiting values of performance levels which are shown in Figure 9a, 9b and 9c. Inelastic deformation limits for columns with P/P_c ratio lesser than 0.2 and beams are defined independent of axial load level, however, beam plastic rotation capacity formula given in the code does not include P/P_c which is the only difference between inelastic capacity calculations of beams and columns with P/P_c lesser than 0.2. On the other hand, compressive and tensile inelastic capacities of braces are defined in terms of axial deformation, as tabulated in Table 2, which are involved in performance limit calculations. In this case, beams were considered as column elements initially, since a potential truss action can increase the axial loads on a beam. So that, beams were evaluated as if they act like a column element and deformation limits for columns were considered to obtain the performance of beams. Accordingly, not only plastic rotations but also axial loads acting on beams were obtained from the nonlinear dynamic analysis results. But most of the beams were found to have P/P_c ratio lesser than 0.2 and evaluated as a beam element. According to the analysis results, no plastic rotations were observed in the beams for all intensity levels. Similarly, columns remained elastic for DD-2 and DD-3 levels, however, there were 3 analysis points noticeable plastic rotation observed with a maximum value of 0.36 in terms of normalized plastic rotation (θ_p/θ_y). But as it can be observed from the Figure 9c, these points were within the IO limits which shows that performance of columns were satisfactory. Plastic rotations in columns can be expressed as differential drifts of sequent stories might cause a rotation demand on end joints.

Table 2. Normalized axial deformations (Δ/Δ_c for compression or Δ/Δ_T for tension) developed in braces and the limits defined by TBEC

GM Intensity Levels	Compression				Tension			
	Avg.	Max.	Min.	TBEC Limit	Avg.	Max.	Min.	TBEC Limit
DD-1	-15.83	-31.94	-3.34	$-7\Delta_c$ (CP)	8.06	16.55	1.50	$11\Delta_T$ (CP)
DD-2	-5.71	-12.66	-1.44	$-5\Delta_c$ (LS)	2.81	6.82	0.94	$8\Delta_T$ (LS)
DD-3	-1.75	-4.81	-0.39	$-0.5\Delta_c$ (IO)	0.91	2.28	0.28	$0.5\Delta_T$ (IO)

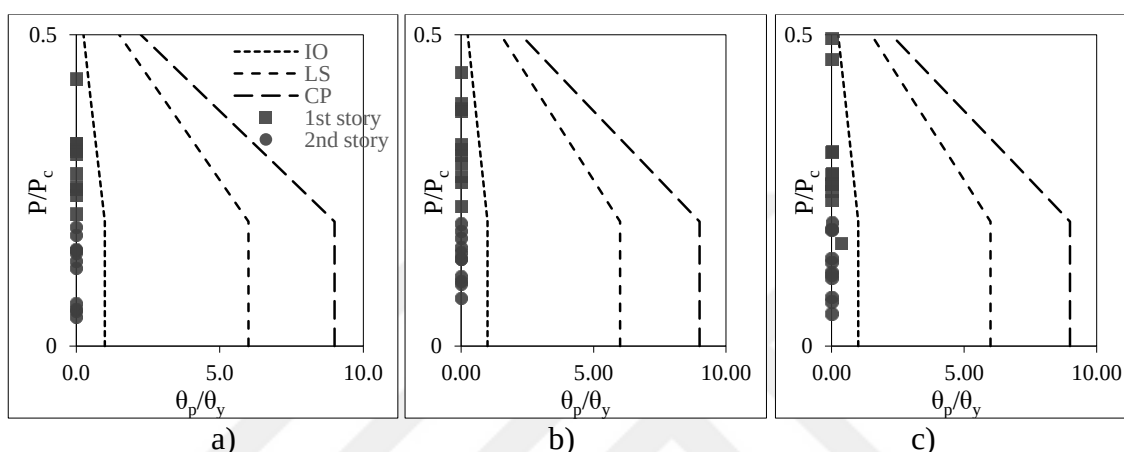


Figure 9. Column plastic rotations depending on a) DD-3, b) DD-2 and c) DD-1 levels.

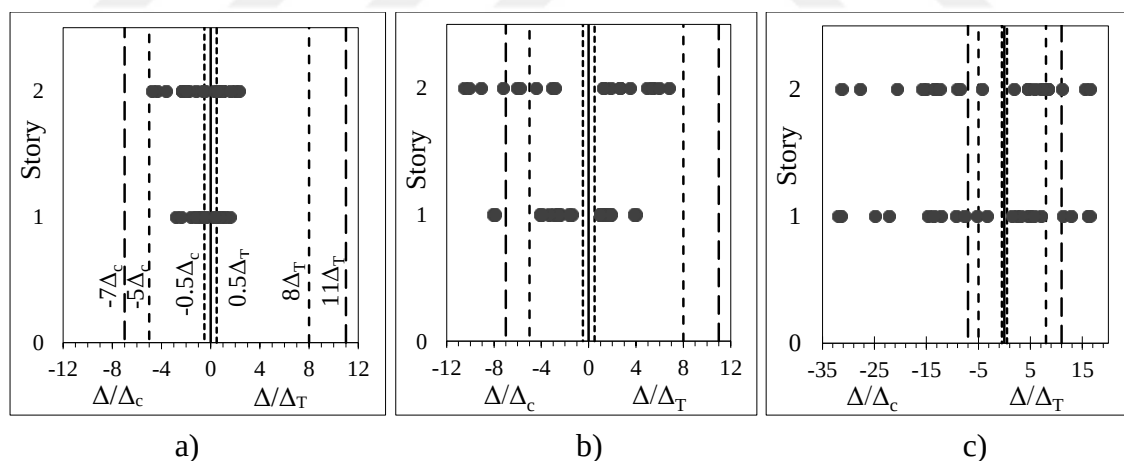


Figure 10. Brace axial deformations depending on a) DD-3, b) DD-2 and c) DD-1 levels.

Figure 10a, 10b and 10c show the normalized axial shortening and elongation of braces as well as deformation limits of TBEC which were indicated with the dashed

lines. For DD-3 level, all of the braces satisfied the LS or IO limits as presented in Figure 10a, and it can be deduced that general performance of braces were LS level since most of the ground motions (GMs) were above the IO limits. Figure 10b indicates the DD-2 level intensity where 4 GMs in 2nd story and 1 GM in 1st story exceeded the CP performance limits for compression while all other GMs were obtained within LS and CP limits. But the mean value of axial deformations for compression in 2nd story was calculated approximately as -7 which shows that braces were slight close to the collapse level since the CP limit is also defined as -7 for compression in TBEC. So, 1st story braces satisfied the LS limits for both compression and tension, but 2nd story braces were not able to meet the LS limit requirements for compression. Furthermore, since DD-1 level is the maximum considered earthquake and ground motions were scaled accordingly, brace axial deformations were excessively larger than other levels for both compression and tension as it is clear in Figure 10c. Significant amounts of elongation and shortenings were observed in braces for this level and most of the ground motions caused large axial deformations beyond CP region and the mean value of compressive deformations, which is provided in Table 2, exceeded the CP limits, too. In other words, a potential failure level was observed especially in compression for DD-1 level earthquake ground motion intensity.

Conclusion

In this study, it was intended to provide an insight into seismic performance of concentrically X-braced frames designed to the latest version of Turkish seismic codes, TBEC. Accordingly, an assessment was performed on a 2 story X-braced frame which

was designed in accordance with TBEC and TSSS, and nonlinear dynamic analysis were performed on the designed system. Analysis results were compared with the specified limits for different performance targets defined in TBEC. It was seen that beams remained elastic and columns were able to resist the seismic loads with negligible plastic deformations at floor levels, which did not lead to strength degradation. Braces were able to resist the lateral loads with limited inelastic deformations which were below the LS limits for DD-3 level. Although there were considerable compressive inelastic deformations observed especially in 2nd story braces for DD-2 level, the system slightly remained within the CP limits. However, DD-1 level scaled ground motions caused failure of the braces under compression and a potential collapse was observed. As a further work, it is planned to consider a wider range of X-braced frames having different geometrical properties such as number of stories, with an improved analytical model to increase the precision of the nonlinear analyses.

References

ATC, 1992, Guidelines for Cyclic Seismic Testing of Components of Steel Structures, ATC-24 Report, Applied Technology Council, Redwood City, California.

Disaster & Emergency Management Authority, AFAD (2018) Turkey earthquake risk interactive map. Ankara. Retrieved from <https://tdth.afad.gov.tr>, Accessed on 25 December 2019

Disaster & Emergency Management Authority, AFAD (2018) Turkish building earthquake code. Ankara: O.G. of T.R.

Fahjan, Y. M. (2008) “Selection and Scaling of Real Earthquake Accelerograms to Fit the Turkish Design Spectra”, *Teknik Dergi*, 19(3): 4423–4444.

Hsiao, P.-C., Lehman, D. E., & Roeder, C. W. (2013) “Evaluation of the response modification coefficient and collapse potential of special concentrically braced frames”, *Earthquake Engineering & Structural Dynamics*, 42(10): 1547–1564. DOI: 10.1002/eqe.2286

McKenna, F., Fenves, G.L., & Scott, M.H. (2016) Open system for earthquake engineering simulations, version 2.5.0. University of California, Berkeley. Retrieved from <http://opensees.berkeley.edu>

Ministry of Environment and Urban Planning. (2016) Turkish specification of steel structures. Ankara: O.G. of T.R.

Ministry of Public Works and Settlement. (2007) Specification for buildings to be built in seismic zones. Ankara: O.G. of T.R.

Pacific Earthquake Research Center, PEER (2020) PEER NGA West 2 ground motion database, Shallow Crustal Earthquakes in Active Tectonic Regimes. CA. Retrieved from <https://ngawest2.berkeley.edu>, Accessed on 31 January 2020

Palmer, K. D., Roeder, C. W., Lehman, D. E., Okazaki, T., Shield, C. K., & Powell, J. (2012) “Concentric X-braced frames with HSS bracing”, *International Journal of Steel Structures*, 12(3): 443–459. DOI: 10.1007/s13296-012-3012-8

Scott, M. H. (2011) “Numerical integration options for the force-based beam-column element in OpenSees”, *Force-Based Element Integration Options in OpenSees*, 1-7.

Tremblay, R., Archambault, M.-H., & Filiatrault, A. (2003) “Seismic Response of Concentrically Braced Steel Frames Made with Rectangular Hollow Bracing Members”, *Journal of Structural Engineering*, 129(12), 1626–1636. DOI: 10.1061/(asce)0733-9445(2003)129:12(1626)

Uriz, P., Filippou, F. C., & Mahin, S. A. (2008) “Model for Cyclic Inelastic Buckling of Steel Braces”, *Journal of Structural Engineering*, 134(4): 619–628. DOI: 10.1061/(asce)0733-9445(2008)134:4(619)

Uriz P., Mahin S. (2004) “Seismic performance assessment of concentrically braced steel frames”, *13th World Congress on Earthquake Engineering*.

APPENDIX B

**CONFERENCE PAPER: INCREMENTAL DYNAMIC
ANALYSIS OF CONCENTRIC X-BRACED FRAMES
DESIGNED PER TBEC 2018**

Conference Paper

Efecan Kor, Dr. Yiğit Özçelik

Presented and Published at the 9th European Conference on Steel and Composite
Structures, hosted in Sheffield, UK – Eurosteel 2021

<https://sites.google.com/sheffield.ac.uk/eurosteel2021>

Abstract

Concentrically braced frames (CBF) are an efficient lateral force resisting system which provides significant lateral stiffness and lateral strength. One of the commonly used configurations of CBFs are cross braced (X-braced) frames where braces are placed diagonally between the beam-column joints of adjacent stories. The current Turkish seismic code, Turkish Building Earthquake Code 2018 (TBEC), was published in 2018 and superseded the former code, Turkish Earthquake Code 2007 (TEC) as of January 2019 with several differences such as introduction of yield mechanisms for CBFs, seismic hazard levels, and calculation of earthquake loads. Due to these differences, CBFs designed to TBEC are expected to yield different member sizes compared to TEC. Although the seismic behaviour of CBFs designed to TEC is well-understood, that of CBFs designed to TBEC is still unknown and there has been no study on this topic. This study aims to assess the seismic performance of X-braced frames designed to TBEC. To this end, a two-story X-braced frame was considered, and the seismic collapse performance of this frame was evaluated by adopting incremental

dynamic analysis (IDA) under a set of twelve ground motions. The results of this preliminary study have provided insight into the collapse performance of CBFs designed to TBEC.

Keywords

Turkish Building Earthquake Code 2018, concentrically braced frames, X-braced frames, collapse performance, incremental dynamics analysis

Introduction

Concentrically braced frames (CBFs) are one of the most efficient type of lateral load resisting steel frames which provide a respectable lateral stiffness and strength to a structure under seismic loads. One of the most common type of CBFs is cross (X) braced frames where braces are connected to the beam column joints diagonally and form an X shape at each story. In CBF systems, braces are intended to be designed as fuse elements of the system to provide inelastic deformation demand during an earthquake and the required energy dissipation is provided by plastic behaviour of these elements. The lateral load resisting mechanism of X-braced configuration can be expressed as one of the braces works under axial tension while the counterpart works under axial compression. As a result of this load distribution mechanism, buckling and yielding behaviour are developed in compression and tension braces respectively. Accordingly, this cyclic plastic mechanism provides the intended energy dissipation in terms of buckling and yielding behaviour.

On the other hand, it is all important that global yield mechanism, inelastic behaviour and seismic performances of such systems shall be assessed and improved by means of elaborative investigations. Currently, there are many parameters involve in the seismic design approaches provided in several seismic codes or design guides. However, these parameters are examined in many studies performed by several researchers as it should be. For instance, Hsiao et al. (2013) [1] evaluates the response modification coefficient (R) and proposes an improvement for low-rise SCBFs. Furthermore, there are several experimental studies which are focused on seismic performance of CBFs in terms of capacity or deformations in order to provide better understanding on inelastic behaviour of such systems. For example, an experimental program Tremblay et al. (2003) [2] was conducted on CBFs with rectangular square HSS members which validated the usage of the effective length factor (K) in compressive strength calculations of bracing members and showed the buckling behaviour of X-braced frames. Likewise, BV Fell et al. (2009) [3] evaluated the effects of various parameters on seismic performance of bracing members by interpreting the outcomes of experiment and seismic provisions comparingly. Such investigations were carried out in order to understand and present the most real-like response of CBFs under lateral loads and develop more advanced levels of seismic design methodologies.

Major aim of the seismic design approach of CBFs is to maintain the structural stability in the event of buckling behaviour of braces under compressive loading due to a seismic loading, as specified in the Turkish Building Earthquake Code 2018 (TBEC 2018) [4] which is the latest version among Turkish seismic codes. This specification was come into force as of January 2019 and overridden the previous version, Turkish Earthquake Code 2007 (TEC 2007) [5], with several fundamental changes such as

introduction of yield mechanism for CBFs, definition of seismic hazard levels and calculation methodologies of earthquake loads. In TEC 2007, seismic zones were categorized into 5 levels for the seismic hazard map of Turkey and the design response spectrum was defined in terms of acceleration coefficient of related seismic zone, and corner period values. But, in order to create the design response spectrum, TBEC requires to obtain short and 1 second period spectral accelerations from the Turkish Seismic Hazard Map of Turkish Disaster & Emergency Management Authority (AFAD) [6] depending on the coordinates of a building. This revised map replaces the seismic zone concept with a peak ground acceleration scaling in order to represent the smaller variances in seismic hazard level. In addition, the response spectrum was defined for an earthquake with a return period of 475 years in the previous code while TBEC allows to create separate spectrums for earthquakes with 4 different return periods (2475, 475, 72 and 43 years). Besides, local soil class was only involved in the corner period calculations in TEC, however, it also affects the amplitude of the design spectral accelerations in TBEC. Similarly, the definition of response modification coefficient (R), was changed in consequence of the transition process to the TBEC. The variety of R factor was specified for a wider range of lateral load resisting systems. Figure 1 represents the difference between response spectrums of TBEC and TEC for the same location.

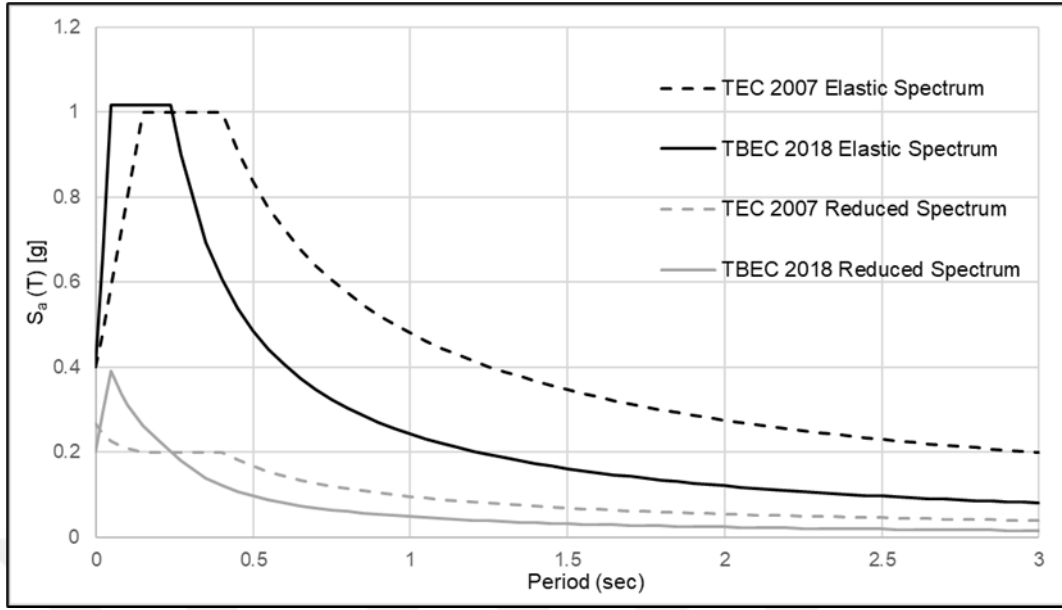


Figure 1 Comparison of response spectrums per TBEC and TEC

The seismic design method that emphasized in TBEC 2018 necessitates to use the expected yield stress, $R_y F_y$, which is the product of material yield stress, F_y , and expected material strength factor, R_y , instead of characteristic yield strength, F_y , for the design of beam and column elements. Moreover, it is also required to use expected critical buckling strength of braces, $1.14 F_{cre} A_g$, for the frame design, where F_{cre} is the expected critical buckling stress which is explained in Turkish Specification of Steel Structures 2016 (TSSS) [7] and A_g is cross sectional area of the compression member. As illustrated in Figure 2, there are 2 yield mechanisms prescribed in TBEC that should be considered in the seismic design of CBFs. Mechanism 1 indicates the expected response of the system before the buckling of compression brace, while Mechanism 2 represents the post-buckling behaviour of the compression brace with a decrease to 30% of the expected compressive strength. Seismic design of such systems must be made by taking above mentioned conditions into consideration in order to prevent large plastic

deformations in beams and especially columns which might lead to a critical global instability.

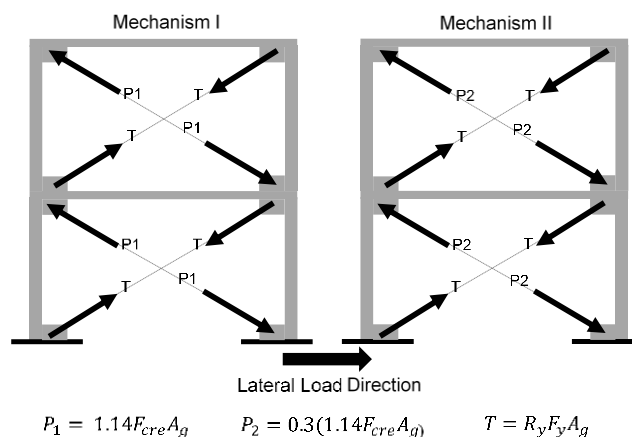


Figure 2 Yield mechanisms defined in TBEC

Consequently, when the above-mentioned changes between two codes considered, it can be deduced that CBFs designed to TBEC are expected to be different. However, there is no such study concentrated on seismic performance of X-braced frames designed to new Turkish seismic code came into effect, TBEC, in literature. In this study, it is aimed to understand the seismic behaviour of CBFs designed per TBEC. In order to achieve that, a 2-story X-braced frame was designed to TBEC and TSSS, and the collapse performance of the structure was assessed by performing Incremental Dynamic Analysis (IDA) procedure. This effective method provides a better understanding of structural responses under severe ground motions and enables to estimate the dynamic behaviour of a structure. The detailed explanation and application of IDA approach is discussed in Vamvatsikos and Cornell (2002) [8]. Implementation fundamentals of this method can be explained as a set of scaled ground motions are applied to a structure recursively and the scale factor is increased by small increments at each step until the system exceeds the collapse limit which are explicable in terms of

several conditions such as interstory drift, a significant stiffness degradation or ductility etc. Selected ground motion records taken from PEER NGA West-2 [9] database are shown in Table 1. These records were chosen to be coherent with several conditions in terms of fault type, expected magnitude or local soil class which were obtained as strike slip fault type, 7.0 to 7.6 magnitude and ZB class respectively.

Table 1 Selected ground motion records

GM¹	RSN²	Event	Station	Magn.
1	282	Trinidad	Rio Dell Overpass	7.2
2	832	Landers	Amboy	7.28
3	864	Landers	Joshua Tree	7.28
4	881	Landers	Morongo Valley FS ³	7.28
5	1144	Gulf of Aqaba	Eilat	7.2
6	1176	Kocaeli	Yarimca	7.51
7	1605	Duzce	Duzce	7.14
8	1615	Duzce	Lamont 1062	7.14
9	1627	Caldiran	Maku	7.21
10	1634	Manjil	Abhar	7.37
11	1762	Hector Mine	Amboy	7.13
12	1770	Hector Mine	Big Bear Lake - FS	7.13

¹Ground Motion, ²Record Sequence Number on PEER Database,
³Fire Station

Structural Design of X-braced Frame

General Properties of the Building

Structural design of the 2 story X-braced steel structure located in Maltepe, Istanbul, was carried out in accordance with the methods and limitations provided in TBEC and TSSS. The building is intended to serve as residential building, so the building occupancy category (BKS), importance factor (I) and live load mass participation factor (n) were taken as 3, 1 and 0.3 respectively from TBEC. These values were used in the lateral load calculations afterwards. The building has a typical floor plan which consists of 5 bays in each direction with 6- and 8-meters bay width in EW and NS respectively. 4 X-braced frames were placed along the longer direction to resist the lateral loadings. General floor plan and elevation view of the building is provided in Figure 3. Connections between each element except column joints of adjacent stories were taken as pin connection.

Gravity and Seismic Design

Gravity loads of the building were assigned as 5 kPa dead and 2 kPa snow, roof live and story live loads. These loads were applied to the system by using tributary area method. In addition to the gravity loads, lateral loads were applied to the structure by adopting the equivalent lateral force procedure (ELF) of TBEC. Several parameters are required to calculate the lateral forces from this method. Short and 1 second period design spectral accelerations (S_{DS} and S_{D1}) were obtained as 1.017 and 0.243 g respectively, from Turkish Seismic Hazard Map of AFAD for ZB type (medium strength rocks with $(V_s)_{30}$ value between 760 – 1500 m/s) soil class and DD-2 level

ground motion intensity which can be clarified as the ground motion level with a 10% probability of exceedance in 50 years (i.e. return period of 475 years). To calculate the design spectral accelerations, mapped spectral accelerations were multiplied by local soil effect factors (F_s and F_1) which are proportional with local soil class. Eventually, the design response spectrum was created as a function of spectral acceleration (S_{ae}) and period (T), where $S_{ae}(T)$ depends on S_{DS} , S_{D1} , long and corner periods. However, the S_a value required for the design, S_{aR} (reduced spectral acceleration), was calculated as a function of building importance factor (I) and response modification factor (R) which is defined as 5 for the structures that resist the lateral loads with high ductility CBFs, in the related table of TBEC. By using above mentioned mass participation factor, the total seismic mass of the structure was found as 1370.5 tons and the first mode period of the structure was calculated as 0.22 seconds from the approximate method defined in TBEC. Also, total base shear acts on the system was calculated with these parameters by adopting equivalent lateral load procedure (ELF) of TBEC. This value was produced as 2330 kN and distributed to each floor level in accordance with the same procedure. Meanwhile, accidental torsion effects were included in the analysis considering a 5% eccentricity for the earthquake forces. S355 grade steel material was used for beams and columns while S235 was used for braces to provide higher ductility.

Table 2 Designed sections

Story No	Beam	Column	Brace
1	W 250x115	W 200x86	HSS 220x220x5
2	W 250x89	W 200x86	HSS 180x180x4

By adopting the approximate second order analysis procedure of TSSS and approximate period calculation methods of TBEC, required strengths and sizes of each member were estimated as a first step. With the assigned sections, first mode period of the structure was recalculated as 0.27 sec by implementing eigenvalue analysis and designed members were checked whether a recalculation is necessary or not, since the period values calculated from the approximate methods were different than the results of eigenvalue analysis. This iterative procedure was repeated until two period values converged to a tolerable difference which does not have an effect neither on loading values nor designed section sizes. Final design of each member is shown in Table 2.

Numerical Model

OpenSees Model

Numerical modelling and nonlinear dynamic analyses of the predesigned structure were performed in OpenSees [10], which is an open-source software framework system for simulating the structural systems subjected to different loading conditions such as lateral static loadings or ground accelerations. In order to simplify the structural model and reduce the complicity of analyses, a two-dimensional single braced bay was considered to represent the global system since the floor plan of the building is not a complex layout. This frame is shown in detail in Figure 4.

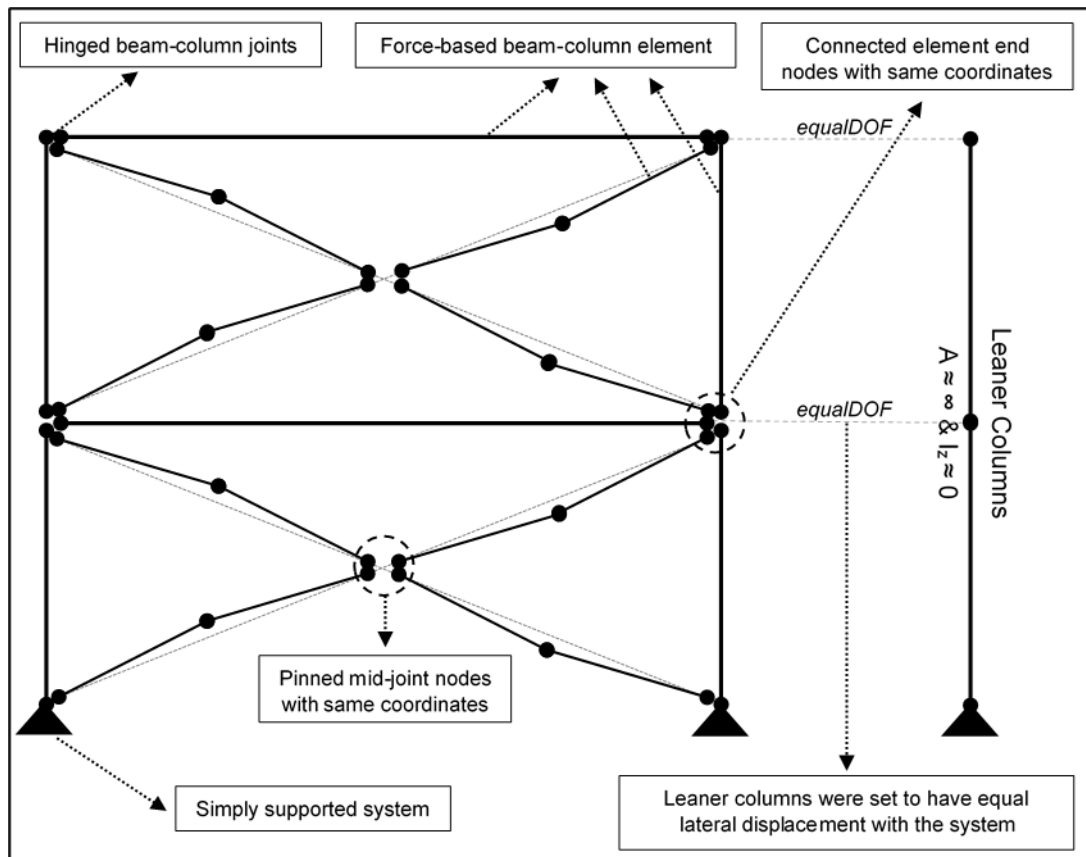


Figure 4 Illustration of the X-braced frame model in OpenSees

Fiber sections consist of quadrilateral patch were assigned for all elements with in-built Steel 02 material which is defined as uniaxial Giuffre-Menegotto-Pinto steel with isotropic strain hardening. The aim of assigning fiber sections is to simulate the distribution of material nonlinearity throughout the cross section of an element and P-M interaction. A strain hardening ratio of 0.3% was assumed for all sections. Also, 5 integration points were assigned along each element based on Gauss-Lobatto quadrature rule. In addition to that, geometrical nonlinearities were also taken into consideration by using the in-built command of OpenSees platform, corotational transformation, whose major tasks are to transform forces and stiffnesses to global coordinates and arrange the transformation between displacements and nodal coordinates as explained by Denavit and Hajjar (2013) [11]. All members except leaner columns were modelled using force-

based beam-column elements. Post buckling behaviour of braces was simulated with an initial imperfection which is applied by dividing the brace elements into 4 pieces and 0.1% camber were assigned at mid lengths of each piece as shown in Figure 5. Applicability and reliability of such implementation is discussed in Uriz et al. (2008) [12].

Besides, leaner columns were introduced to the frame in order to contribute the gravity load related P-Δ effects on the system as shown in Figure 4. To achieve that, top nodes of leaner columns were connected as slave nodes to each story at the same height and set to have equal displacement along 1st degree of freedom (i.e. lateral displacement). Leaner columns were modelled with elastic beam-column element having large cross-sectional area and small inertia in order to eliminate the effect of axial deformation and participation to general lateral stiffness respectively. Those elements were loaded with the quarter of gravity loads acting on each story. On the other hand, mass and stiffness proportional Rayleigh damping was assigned to the system with a damping ratio of 2%.

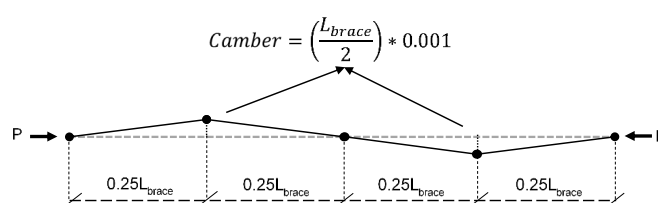


Figure 5 Application of initial imperfection on brace modelling

Model validation

In order to determine the accuracy of the numerical model prescribed before, outcomes of an experimental study on performance of concentric X-braced frames

conducted by Palmer et al. (2012) [13] were taken to compare the results obtained from nonlinear static pushover analysis of the model. The most suitable specimen among several types was chosen which is a single bay one story planar frame with HSS braces having slotted tube knife-plate connection at mid joint. In the OpenSees model, measured properties of materials were used such as yield strength. Through experiment, the specimen was subjected to a cyclic displacement loading protocol at the top of the frame. Base shear value and story drift ratio of the X-braced frame were measured during the cyclic loading process.

Since it was desired to model the system to accurately represent the structural behaviour, several boundary conditions of members were considered to compare the response of the system:

- Pinned connections at brace ends – Pinned column base,
- Pinned connections at brace ends – Fixed column base,
- Fixed connections at brace ends – Fixed column base.

Purpose of changing the column base connection type is columns of the specimen built for the experiment is connected to a beam element at the bottom ends. In other words, bottom end connections of columns were expected to respond between a fixed and pinned support behaviour. In order to better understand such response of the system, both conditions were taken into consideration. The same circumstance is also valid for brace end connections. These conditions were integrated in the parametric model which was then subjected to the cyclic displacement procedure specified in the experiment.

Analysis results showed that response of the system get closer when pin connections were assigned to brace ends compared to fixed end connections. Besides, fixed column base assumption produced more accurate results in comparison to pinned column base. Actual behaviour of the connections lies between pin and fix type as such can be concluded from the difference between the results obtained from both pinned and fixed end cases. It was observed that when pinned connections are used in the model, the system became more flexible but analysis results were found to be close substantially. Another reason of the difference is members were modelled center to center which increased the flexibility of the system. As a general outcome, pinned connections were found to be closer to the experimental data in comparison to fixed connections. For instance, this variance can be expressed in terms of differences between maximum base shear values of analytical and experimental results which are obtained as -5% and +16% for pinned and fixed end conditions respectively. Although base shear values read from the curves developed from analysis results and experimental data are not identical, the model estimates quite similar values and simulates the general behaviour of the system as shown in Figure 6.

Consequently, pinned connections were used in the numerical model for this study.

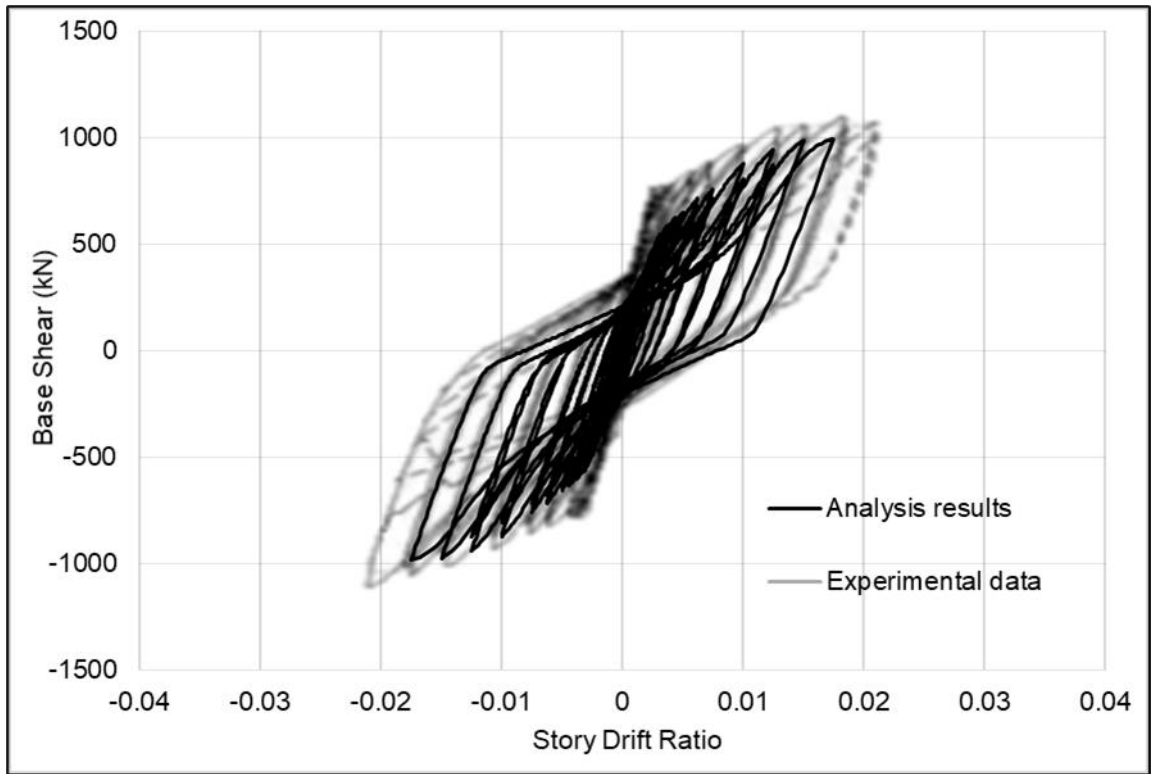


Figure 6 Comparison of the analytical and experimental results

Incremental Dynamic Analysis

Ground Motion Scaling

In this study, X-braced frames were subjected to a set of ground motions taken from PEER NGA Database (shown in Table 1 and Figure 7), which were used in order to perform incremental dynamic analysis and evaluate the seismic performance of the designed CBF. Ground motions were scaled up or down with separate factors which were calculated by using the response spectrums of each ground motion. First mode period of the system and corresponding spectral acceleration value on the response spectrum were taken as the target intersection point of response spectrum curves of each ground motion. As a result, scale factors were obtained as the coefficient required to shift response spectrum curves to intersect the predefined target point. Above

mentioned parameters were integrated into the model to perform nonlinear dynamic analysis in OpenSees platform.

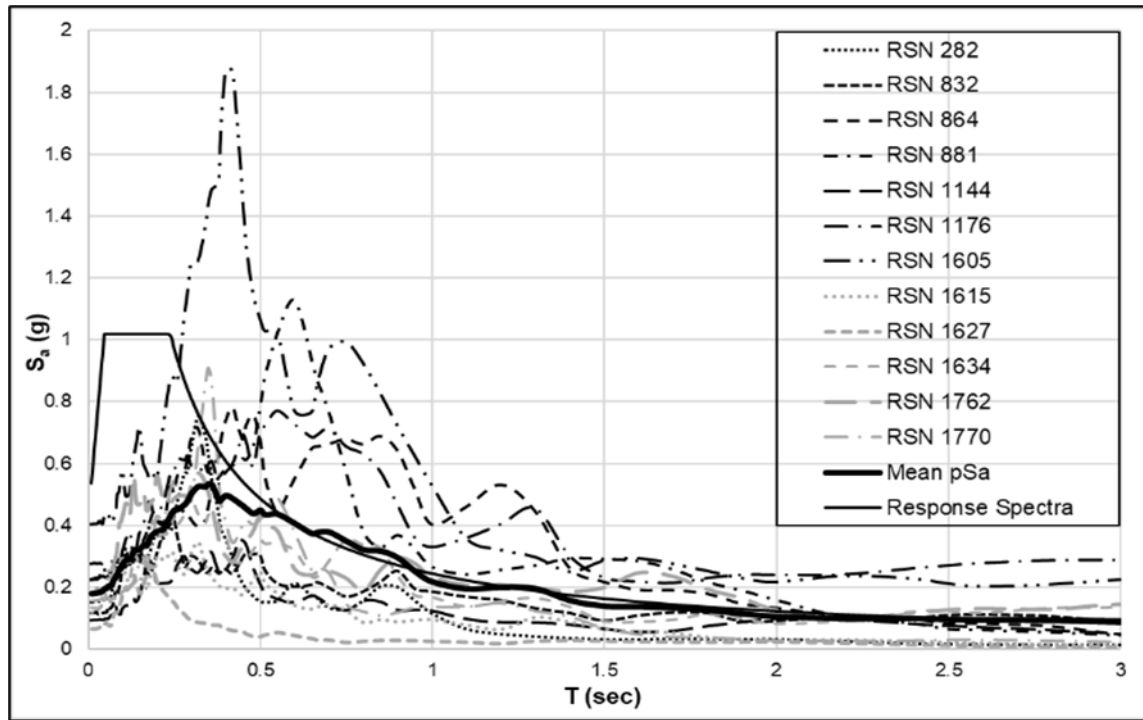


Figure 7 Response spectrum of each ground motion record

IDA Curve & Limit States

Using the results of nonlinear dynamic analysis, IDA curves were produced as a function of scalable ground motion intensity measure (IM) and damage measure (DM) which can be defined with several parameters. For instance, spectral acceleration (S_a) or peak ground acceleration (PGA) values can be selected for IM while maximum interstory drift ratio (θ), peak story ductility (μ) or maximum base shear can be chosen as DM. As a more general definition, IM is the function of a scaled accelerogram and DM is the function of structural response to measure the effects of IM. The multi-record IDA curve of the X-braced frame is provided in Figure 8. Furthermore, there might be more than one points on an IDA curve which exceed one of the limit states in terms of

DM as well as IM. This phenomenon is defined as weaving behaviour which might mislead the assessment and should be taken into consideration. Explanation for this issue is the structural recovery (i.e. hardening) at relatively higher IM levels and these regions should not be considered as possible collapse points. An example of this case is shown in Figure 9.

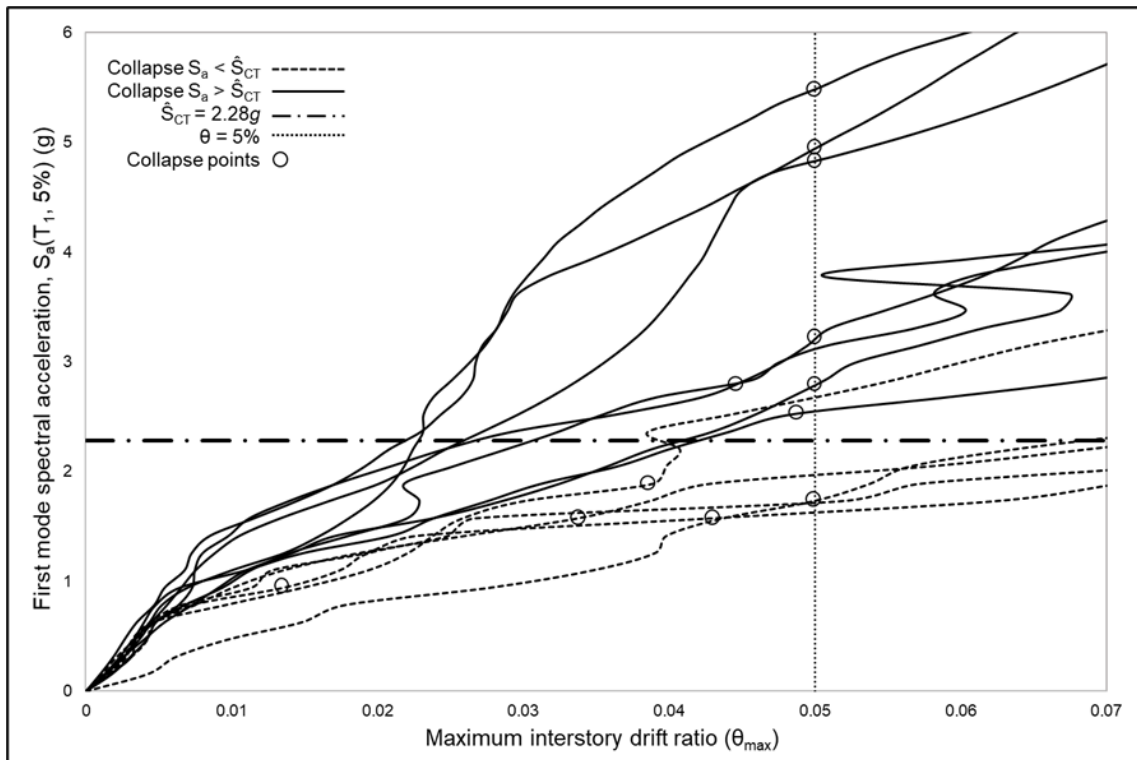


Figure 8 Multi-record IDA curve

For this study, non-simulated collapse criteria were taken as not only if the maximum interstory drift of 5% is exceeded but also if the tangent between adjacent points on IDA curve of a ground motion reduces below 20% of the initial slope, and it was accepted that collapse limit state have been achieved according to the DM and IM rules respectively. Reason behind the selection of these values is FEMA350 [14] suggests a tangent slope rule which is defined as if the straight-line slope between consecutive points is decreases under 20% of the elastic slope or the story drift value

exceeds 10%. However, the experimental study on CBFs by Lehman and Roeder (2008) [15], and NEHRP guide [16] indicates that structural elements including connections may not provide the desired resistance beyond 5% story drift so this value was chosen as potential collapse limit based on DM rule. Through the analysis procedure, each ground motion was applied to the system step by step with incrementally increasing scale factor and damage states were checked in each step whether the structure exceeds either one of the prescribed limit conditions. This iterative procedure was repeated with increasing scale factor until one of the collapse limits has been observed from IDA curves for each ground motion.

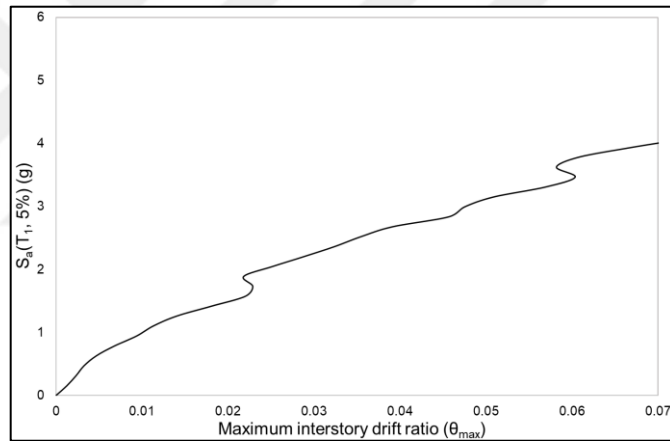


Figure 9 Weaving on an IDA curve

Collapse Fragility Curve

By evaluating the outcomes of IDA, a collapse fragility curve as shown in Figure 10, was developed as a function of ground motion intensity and collapse probability using a cumulative distribution function. The dispersion value (β) of the lognormal distribution was taken as 0.6. The curve was plotted by means of FEMA P-58-3 [17] tools. Afterwards, by adopting the methodology described in FEMA P-695 [18] guide, the median collapse capacity intensity, $\hat{S}_{CT}$, which is the spectral

acceleration value that corresponds 50% probability of collapse on the fragility curve was obtained as 2.28g. In other words, this point indicates the spectral intensity in which half of the ground motions lead the structure to go through collapse state. The fitted curve and median collapse capacity intensity value are shown in Figure 10. Afterwards, a collapse margin ratio (CMR) of 1.37, was calculated as the ratio of $\hat{S}_{CT}$ to S_{MT} , where S_{MT} is 1.67g and defined as corresponding spectral acceleration value of maximum considered earthquake.

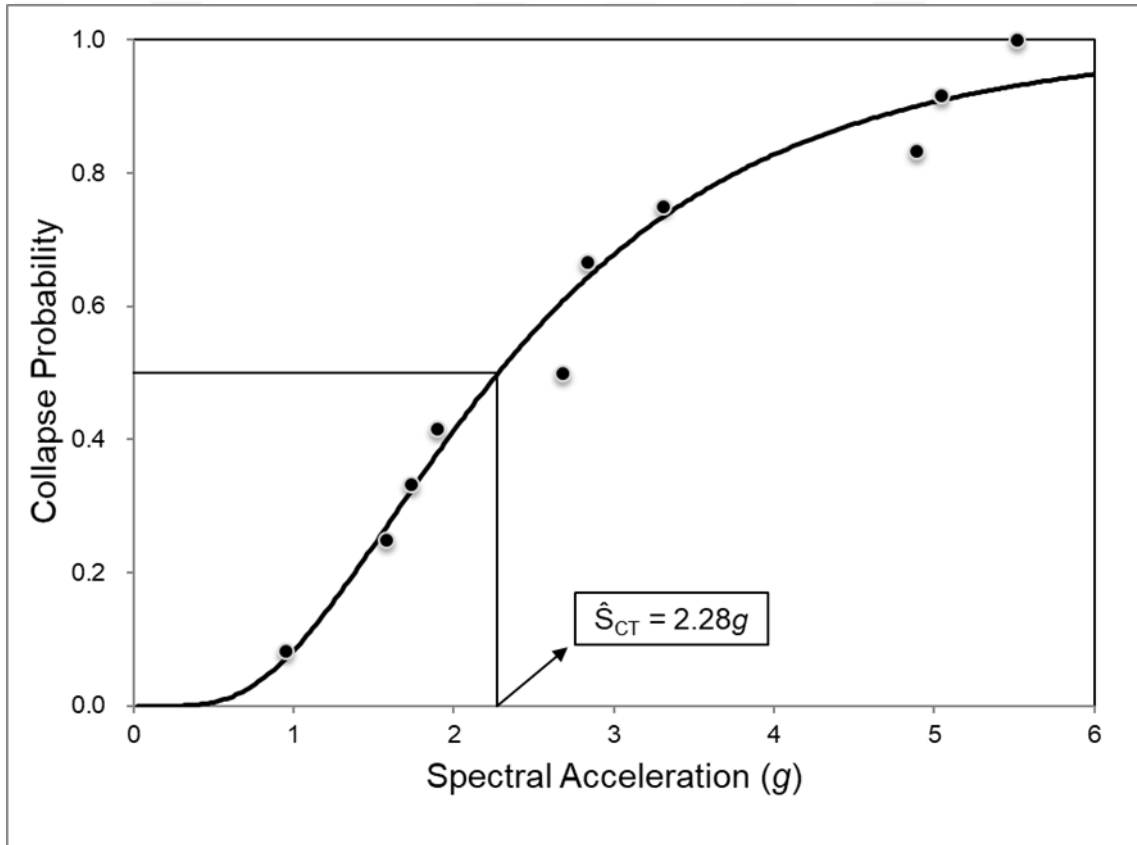


Figure 10 Collapse fragility curve

Conclusion

This study aimed to understand and present the seismic behaviour of X-braced structures designed per the latest Turkish seismic code, TBEC, which came into effect in January 2019 and superseded the previous version, TEC 2007. In order to accomplish

this objective, a 2 story X-braced structure was designed to TSSS and TBEC, and a numerical model was created in OpenSees platform. A set of ground motions were selected with specific seismic parameters considered to represent realistic conditions. By using the selected ground motion records, incremental dynamic analysis was performed on the predesigned X-braced frame model and the capacity points were determined according to 2 different collapse limit states (IM and DM rules) on the IDA curve. In conclusion, collapse fragility curve was created with the capacity points observed from IDA and as a result, the median collapse capacity was obtained from the IDA curve. After all, the collapse margin ratio for the X-braced frame was calculated. The CMR of 1.37 revealed that the frame considered that was designed to TBEC has showed satisfactory performance in terms of collapse. Note that this manuscript only outlines the initial findings of an ongoing research effort. The authors have been undertaking a parametric study to evaluate the seismic collapse behaviour of CBFs designed to TBEC considering various parameters such as number of stories, seismicity of the site, and earthquake load to vertical load ratio. By the end of this ongoing research, it is expected to quantify the seismic collapse performance of CBFs designed to TBEC and propose recommendations/modifications to TBEC.

References

Hsiao, P.-C., Lehman, D.E. & Roeder, C.W. (2013) *Evaluation of the response modification coefficient and collapse potential of special concentrically braced frames*. Earthquake Engineering & Structural Dynamics, 42: 1547-1564. DOI:[10.1002/eqe.2286](https://doi.org/10.1002/eqe.2286)

Tremblay, R., Archambault, M.-H., & Filiatrault, A. (2003) *Seismic Response of Concentrically Braced Steel Frames Made with Rectangular Hollow Bracing Members*. Journal of Structural Engineering, 129(12), 1626–1636. DOI: 10.1061/(ASCE)0733-9445(2003)129:12(1626)

Fell, B. V., Kanvinde, A. M., Deierlein, G. G., & Myers, A. T. (2009) *Experimental Investigation of Inelastic Cyclic Buckling and Fracture of Steel Braces*. Journal of Structural Engineering, 135(1), 19–32. DOI: 10.1061/(ASCE)0733-9445(2009)135:1(19)

Disaster & Emergency Management Authority, AFAD. (2018) *Turkish building earthquake code*. Ankara: O.G. of T.R.

Ministry of Public Works and Settlement. (2007) *Specification for buildings to be built in seismic zones*. Ankara: O.G. of T.R.

Disaster & Emergency Management Authority, AFAD (2018) *Turkey earthquake risk interactive map*. Ankara. Retrieved from <https://tdth.afad.gov.tr>, Accessed on 03 December 2019

Ministry of Environment and Urban Planning. (2016) *Turkish specification of steel structures*. Ankara: O.G. of T.R.

Vamvatsikos, D., & Cornell, C. A. (2002) *Incremental dynamic analysis*. Earthquake Engineering & Structural Dynamics, 31(3), 491–514. DOI: 10.1002/eqe.141

Pacific Earthquake Research Center, PEER (2020) *PEER NGA West 2 ground motion database, Shallow Crustal Earthquakes in Active Tectonic Regimes*. CA. Retrieved from <https://ngawest2.berkeley.edu>, Accessed on 31 December 2019

McKenna, F., Fenves, G.L., & Scott, M.H. (2016) *Open system for earthquake engineering simulations, version 2.5.0*. University of California of California, Berkeley. Retrieved from <http://opensees.berkeley.edu>

Denavit, M. D., & Hajjar, J. F. (2013) *Description of geometric nonlinearity for beam-column analysis in OpenSees*. Department of Civil and Environmental Engineering Reports. Report No. NEU-CEE-2013-02. Department of Civil and Environmental Engineering, Northeastern University, Boston, Massachusetts.

Uriz, P., Filippou, F. C., & Mahin, S. A. (2008) *Model for Cyclic Inelastic Buckling of Steel Braces*. Journal of Structural Engineering, 134(4), 619–628. DOI: 10.1061/(ASCE)0733-9445(2008)134:4(619)

Palmer, K. D., Roeder, C. W., Lehman, D. E., Okazaki, T., Shield, C. K., & Powell, J. (2012) *Concentric X-braced frames with HSS bracing*. International Journal of Steel Structures, 12(3), 443–459. DOI: 10.1007/s13296-012-3012-8

Federal Emergency Management Agency. (2000) *Seismic design criteria for steel moment-frame buildings: FEMA 350*. Washington, D.C.

Lehman, D., & Roeder, C. W. (2008). *Improved Seismic Design of Concentrically Braced Frames and Gusset Plate Connections*. Structures Congress 2008. DOI:10.1061/41016(314)258

Sabelli, R., Roeder, C. W., & Hajjar, J. F. (2013) *Seismic design of steel special concentrically braced frame systems: A guide for practicing engineers*. NEHRP Seismic Design Technical Brief No. 8, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 13-917-24.

U.S. Department of Homeland Security, FEMA. (2009) *FEMA P-695: Quantification of building seismic performance factors*. Washington, D.C.

U.S. Department of Homeland Security, FEMA. (2018) *FEMA P-58-3: Seismic performance assessment of buildings | Volume 3: Supporting electronic materials and background documentation* (2018th ed., Vol. 3). Washington, D.C. Retrieved from <https://www.fema.gov/assets/documents>, Accessed on 14 February 2020

APPENDIX C

**JOURNAL PAPER: SEISMIC PERFORMANCE
ASSESSMENT OF CONCENTRICALLY BRACED
FRAME SYSTEMS DESIGNED PER THE TURKISH
BUILDING EARTHQUAKE CODE 2018**

Journal Paper

Efecan Kor, Dr. Yiğit Özçelik

Submitted to the Journal “*Structures*” in December 2021

ABSTRACT

Concentrically braced frames (CBFs) are widely used steel framing systems that employ remarkable lateral stiffness to a structure. Their elastic stiffness avoids large deformations to be developed within the structure which might affect serviceability. Also, the post buckling behavior of the braces provides the ductility during a major earthquake event. However, to avoid a potential instability due to inelastic deformations, the braced frame system must be designed and detailed properly. To that end, seismic design codes including the Turkish Building Earthquake Code 2018 (TBEC 2018) employ various design approaches and requirements. TBEC 2018 superseded the previous version, the Turkish Earthquake Code 2007 (TEC 2007) with several new design approaches as well as various changes in the previous applications, which can be expected to affect the seismic performance of CBF designs. Therefore, in this study, it is aimed to carry out a seismic performance assessment on the CBFs designed per TBEC 2018. A set of archetypes was designed to reflect different applicable building heights, aspect ratios and seismic hazard levels. A nonlinear

numerical model was developed on OpenSEES, to simulate an accurate behavior of the structural response and to perform nonlinear static and dynamic analyses. An assessment method that is compliant with FEMA P-695 was followed for the performance evaluation. Effects of different building heights, aspect ratios and seismic hazard levels are also investigated and reported in the study. In general, the results revealed that the seismic performance of CBFs designed per TBEC 2018 is satisfactory for moderate seismicity under maximum considered earthquake level, however, it was found inadequate for high seismicity.

Keywords: Concentrically braced frames (CBF), seismic design, performance assessment, incremental dynamic analysis (IDA), FEMA P-695, TBEC 2018

INTRODUCTION and BACKGROUND

Concentrically braced frames (CBFs) are one of the most efficient lateral load-resisting systems in which centerlines of members meet at a joint intersect at a point to resist lateral loads as a truss system. The members of CBFs are subjected to axial loads primarily; consequently, CBFs are associated with significant lateral stiffness and strength. The braces of CBFs are subjected to large deformations in tension and compression during strong ground shaking. Braces typically buckle or yield at moderate story drifts of 0.3% to 0.5%. Research [1,2] revealed that, with proper detailing such as limiting width-to-thickness ratios of braces and special design of end connections, the post-buckling behavior of CBFs can be improved significantly, braces can sustain inelastic axial deformations 10 to 20 times their yield deformation, and a high level of ductility can be achieved.

The specification for structural steel buildings and seismic design provisions in Turkey have recently undergone significant modifications. TS 648 Building Code for Steel Structures [3] was the code used in Turkey for the design of steel structures between 1980 and 2016. The code was based on the US and German codes of 1970s and adopted the allowable stress design philosophy. In 2016, TS 648 was superseded by the Specification for the Design, Calculation, and Construction of Steel Structures (SDCCSS) [4] that can be considered a Turkish adaptation of the AISC 360-10 Specification for Structural Steel Buildings [5] with minor differences. Similarly, the Turkish Building Earthquake Code 2018 (TBEC 2018) [6] came into force in 2019 and replaced the previous seismic provisions, the Turkish Earthquake Code 2007 (TEC 2007) [7]. While maintaining some sections of TEC 2007 as is, TBEC 2018 includes excerpts directly (or in a slightly modified fashion) adopted from ASCE 7-16 Minimum Design Loads for Buildings and Other Structures [8] and AISC 341-16 Seismic Provisions for Structural Steel Buildings [9] as well as new set of rules that had not been codified before. The most notable differences between TBEC 2018 and TEC 2007 can be listed as follow:

Similar to AISC 341-16, TBEC 2018 mandates explicit consideration of the inelastic behavior of the braces of CBFs for the design of all structural elements but braces through use of plastic-mechanism analyses that include (1) an analysis in which all braces have reached their maximum forces either in compression or tension and (2) an analysis in which tension braces are at their expected strength level while compression braces have buckled and retain only a small percentage of their buckling strength. On the other hand, TEC 2007 requires an elastic analysis under horizontal seismic forces including overstrength for the design of structural elements except for braces.

TBEC 2018 adopts the approach given in ASCE 7-16 in which both the ordinates and the corner periods (T_A and T_B in Figure 1(a)) of the elastic design response spectrum are functions of design-basis earthquake (DBE) spectral response acceleration parameters at short periods, S_{DS} , and at 1-s periods, S_{D1} , that are adjusted for site class effects. On the other hand, the ordinates of the elastic design response spectrum in TEC 2007 are solely based on peak ground acceleration coefficient (A_o) and require no adjustment for site class effects, and the corner periods of the spectrum are predefined values set per site class.

TEC 2007 uses a parameter called earthquake load reduction factor ($R_a(T)$), a function of the response modification factor (R-factor), the corner periods of the design response spectrum, and the fundamental period of the structure, T , to reduce the elastic design spectrum (Figure 1(b)). TBEC 2018 uses the same parameter for seismic force reduction; however, the equation given for $R_a(T)$ has been modified (Figure 1(c)).

The seismic hazard maps of Turkey have been recently updated and TBEC 2018 deploys these maps [10] that typically generates higher spectral accelerations compared to the older maps.

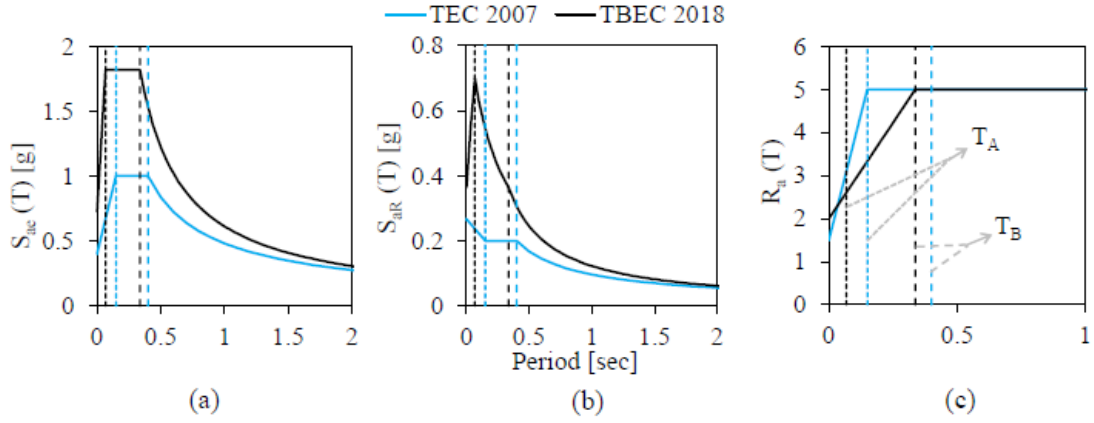


Figure 1. Comparison of TBEC 2018 and TEC 2007 in terms of (a) elastic response spectrum, (b) reduced response spectrum, and (c) earthquake load reduction factor (The spectra are given for Sarkoy, Turkey assuming a site soil class of ZC)

The aforementioned differences between the former and current seismic provisions as well as employing a new specification for structural steel buildings are expected to have a significant impact on the seismic forces and the seismic design of lateral force resisting systems as well as their seismic performance. Except for a few studies on the seismic performance of highly-ductile CBFs [11–14] (equivalent to special CBFs in AISC 341-16), research on highly-ductile CFBs is quite limited and, to the authors' knowledge, there is no study in the literature on the seismic behavior and collapse performance of highly-ductile CBFs designed to new Turkish codes.

The aim of this study is to evaluate the collapse performance of highly-ductile CBFs designed to the new Turkish specifications for steel structures and seismic provisions, namely, TBEC 2018 and SDCCSS. The seismic performance evaluation is performed by employing the Methodology given in FEMA P-695 [15]. The application of the Methodology and the seismic performance of highly-ductile CBFs are presented herein.

OVERVIEW OF THE FEMA P-695 METHODOLOGY

The FEMA P-695 Methodology is a framework that requires nonlinear static and nonlinear response-history analysis to evaluate the safety of lateral force-resisting systems against collapse. Although the Methodology primarily aims at the quantification of seismic performance factors for new structural systems, it can be used for the evaluation of the seismic performance of existing systems as well. The first step of the Methodology is to select and design several archetypes with variations in design parameters such as number of stories, bay width, and earthquake load level. The index archetypes are assembled into performance groups that reflect the design parameters. The next step is to construct the nonlinear models of the archetypes such that all possible failure modes that might lead to strength or stiffness degradation need to be included in the models to the extent possible. For the limit states that cannot be simulated in nonlinear models, non-simulated collapse criteria can be implemented, i.e., such limit states can be checked on structural response parameters such as interstory drifts and axial strain of braces. In the analysis step, nonlinear static analysis is performed to determine the overstrength factor (Ω_o in ASCE 7-16 and D in TBEC 2018) and period-based ductility, μ_T , of each archetype. The Methodology includes a set of 22 far-field ground motion pairs. In case of two-dimensional analysis, an incremental dynamic analysis is conducted for each archetype using all 44 ground motion records. The ground motions are scaled successively until the median collapse capacity is determined ($\hat{S}_{CT}$) for each index archetype. $\hat{S}_{CT}$ is defined as the ground motion intensity at which 22 of 44 ground motion records cause collapse of the archetype. The ratio of $\hat{S}_{CT}$ to the ground motion intensity at the Maximum Considered Earthquake level, S_{MT} , is called the collapse margin ratio (CMR). CMR is multiplied by a parameter called the spectral shape factor, SSF, to obtain an adjusted collapse margin ratio (ACMR). The

seismic performance is deemed acceptable if (1) $ACMR$ of each index archetype is greater than the target $ACMR$ for a collapse probability of 20% ($ACMR_{20\%}$) and (2) the average value of $ACMR$ within a performance group is greater than the target $ACMR$ for a collapse probability of 10% ($ACMR_{10\%}$). The target $ACMR$ s that hinge on the total system collapse uncertainty (β_{TOT}) are tabulated in FEMA P-695 for several collapse probabilities. β_{TOT} depends on different sources of uncertainty stemming from design requirements, quality of test data, nonlinear modeling, and record-to-record variability.

DESIGN AND SELECTION OF ARCHETYPES

The Methodology requires structural design of a wide range of archetypes that cover characteristics affecting the seismic performance. In this study, bay width (L) and the number of stories (N) were considered the primary parameters, as well as the seismic design category (SDC), in defining archetypes. Two square floor plans with a side dimension of 30.0 m were adopted (Figure 2). In the first floor plan (Figure 2(a)), there are two bays with CBFs with a L of 7.5 m on both perimeter frames in the W-E direction which are indicated as BF1 while four bays with CBFs with a L of 5.0 m in the N-S direction which are labeled as BF2. The second floor plan is almost identical to the first floor plan. The only difference is that the CBFs in the N-S direction have a L of 3.5 m which are denoted as BF3. All BF-1, BF-2, and BF-3 type frames were designed in this study. Although there are several types of CBFs such as diagonally braced, V-braced, and inverted V-braced, only X-braced CBFs were considered in the designs. To consider structures with short and long periods, archetypes with N of 2, 4, 6, 8, 10, and 12 were studied. The story height, H , was selected as 3.5 m and kept constant for all stories for all designs. Consequently, this study covers CBFs with building heights ranging from 7 m to 42 m. Please note that TBEC 2018 allows CBFs up to a height of 42 m. An example of the elevation view is given for a two-story frame (Figure 2(b)).

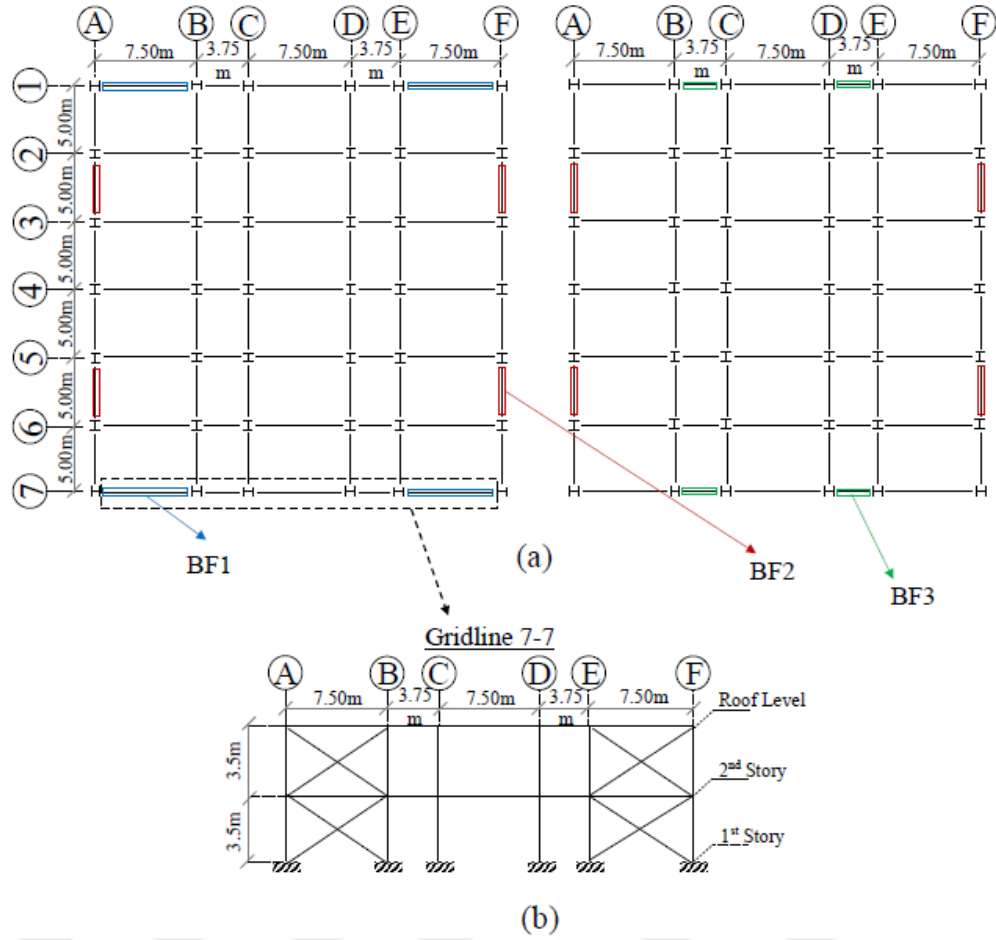


Figure 2. (a) Floor plans of archetypes and (b) elevation view of two-story archetypes

In the design stage, all beam-to-column connections and column bases were considered simple connections. The S355 grade steel with an elastic modulus, E , of 200 GPa and a yield strength, f_y , of 355 MPa, that is a common type of steel grade used in Turkey and Europe, was selected for all members. A dead load of 5 kN/m² and a live load of 2 kN/m² as well as a snow load of 2 kN/m² were considered. The seismic weight at the floor and roof levels was estimated as 5040 kN. According to FEMA P-695, the highest seismic hazard level is Seismic Design Category (SDC) D_{max} that has design earthquake spectral response acceleration parameters at short period, S_{DS} , and at 1.0 second period, S_{D1} , of 1.0 g and 0.6 g, respectively. The second highest seismic hazard level is defined as SDC C_{max} with an S_{DS} of 0.5 g and an S_{D1} of 0.2 g. Considering the design spectral response acceleration parameters given for both SDC D_{max} and C_{max} , and

adopting The Equivalent Lateral Force method, earthquake forces acting on the archetypes were determined. Please note that the importance factor, I , was taken as 1.0. R-factor of 5 was used, as proposed in TBEC 2018 for the systems that use only highly-ductile CBFs to resist the seismic loads. Designs were performed per TBEC 2018 and SDCCSS by minimizing the weight of the structural framing. In the design of beams and columns, capacity design principles were adopted as per TBEC 2018. European hollow square sections (HSS) were adopted for braces while European wide-flange sections (HE) and I-shape sections (IPE and IPN) were used for columns and beams, respectively. The combinations of considered L_s , N_s , and $SDCs$ yielded 36 archetypes. The naming scheme for the archetype as follows: $S(\text{number of stories})$ – (braced frame type: $BF1$, $BF2$ or $BF3$) – (SDC: D for $D_{\max}$ or C for $C_{\max}$). The selected sections for braces, columns, and beams of the archetypes are given in Table 1 and Table 2.

Table 1. Member sizes of SDC D_{max} archetypes

Model	S	Column	Beam	Brace	Model	S	Column	Beam	Brace	Model	S	Column	Beam	Brace
S12-BF1-D	1	HE1000M	IPE500	140X8	S10-BF1-D	1	HE650M	IPE500	140X8	S8-BF1-D	1	HE600B	IPE500	140X8
	2	HE1000M	IPE500	140X8		2	HE650M	IPE500	140X8		2	HE600B	IPE500	120X8
	3	HE1000M	IPE500	140X8		3	HE650M	IPE500	140X8		3	HE600B	IPE500	120X8
	4	HE300M	IPE500	140X8		4	HE280M	IPE500	120X8		4	HE340B	IPE500	120X8
	5	HE300M	IPE500	120X8		5	HE280M	IPE500	120X8		5	HE340B	IPE450	110X8
	6	HE300M	IPE500	120X8		6	HE280M	IPE500	120X8		6	HE340B	IPE450	110X6.3
	7	HE240M	IPE500	120X8		7	HE200M	IPE450	110X8		7	HE180B	IPE400	100X6
	8	HE240M	IPE450	110X8		8	HE200M	IPE450	110X6.3		8	HE180B	IPE330	90X6
	9	HE240M	IPE450	110X6.3		9	HE200M	IPE400	100X6	S8-BF2-D	1	HE650B	IPE360	110X6.3
	10	HE180M	IPE450	110X6.3		10	HE100M	IPE330	90X6		2	HE650B	IPE360	110X6.3
	11	HE180M	IPE400	100X6	S10-BF2-D	1	HE700M	IPE400	110X8		3	HE650B	IPE360	110X6.3
S12-BF2-D	12	HE180M	IPE330	90X6		2	HE700M	IPE400	110X8		4	HE340B	IPE360	110X6.3
	1	HE1000M	IPE400	110X8		3	HE700M	IPE360	110X6.3		5	HE340B	IPE360	100X6
	2	HE1000M	IPE400	110X8		4	HE280M	IPE360	110X6.3		6	HE340B	IPE330	90X6
	3	HE1000M	IPE400	110X8		5	HE280M	IPE360	110X6.3		7	HE180B	IPE330	80X6
	4	HE300M	IPE400	110X8		6	HE280M	IPE360	100X6	S8-BF3-D	8	HE180B	IPE240	70X5
	5	HE300M	IPE360	110X6.3		7	HE200M	IPE360	100X6		1	HE650B	IPN260	100X6
	6	HE300M	IPE360	110X6.3		8	HE200M	IPE330	90X6		2	HE650B	IPN260	100X6
	7	HE240M	IPE360	100X6.3		9	HE200M	IPE330	80X6		3	HE650B	IPN260	100X6
	8	HE240M	IPE360	100X6	S10-BF3-D	10	HE100M	IPE240	70X5		4	HE340B	IPN260	90X6
	9	HE240M	IPE330	90X6		1	HE500M	IPN260	100X6		5	HE340B	IPN240	90X6
	10	HE160M	IPE330	90X6		2	HE500M	IPN260	100X6		6	HE340B	IPN240	80X6
S12-BF3-D	11	HE160M	IPE300	80X5		3	HE500M	IPN260	100X6		7	HE180B	IPN220	80X5
	12	HE160M	IPE240	70X5		4	HE260M	IPN260	100X6		8	HE180B	IPN180	60X6
	1	HE1000M	IPN260	110X6.3		5	HE260M	IPN260	90X6.3	S6-BF1-D	1	HE400B	IPE500	120X8
	2	HE1000M	IPN260	110X6.3		6	HE260M	IPN260	90X6		2	HE400B	IPE500	120X8
	3	HE1000M	IPN260	100X6.3		7	HE200M	IPN240	90X6		3	HE400B	IPE500	120X8
	4	HE300M	IPN260	100X6		8	HE200M	IPN220	80X5		4	HE220B	IPE450	110X8
	5	HE300M	IPN260	100X6		9	HE200M	IPN220	70X5		5	HE220B	IPE400	110X6.3
	6	HE300M	IPN260	100X6		10	HE100M	IPN180	60X5		6	HE220B	IPE330	90X6
	7	HE240M	IPN260	90X6	S4-BF1-D	1	HE220M	IPE500	120X8		1	HE550A	IPE360	110X6.3
	8	HE240M	IPN240	90X6		2	HE220M	IPE450	110X8	S6-BF2-D	2	HE550A	IPE360	110X6.3
	9	HE240M	IPN240	80X6		3	HE220M	IPE400	110X6.3		3	HE550A	IPE360	100X6
	10	HE160M	IPN220	80X5		4	HE100M	IPE330	90X6		4	HE500A	IPE360	100X6
	11	HE160M	IPN220	70X5	S4-BF2-D	1	HE220M	IPE360	100X6.3		5	HE500A	IPE330	90X6
	12	HE160M	IPN180	60X5		2	HE220M	IPE360	100X6		6	HE500A	IPE240	80X5
S2-BF1-D	1	HE180B	IPE450	110X6.3		3	HE220M	IPE330	90X6	S6-BF3-D	1	HE400B	IPN260	100X6
	2	HE180B	IPE330	100X6		4	HE100M	IPE240	80X5		2	HE400B	IPN260	100X6
S2-BF2-D	1	HE180B	IPE330	90X6	S4-BF3-D	1	HE220M	IPE270	90X6		3	HE400B	IPN260	90X6
	2	HE180B	IPE240	80X5		2	HE220M	IPE270	90X6		4	HE220B	IPN240	90X6
S2-BF3-D	1	HE180B	IPE240	80X5		3	HE220M	IPE240	80X6		5	HE220B	IPN220	80X5
	2	HE180B	IPE200	70X5		4	HE100M	IPE200	70X5		6	HE220B	IPN180	70X5

*All brace members are square hollow sections where the first dimension designates the width and the second one indicates the wall thickness.

Table 2. Member sizes of SDC C_{max} archetypes

Model	Story	Column	Beam	Brace	Model	Story	Column	Beam	Brace	Model	Story	Column	Beam	Brace
S12-BF1-C	1	HE550B	IPE400	100X6.3	S10-BF1-C	1	HE550A	IPE400	100X6	S8-BF1-C	1	HE340B	IPE400	90X6.3
	2	HE550B	IPE400	100X6		2	HE550A	IPE400	100X6		2	HE340B	IPE400	90X6
	3	HE550B	IPE400	100X6		3	HE550A	IPE400	100X6		3	HE340B	IPE400	90X6
	4	HE360B	IPE400	100X6		4	HE500A	IPE400	90X6		4	HE240B	IPE400	90X6
	5	HE360B	IPE400	90X6		5	HE500A	IPE400	90X6		5	HE240B	IPE400	90X6
	6	HE360B	IPE400	90X6		6	HE500A	IPE400	90X6		6	HE240B	IPE360	80X5
	7	HE340B	IPE400	90X6		7	HE500A	IPE360	80X6		7	HE160B	IPE360	80X5
	8	HE340B	IPE400	90X6		8	HE500A	IPE360	80X5		8	HE160B	IPE300	60X6
	9	HE340B	IPE360	80X6		9	HE500A	IPE360	80X5	S8-BF2-C	1	HE360B	IPE330	80X6.3
	10	HE180B	IPE360	80X5		10	HE500A	IPE300	60X6		2	HE360B	IPE330	80X6
	11	HE180B	IPE360	70X5	S10-BF2-C	1	HE500B	IPE330	90X6		3	HE360B	IPE330	80X6
	12	HE180B	IPE300	60X6		2	HE500B	IPE330	90X6		4	HE240B	IPE300	80X5
S12-BF2-C	1	HE300M	IPE330	90X6		3	HE500B	IPE330	80X6.3	S8-BF3-C	5	HE240B	IPE300	80X5
	2	HE300M	IPE330	90X6	S10-BF3-C	4	HE340B	IPE330	80X6		6	HE240B	IPE300	70X5
	3	HE300M	IPE330	90X6		5	HE340B	IPE330	80X6		7	HE160B	IPE270	60X5
	4	HE260M	IPE330	90X6		6	HE340B	IPE300	80X5		8	HE160B	IPE240	60X4
	5	HE260M	IPE330	80X6		7	HE220B	IPE300	70X5		1	HE400B	IPE270	80X6.3
	6	HE260M	IPE330	80X6		8	HE220B	IPE300	70X5		2	HE400B	IPE270	80X6
	7	HE200M	IPE300	80X5		9	HE220B	IPE270	60X4		3	HE400B	IPE270	80X6
	8	HE200M	IPE300	80X5		10	HE120B	IPE240	50X5		4	HE240B	IPE240	70X6.3
	9	HE200M	IPE300	70X5		1	HE550B	IPE270	90X6		5	HE240B	IPE240	80X5
	10	HE140M	IPE300	60X6		2	HE550B	IPE270	90X6		6	HE240B	IPE220	60X6
	11	HE140M	IPE270	60X4		3	HE550B	IPE270	80X6.3	S6-BF1-C	7	HE140B	IPE200	60X4
	12	HE140M	IPE240	50X5		4	HE340B	IPE270	80X6		8	HE140B	IPE160	50X3
S12-BF3-C	1	HE320M	IPE270	90X6		5	HE340B	IPE270	80X6		1	HE220M	IPE400	90X6
	2	HE320M	IPE270	90X6		6	HE340B	IPE240	80X5		2	HE220M	IPE400	90X6
	3	HE320M	IPE270	90X6		7	HE200B	IPE240	60X6		3	HE220M	IPE400	90X6
	4	HE260M	IPE270	90X6		8	HE200B	IPE220	60X5		4	HE140M	IPE360	80X6
	5	HE260M	IPE270	80X6		9	HE200B	IPE200	60X4		5	HE140M	IPE360	80X5
	6	HE260M	IPE270	80X6		10	HE120B	IPE160	50X3		6	HE140M	IPE300	70X5
	7	HE200M	IPE240	80X5	S4-BF1-C	1	HE220B	IPE400	90X6	S6-BF2-C	1	HE220M	IPE330	80X6
	8	HE200M	IPE240	80X5		2	HE220B	IPE400	90X6		2	HE220M	IPE330	80X6
	9	HE200M	IPE240	60X6		3	HE220B	IPE360	80X5		3	HE220M	IPE300	80X5
	10	HE140M	IPE220	60X5		4	HE120B	IPE300	70X5		4	HE140M	IPE300	70X5
	11	HE140M	IPE200	50X5	S4-BF2-C	1	HE220B	IPE300	80X5	S6-BF3-C	5	HE140M	IPE300	60X6
	12	HE140M	IPE160	50X3		2	HE220B	IPE300	80X5		6	HE140M	IPE240	60X4
S2-BF1-C	1	HE160B	IPE360	80X6		3	HE220B	IPE300	70X5		1	HE220M	IPE270	80X6
	2	HE160B	IPE300	80X5		4	HE120B	IPE240	60X4		2	HE220M	IPE270	80X6
S2-BF2-C	1	HE160B	IPE300	70X5	S4-BF3-C	1	HE220B	IPE240	80X5		3	HE220M	IPE240	80X5
	2	HE160B	IPE240	60X5		2	HE220B	IPE240	80X5		4	HE140M	IPE240	60X6
S2-BF3-C	1	HE160B	IPE220	60X4		3	HE220B	IPE220	60X5		5	HE140M	IPE220	60X5
	2	HE160B	IPE180	60X4		4	HE120B	IPE180	50X4		6	HE140M	IPE160	50X3

*All brace members are SHS profiles.

NUMERICAL MODELING OF ARCHETYPES AND MODEL VALIDATION

The seismic performance of CBFs is dominated by the behavior of the braces. Consequently, the numerical model must reliably predict the elastic and inelastic behavior of the braces including the stiffness and strength degradation modes. To accurately represent the tensile strength, buckling strength, post-buckling behavior of braces, Uriz et al. [16] proposed a brace model in which a brace member requires to be divided into two inelastic beam-column elements with an initial camber displacement of 0.05-0.1% of the brace length at brace midspan. Hsiao et al. [17] improved the brace model developed by Uriz et al. and suggested dividing a brace member into twelve beam-column elements with an initial camber that follows a sine curve. Idealizing gusset plate connections as pinned or rigid joints and ignoring the effect of gusset plate connections might result in poor predictions of the deformation capacity of the system. Pursuant to overcoming this problem, Hsiao proposed to use rotational springs at the brace ends to simulate the nonlinear out-of-plane rotational behavior of gusset plate connections. As the gusset plates undergo negligible in-plane deformation compared to other deformation modes of the frame, three rigid links were used to mimic the associated rigidity in the gusset plate region: (1) The link extends from the work point of the connection to the physical end of the brace, (2) the link extends from the work point of the connection to the physical end of the gusset along the length of column, and (3) the link extends from the work point of the connection along the length of the beam and extends out to 75% of the gusset plate dimension on the beam. The brace model developed by Hsiao et al. was adopted in this study. Please note that an elliptical clearance of $8t_p$ was provided for the gusset plates of all braces of the archetypes to accommodate brace end rotations, where t_p is the gusset plate thickness. The initial

dimensions of the gusset plates were determined per the recommendations of Lehman et al. [18] and the Balanced Design Procedure (BDP) proposed by Roeder et al. [19] was used to determine the final dimensions of the gusset plates. For the sake of brevity, the gusset plate dimensions are not provided herein; however, they can be found in Kor (2021) [20].

Another key parameter to be considered in brace modeling is the brace fracture under cyclic loading as it might cause significant deterioration in member strength and lateral stiffness of the system as well as a potential collapse. As it was shown by Fell et al. [21], when the inelastic deformation demands reach to significantly high levels due to increasing lateral demands, fracture is expected to initiate within the cross-section of the braces under compression which were already buckled, and reduced strength causes braces to become vulnerable against accommodation of more inelastic deformation. Karamanci and Lignos [22] investigated the fracture development in bracing elements resulting from low cycle fatigue, and recommended predictive equations based on power law-fitting model for various structural steel sections to determine a strain value at failure, ε_0 . The expression derived for rectangular HSS sections is given in Eq. 1, where kL , r , w , and t are the effective length of the member, the radius of gyration, the width, and the thickness of the member's cross section, respectively.

$$\varepsilon_0 = 0.291 \left(\frac{kL}{r} \right)^{-0.484} \left(\frac{w}{t} \right)^{-0.613} \left(\frac{E}{f_y} \right)^{0.3} \quad \text{Eq. 1}$$

The seismic response of the archetypes was assessed by means of numerical analysis. Two-dimensional models of the archetypes were constructed in OpenSEES [23]. In this study, force-based nonlinear beam column elements consisting of fiber sections (distributed plasticity approach) were adopted for columns, beams and braces. The aforementioned rigid end offset approach proposed by Hsiao et al. [17] was

adopted to account for the gusset plates. The braces were separated into 12 sub-elements to form a typical sine curve shape with an apex of 1/500 of the brace length in order to simulate brace buckling. A Menegotto-Pinto (Steel02 in OpenSEES) material with a yield strength of 444 MPa (expected yield strength of S355 wide-flange sections per TBEC 2018) was used for beams and columns. Similarly, the same material was adopted for braces with a yield strength of 497 MPa (expected yield strength of S355 HSS sections per TBEC 2018). A 1.0% isotropic strain hardening is adopted for all members. The values of 17, 0.925, and 0.15 were used for the Steel02 material parameters, R0, CR1, and CR2, respectively, to calibrate the stress-strain curve of the material at the transition region between elastic and plastic plateaus. Besides, to account for the brace fracture, another material of the OpenSEES library was employed for the braces, namely the fatigue material, that wraps onto the pre-assigned material and disregards the stiffness of the wrapped member if the accumulated damage within the member reaches a predefined limit. Please note that this material records damage accumulated in the wrapped element after each time increment during analysis. This limit can be expressed as the value of strain at failure and is symbolized with ϵ_0 , which was calculated using the aforementioned predictive equation (Eq. 1). The column bases were assumed to be simple connections. In order to account for the P-D effects, a leaner column was introduced in the numerical model. The leaner column was modeled using elastic truss elements with very high axial stiffness that are connected to the frame at each floor level in the lateral direction by use of the equalDOF command in OpenSEES. Lumped masses are placed on two nodes (the intersections of beam and column centerlines) at each floor level. Furthermore, a mass and stiffness proportional Rayleigh damping of 2% was assumed for the nonlinear dynamic analysis, in compliance with the

damping ratio range of 2% to 5% suggested by the Methodology. A representative two-story model is shown schematically in Figure 3.

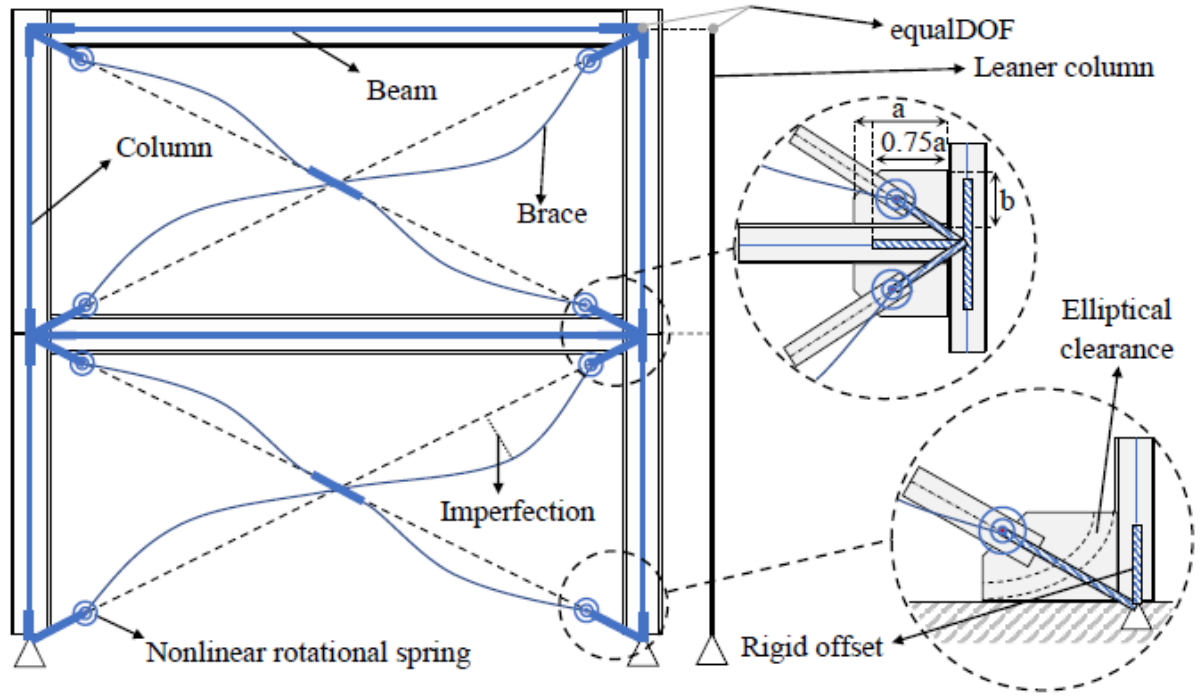


Figure 3. 2D nonlinear numerical model

The modeling approach described herein was used to model the cyclic response of a full-scale one-story one-bay CBF specimen tested by Palmer et al. [24]. Figure 4 shows the comparison of the base shear vs. drift responses of the test and the numerical model. The base shear peak values for drifts less than 0.5% was slightly underestimated by the model. This can be attributed to the fact that the imperfections assumed the model might be larger than the test specimen as this leads to a less stiff structure. However, it is seen that the model captures the response of the specimen in the post buckling region successfully in terms of both base shear and lateral stiffness, which is also in alignment with the fact that the effect of the amplitude of out-of-plane initial imperfections on the lateral stiffness is prominent for lower drifts while it vanishes as drifts increase [16,25,26]. In general, it can be deduced that the model is capable of capturing the behavior of CBFs and deemed sufficient for the purposes of this study.

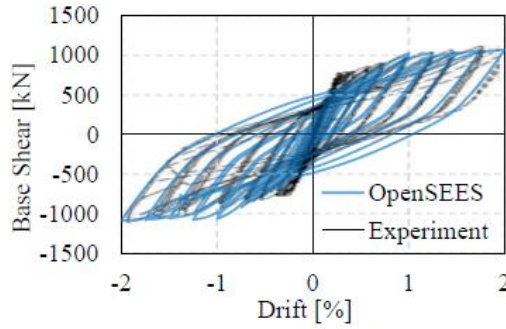


Figure 4. Comparison between the base shear vs. drift of the experiment and OpenSEES

ANALYSIS RESULTS

The aim of the study is to evaluate the seismic performance of CBFs designed per TBEC 2018 against collapse. Adopting the modeling approach described herein, the numerical models of all archetypes were built in OpenSEES. Per the Methodology, nonlinear static analyses (NSA) and nonlinear dynamic analyses (NDA) of the archetypes were performed on a cloud-based cyberinfrastructure called DesignSAFE [27].

For NSA, a lateral load distribution proportional to the fundamental mode shape of the archetype was considered and the model was pushed monotonically until observing collapse. The most prominent failure modes leading to strength or stiffness degradation of CBFs, namely, buckling, yielding, and fracture of braces and hinging in beams and columns can be captured by the model. However, other failure modes such as modes associated with the gravity frame were not included in the model. To account for such failure modes, a damage measure limit of an 8% interstory drift ratio (ISDR) was selected in accordance with the recommendations of the experimental study [28], which suggests the connections of gravity frame system are not able to accommodate rotations beyond an ISDR of 8-9%. Similarly, per the Methodology, an intensity

measure limit of a base shear (V) that is 80 % of the maximum base shear capacity (V_{max}) in post-peak region of the pushover curve was set. The archetypes were assumed to collapse if any of the damage and intensity measure limit was reached.

Figure 5 shows the normalized base shear (V/W where W is the seismic weight) vs. roof drift responses of all archetypes. V_{max} and δ_u were retrieved from Figure 5, where δ_u is the roof displacement at which collapse is observed. The Methodology requires the system overstrength, Ω , and the period-based ductility, μ_T , to be calculated. Ω and μ_T are defined as the ratio of V_{max} to the design base shear, V_d , and the ratio of δ_u to the effective yield roof drift displacement, $\delta_{y,eff}$, respectively. Eq. 2 and Eq. 3 are given for Ω and μ_T , respectively. $\delta_{y,eff}$ is defined as the abscissa of the point where a line passing through the origin and 60% of V_{max} on the pushover curve and another horizontal line passing through V_{max} intersect. T , δ_u , $\delta_{y,eff}$, μ_T , Ω , V_d , V_{max} and W are tabulated in Table 3 for each archetype.

$$\Omega = \frac{V_{max}}{V_d} \quad \text{Eq. 2}$$

$$\mu_T = \frac{\delta_{y,eff}}{\delta_u} \quad \text{Eq. 3}$$

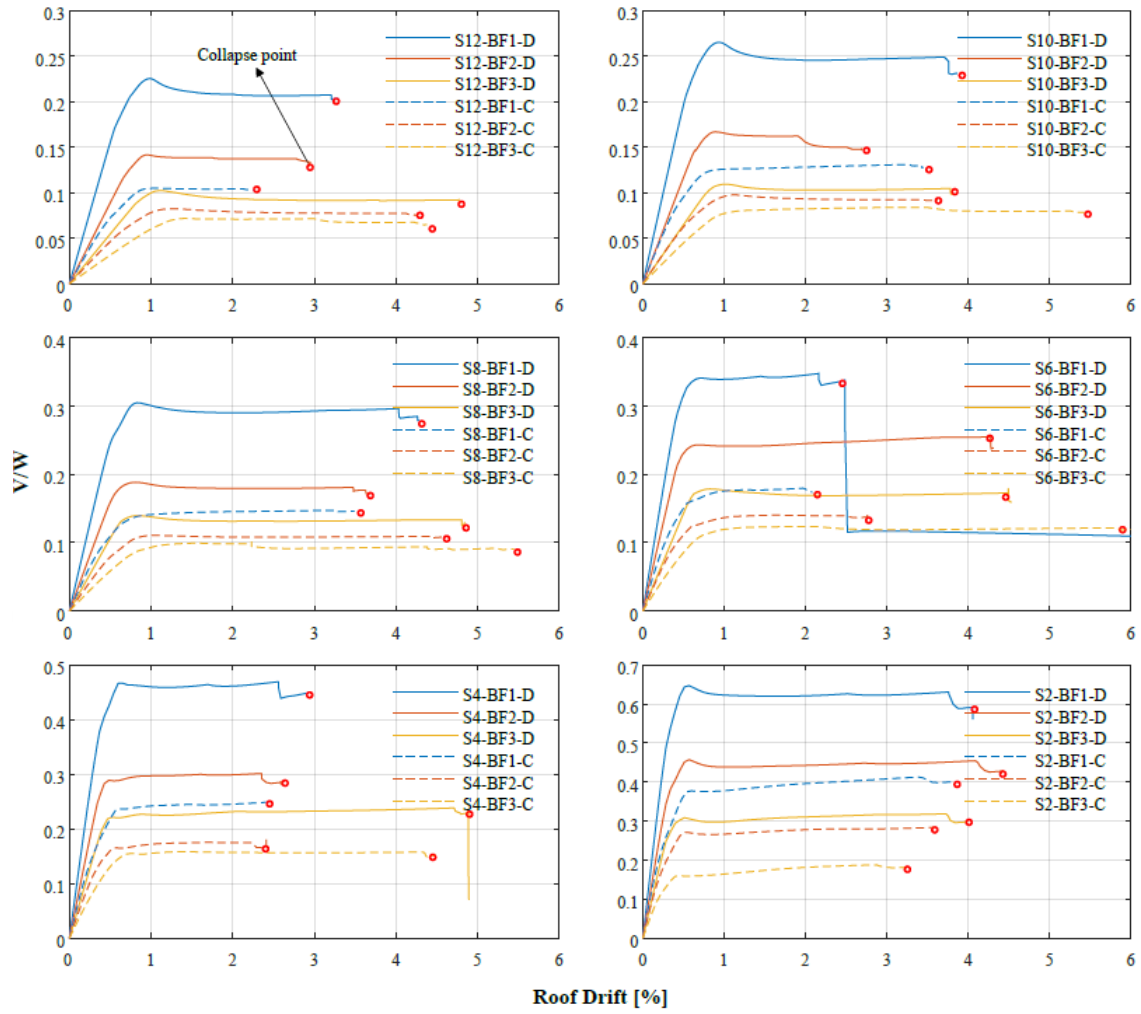


Figure 5. Normalized pushover curves of the archetypes

Table 3. Nonlinear static analysis (NSA) results

Model	T [sec]	δ_u [m]	$\delta_{y,eff}$ [m]	μ_T	Ω	V_d [kN]	V_{max} [kN]	W [kN]
S12-BF1-D	1.32	1.38	0.30	4.56	2.48	1375	3408	15120
S12-BF2-D	1.32	1.24	0.33	3.71	1.56	1375	2140	15120
S12-BF3-D	1.32	2.00	0.39	5.10	1.12	1375	1545	15120
S10-BF1-D	1.15	1.32	0.23	5.62	2.54	1313	3341	12600
S10-BF2-D	1.15	0.94	0.25	3.78	1.60	1313	2101	12600
S10-BF3-D	1.15	1.33	0.30	4.50	1.05	1313	1377	12600
S8-BF1-D	0.97	1.13	0.17	6.71	2.47	1242	3073	10080
S8-BF2-D	0.97	1.03	0.16	6.32	1.53	1242	1899	10080
S8-BF3-D	0.97	1.35	0.18	7.39	1.13	1242	1406	10080
S6-BF1-D	0.78	0.45	0.10	4.39	2.28	1156	2634	7560
S6-BF2-D	0.78	0.91	0.11	8.57	1.68	1156	1947	7560
S6-BF3-D	0.78	0.95	0.13	7.23	1.17	1156	1354	7560
S4-BF1-D	0.58	0.36	0.06	5.88	2.30	1030	2366	5040
S4-BF2-D	0.58	0.36	0.06	6.29	1.48	1030	1519	5040

S4-BF3-D	0.58	0.64	0.07	9.74	1.16	1030	1196	5040
S2-BF1-D	0.34	0.28	0.03	11.07	2.41	677	1630	2520
S2-BF2-D	0.34	0.31	0.03	11.87	1.70	677	1152	2520
S2-BF3-D	0.34	0.28	0.03	11.09	1.18	677	802	2520
S12-BF1-C	1.32	0.95	0.30	3.22	3.46	458	1585	15120
S12-BF2-C	1.32	1.75	0.38	4.66	2.72	458	1244	15120
S12-BF3-C	1.32	1.84	0.48	3.79	2.37	458	1084	15120
S10-BF1-C	1.15	1.20	0.22	5.38	3.77	438	1649	12600
S10-BF2-C	1.15	1.26	0.27	4.65	2.81	438	1229	12600
S10-BF3-C	1.15	1.90	0.33	5.71	2.41	438	1057	12600
S8-BF1-C	0.97	0.83	0.17	4.87	3.57	414	1480	10080
S8-BF2-C	0.97	1.16	0.19	6.16	2.69	414	1114	10080
S8-BF3-C	0.97	1.52	0.25	6.19	2.48	414	1026	10080
S6-BF1-C	0.78	0.44	0.11	4.02	3.52	385	1358	7560
S6-BF2-C	0.78	0.58	0.12	4.97	2.76	385	1062	7560
S6-BF3-C	0.78	1.27	0.15	8.52	2.43	385	934	7560
S4-BF1-C	0.58	0.34	0.07	5.19	3.60	348	1253	5040
S4-BF2-C	0.58	0.34	0.06	5.33	2.61	348	907	5040
S4-BF3-C	0.58	0.63	0.08	8.05	2.29	348	798	5040
S2-BF1-C	0.34	0.27	0.03	8.51	3.78	275	1040	2520
S2-BF2-C	0.34	0.25	0.03	8.71	2.60	275	714	2520
S2-BF3-C	0.34	0.23	0.03	8.77	1.72	275	473	2520

The most notable conclusion of NSA is that the archetypes designed to SDC C_{max} have higher Ω compared to those designed to SDC D_{max} regardless of the other parameters considered in this study such as L and N . On average, for a given L and N , Ω of an archetype designed to SDC C_{max} is 50 % higher than Ω of that designed to SDC D_{max} . Another observation with regards to Ω is that archetypes with narrower L s has higher Ω values. In terms of μ_T , about 80% of the SDC D_{max} archetypes are more ductile than the corresponding the SDC C_{max} archetypes. Especially short-period archetypes of the SDC D_{max} group are found to be approximately 30% more ductile than those of the SDC C_{max} group, on average.

After completing NSA, an incremental dynamic analysis (IDA) procedure was run for each archetype under 44 ground motion records per the Methodology. The

results of the IDAs are shown in Figure 6a and Figure 6b for the archetypes designed to SDC $D_{\max}$ and $C_{\max}$, respectively. It can be seen from Figure 6a that the long-period archetypes of the SDC $D_{\max}$ group underwent higher ISDRs compared to the short-period archetypes of the same group for a given spectral intensity level. The same trend was observed for the archetypes designed to the SDC $C_{\max}$, however, the difference between the short- and long-period archetypes was not as pronounced as the case in the SDC $D_{\max}$ group. This result can be attributed to the fact that the Equivalent Lateral Force method used in the design does not account for the higher-mode effects and these effects led to soft story formation in taller archetypes. As the SDC $C_{\max}$ group has higher Ω_s compared to the SDC $D_{\max}$ group, the taller archetypes in the SDC $C_{\max}$ group are less susceptible to the higher-mode effects compared to the SDC $D_{\max}$ group.

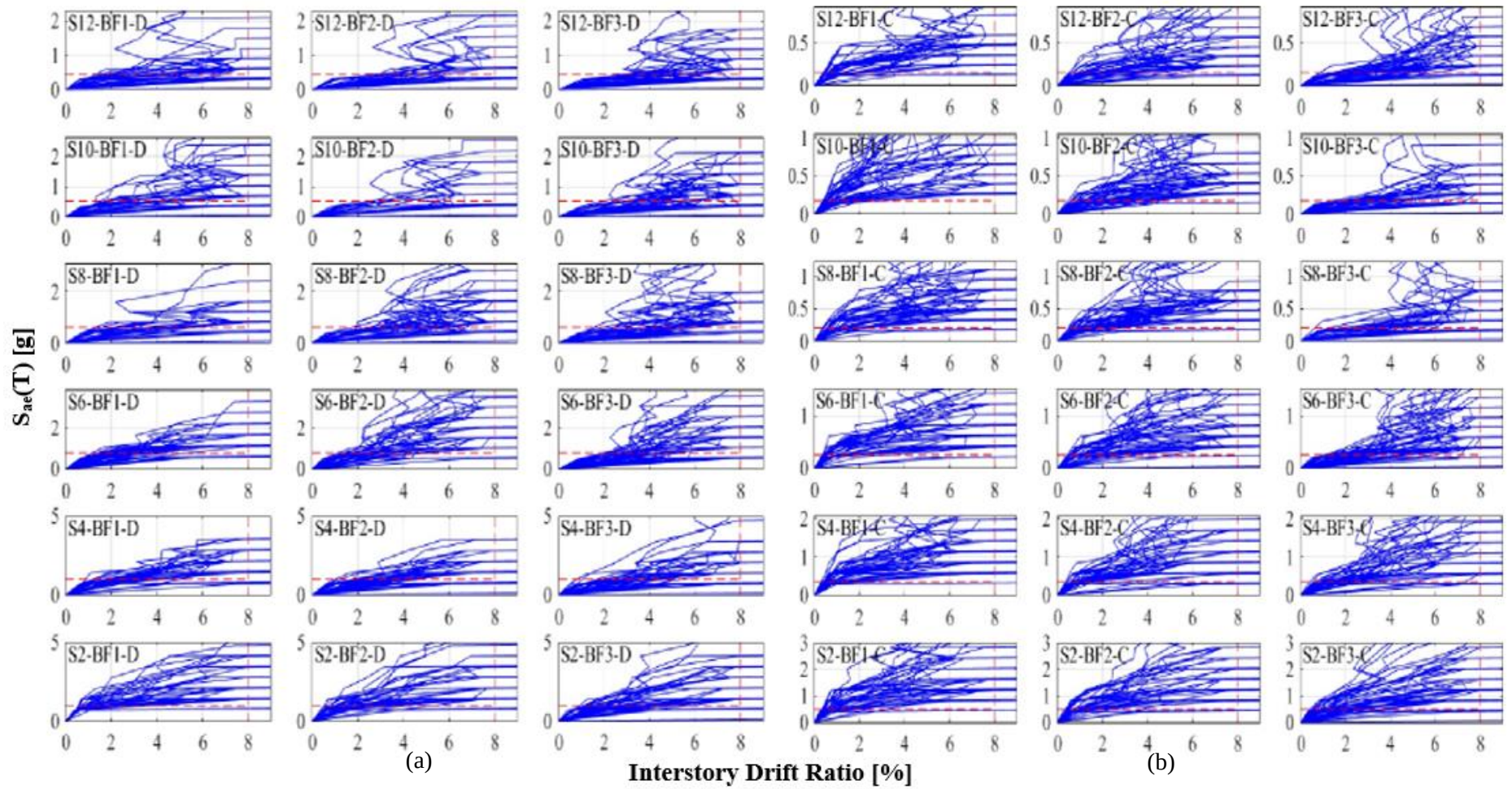


Figure 6. IDA curves (a) the archetypes designed to SDC $D_{\max}$ and (b) the archetypes designed to SDC $C_{\max}$

PERFORMANCE ASSESSMENT

The assessment method of FEMA P-695 suggests a probabilistic approach to quantify the performance of seismic load resisting systems. Seismic performance of an archetype is assessed in accordance with the results of NSA and NDA procedures by adopting the Methodology. NSA procedure provides the system overstrength and the period-based ductility, while NDA procedure provides gathering the collapse data. To perform the assessment, the collapse data is utilized to verify the acceptability of collapse probabilities.

In order to evaluate the results of NSA and NDA procedures with regards to the assessment method, it is required to account for various sources of uncertainties in addition to the analysis results due to the contribution of uncertainties to the variability in collapse capacity. The Methodology defines a parameter named the total system uncertainty (β_{TOT}) for each archetype which contains all sources of uncertainty such as record-to-record uncertainty (β_{RTR}), modeling uncertainty (β_{MDL}), design requirements uncertainty (β_{DR}), and test data uncertainty (β_{TD}). β_{RTR} accounts for the different frequency content of the ground motion records imposed to the archetypes. Other sources of uncertainty represent accuracy of the numerical model, quality and availability of the test data and design requirements. β_{MDL} is related with the accuracy of the numerical model and depends on whether the structural response is adequately represented or not. β_{DR} is an indicator of the robustness of the design requirements and scales the safety of the building design against unanticipated collapse mechanisms. The last source of uncertainty, β_{TD} , rates the test data regarding the system in terms of availability, quality and quantity. The Methodology assigns a numerical rating scale for these sources of uncertainty depending on the fulfilment of requirements, however, record-to-record uncertainty is defined as a function of period-based ductility and

calculated using the expression given below (Eq. 4), within the range between 0.2 and 0.4. β_{TOT} gathers all abovementioned components together, and it is defined as the square root of the sum of squares of β_{RTR} , β_{MDL} , β_{DR} and β_{TD} . Since the four random variables are assumed to be statistically independent, the total collapse uncertainty, β_{TOT} , is given by Eq. 5. CBF systems are commonly used in steel buildings in practice, and they have been widely investigated. As a result of those studies, various design requirements were developed, and there is a large scale of test data is available regarding the CBF behavior. Based on that, it was deemed appropriate to assume β_{MDL} , β_{DR} and β_{TD} as the minimum value of the rating scale (0.1), which indicates the highest quality level as superior (A), considering the availability of test data and seismic design requirements that aim to achieve efficient and safe designs.

$$0.2 < \beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.4 \quad \text{Eq. 4}$$

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{MDL}^2 + \beta_{DR}^2 + \beta_{TD}^2} \quad \text{Eq. 5}$$

Based on NDA results, CMR is established as the ratio of the median collapse capacity to the maximum considered earthquake (MCE) level spectral intensity, and the amplification of CMR with SSF results in ACMR, as it is explained in Chapter 2. The purpose of the amplification with SSF is accounting for the effect of different spectral shapes of the ground motion set used, depending on period-based ductility, structural period, and seismic design category. It is also prescribed in Chapter 2 that the calculated ACMR values are compared to the target ACMRs, to verify if a trial value of a R factor passes or fails the evaluation. However, this study intends to measure and quantify the collapse performance of the CBF systems by means of probabilities of failure, instead of validating the trial R factors.

It is required to collect the collapse data for the probabilistic collapse performance evaluation approach. Spectral accelerations at the collapse state were recorded for each GM, which establishes the collapse data. Probabilities of failure at each spectral intensity were calculated using the collapse data, as the ratio of number of GMs that cause failure, to total number of GMs, 44, according to the below Eq. 6. In the expression, $p_{f,x}$, $GM_{fail,x}$ and GM_{tot} represent the probability of failure at spectral intensity of x , number of GMs that cause failure at x and total number of GMs, respectively.

$$p_{f,x} = \frac{GM_{fail,x}}{GM_{tot}} \quad \text{Eq. 6}$$

Collapse fragility curves of each archetype were constructed based on curve fitting through lognormal cumulative distribution function based on the calculated probabilities of failure. The median collapse capacity of each system was observed from the collapse fragility curves, which is the point on the curve that indicates the spectral intensity at 50% of probability of failure. Besides that, dispersion (β) values were revealed as a result of the curve fitting. However, those β values cannot contain and represent all components of the β_{TOT} , namely sources of uncertainty, defined in FEMA P-695. To account for all sources of uncertainty, β_{TOT} of each archetype were calculated using the expression (Eq. 4) and employed as the input dispersion values of fragility functions. A detailed discussion regarding the β_{TOT} is given at the beginning of this chapter. After the total system uncertainties were calculated, median collapse capacities were adjusted with the *SSF* in compliance with the suggestions of the Methodology and symbolized as $\hat{S}_{CT}$. Collapse fragility curves of each system were reconstructed by using the aforementioned β_{TOT} and $\hat{S}_{CT}$ values. Please note that the first collapse fragility was only established to obtain the random variable representing median collapse intensity, S_{CT} , while the second one is for the assessment of the system in terms of collapse

behavior. Also, it is possible to accept the point where 22 GMs out of 44 cause failure as S_{CT} without a full IDA application according to FEMA P-695, however in this study it is intended to capture a better estimate of S_{CT} . To that end, a full IDA approach was adopted to collect a wider data range which enabled to observe S_{CT} values based on collapse fragility functions instead of directly accepting the point where half of the GMs cause failure.

Figure 7 presents collapse fragility curves of each archetype, in terms of probability of failure vs. spectral intensity normalized by design spectral acceleration, with the normalized median spectral intensity plotted with marker points. Figure 7a and Figure 7b includes SDC D_{max} and SDC C_{max} archetypes, respectively. The first observation that can be made from the comparison of Figure 7a and b is that SDC C_{max} archetypes show significantly higher performance in terms of collapse capacities. Almost entire SDC C_{max} set, except 2 BF3 type individual archetypes, has remarkably higher values of normalized spectral intensities at collapse levels, compared to the SDC D_{max} . Collapse spectral intensities of SDC C_{max} archetypes are found to be 4 to 7.9 times greater than the design spectral accelerations, while this difference is relatively low for the SDC D_{max} archetypes which ranges between 1.23 to 3.86. This is a primary indicator which presents that SDC C_{max} designs have better performance compared to SDC D_{max} by means of collapse performance. On the other hand, the structural period, which depends on number of stories or building height, was another difference that is considered for the archetype selection in order to investigate the relation between seismic performance and structural period. Collapse fragility curves showed that the long period systems are found to be more vulnerable compared to the short periods, since their collapse level spectral intensities are relatively higher. Even normalized collapse spectral intensities of a number of SDC D_{max} -long period archetypes observed

to be less than 2.0 while this ratio can exceed 3.0 in a couple of short period systems. The same behavior was also obtained between long and short period SDC C_{max} archetypes. This situation can be associated with the effect of increasing number of buckled or fractured braces proportional with the number of stories, which can yield to a dramatic drop in the lateral stiffness of the related stories. Apart from the seismic hazard level and structural period, aspect ratio (bay width-to-story height) of the braced frame was not found to have a remarkable effect on the collapse capacity of SDC D_{max} archetypes however, normalized median collapse capacities of SDC C_{max} group decreases by approximately 15% with the decreasing bay width, on average.

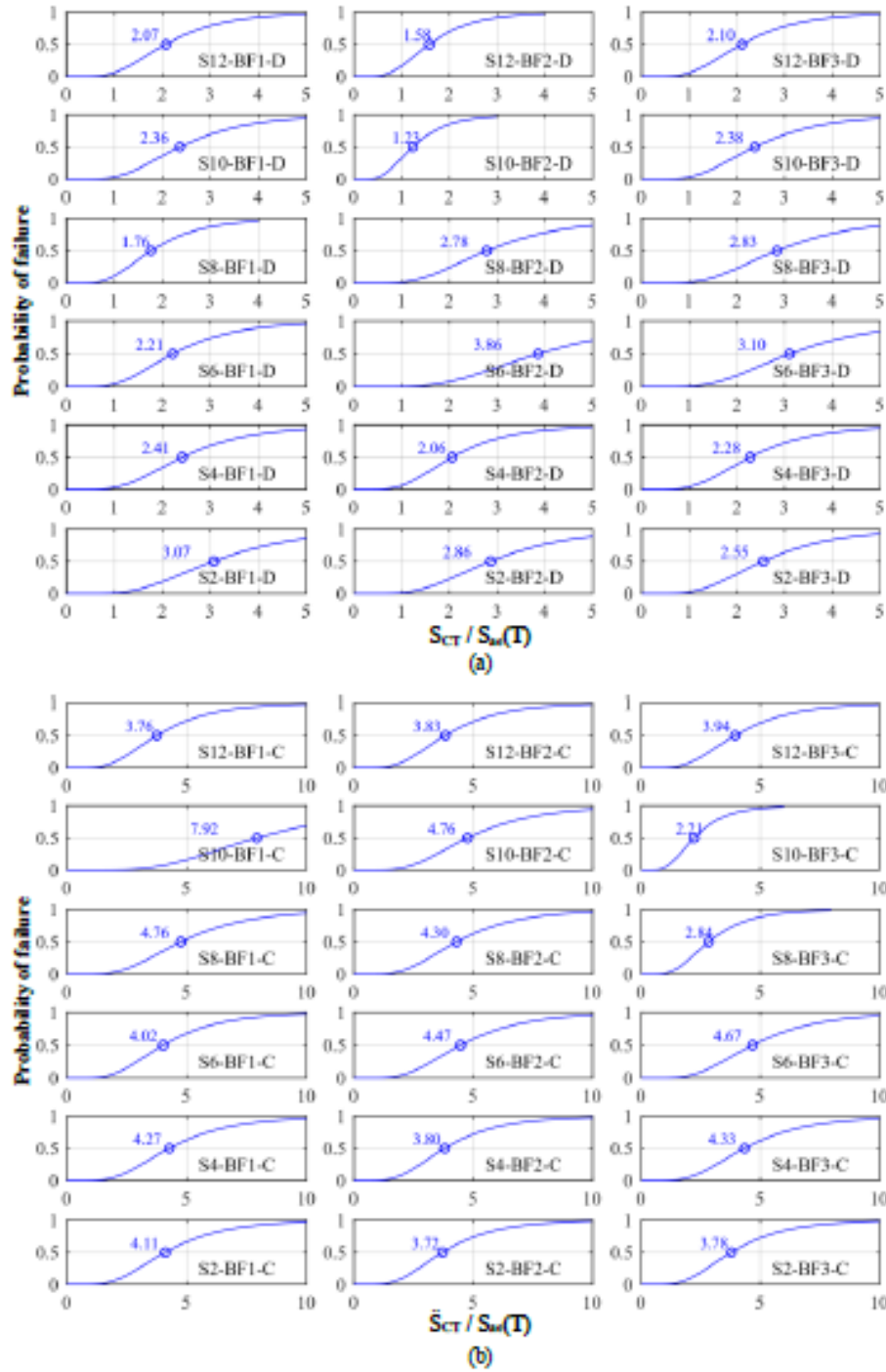


Figure 7. Fragility curves with adjusted mean collapse capacity normalized with respect to design spectral intensity ($\hat{S}_{CT}/S_{ae}(T)$) on x-axis vs probability of failure (p_f) on y-axis, for (a) SDC D_{max} and (b) SDC C_{max} archetypes

In accordance with the fragility curves, collapse probabilities of each archetype were obtained as a function of the design earthquake level spectral accelerations. To establish the collapse probabilities under the effect of infrequent seismic actions and evaluate the collapse performance of SCBFs, it is required to transform the DBE level

spectral accelerations to MCE level. ASCE/SEI 07-16 defines a constant ratio between the $S_{ae}(T)$ values of MCE and DBE level ground motion intensity response spectrums (MCE/DBE) as 1.5, while neither TBEC 2018 nor the previous versions of Turkish seismic codes specify such ratio. However, this transformation is necessary to perform the collapse assessment under MCE level earthquakes. Therefore, the mapped spectral acceleration coefficients of the Interactive Seismic Hazard Maps [10] were retrieved to investigate the relation between MCE and DBE level GM intensity levels. DBE level coefficients were compared with regards to MCE level coefficients. As a result of this process, MCE to DBE ratios, which will be referred to as τ hereupon, are found to be ranging between 1.40 to 1.75. MCE level collapse probabilities were calculated by transforming the DBE level probabilities of failure to MCE level by employing the τ ratio as 1.40 and 1.75, namely minimum and maximum; and illustrated in Figure 8 for SDC D_{max} and SDC C_{max} . Similarly, data regarding the collapse performance of all archetypes is reported in Table 4, including the uncertainties, design spectral accelerations, normalized median collapse capacities and MCE level probabilities of failure.

Table 4. Calculated collapse probabilities of each archetype with respect to the results of NDA

Label	β_{RTR}	β_{TOT}	$S_{ae}(T)$	$\hat{S}_{CT}$	Pr	
			[g]	Normalized w.r.t $S_{ae}(T)$	$\tau = 1.40$	$\tau = 1.75$
S12-BF1-D	0.4	0.436	0.45	2.07	45%	64%
S12-BF2-D	0.4	0.436	0.45	1.58	65%	80%
S12-BF3-D	0.4	0.436	0.45	2.10	46%	64%
S10-BF1-D	0.4	0.436	0.52	2.36	35%	54%
S10-BF2-D	0.4	0.436	0.52	1.23	81%	91%
S10-BF3-D	0.4	0.436	0.52	2.38	32%	50%
S8-BF1-D	0.4	0.436	0.62	1.76	60%	76%
S8-BF2-D	0.4	0.436	0.62	2.78	23%	39%
S8-BF3-D	0.4	0.436	0.62	2.83	23%	40%
S6-BF1-D	0.4	0.436	0.76	2.21	33%	51%
S6-BF2-D	0.4	0.436	0.76	3.86	8%	17%
S6-BF3-D	0.4	0.436	0.76	3.10	16%	30%

Label	β_{RTR}	β_{TOT}	$S_{ae}(T)$	$\hat{S}_{CT}$	pr	
			[g]	Normalized w.r.t $S_{ae}(T)$	$\tau = 1.40$	$\tau = 1.75$
S4-BF1-D	0.4	0.436	1.00	2.41	27%	45%
S4-BF2-D	0.4	0.436	1.00	2.06	40%	59%
S4-BF3-D	0.4	0.436	1.00	2.28	35%	53%
S2-BF1-D	0.4	0.436	1.00	3.07	14%	28%
S2-BF2-D	0.4	0.436	1.00	2.86	18%	33%
S2-BF3-D	0.4	0.436	1.00	2.55	25%	42%
S12-BF1-C	0.4	0.436	0.15	3.76	4%	10%
S12-BF2-C	0.4	0.436	0.15	3.83	5%	11%
S12-BF3-C	0.4	0.436	0.15	3.94	4%	9%
S10-BF1-C	0.4	0.436	0.17	7.92	0%	0%
S10-BF2-C	0.4	0.436	0.17	4.76	1%	4%
S10-BF3-C	0.4	0.436	0.17	2.21	30%	48%
S8-BF1-C	0.4	0.436	0.21	4.76	1%	4%
S8-BF2-C	0.4	0.436	0.21	4.30	2%	7%
S8-BF3-C	0.4	0.436	0.21	2.84	14%	27%
S6-BF1-C	0.4	0.436	0.25	4.02	2%	7%
S6-BF2-C	0.4	0.436	0.25	4.47	1%	5%
S6-BF3-C	0.4	0.436	0.25	4.67	1%	4%
S4-BF1-C	0.4	0.436	0.35	4.27	2%	5%
S4-BF2-C	0.4	0.436	0.35	3.80	3%	8%
S4-BF3-C	0.4	0.436	0.35	4.33	2%	5%
S2-BF1-C	0.4	0.436	0.50	4.11	2%	6%
S2-BF2-C	0.4	0.436	0.50	3.72	4%	9%
S2-BF3-C	0.4	0.436	0.50	3.78	3%	8%

FEMA P-695 suggests 20% and 10% of acceptable collapse probabilities for individual archetypes and performance groups, respectively. A performance group is defined as a part within an archetype set that classifies the individual archetypes depending on several similarities such as seismic hazard level, structural layout or material etc. However in this study, it is intended to investigate the individual behavior of each archetype based on collapse performance, so the archetypes were not evaluated as separate performance groups.

Table 4 reveals the vulnerability of SDC $D_{\max}$ designs against MCE level ground motions at the first sight, since the collapse probabilities of those archetypes are intensely high. Secondly, it is observed that the collapse probabilities are in an increasing trend towards short to long period archetypes. Please note a similar deduction was made previously in collapse fragilities. This observation is not only valid for SDC $D_{\max}$ but also for SDC $C_{\max}$. Both groups become vulnerable against MCE level seismic actions as the periods lengthen. Especially 10 and 8 story BF3-C archetypes have 3 to 4 times greater collapse probabilities compared to the average. However, these individual archetypes are not deemed to represent the whole behavior of SDC $C_{\max}$ CBF designs, since they consist only 10% of the group. Besides that exception, it can be inferred that this group has sufficient collapse performance in general, since the probabilities of failure within the group are less than 10% approximately, for both values of τ ratios. On the other hand, collapse probabilities of almost all SDC $D_{\max}$ archetypes are found critical for the τ ratio of 1.75. Likewise, the other τ ratio, 1.40, yields to significant level of collapse probabilities, except 4 short period systems having less than 20% collapse probability. In particular, 8 to 12 story archetypes have dramatically higher collapse probabilities, which indicates that they are not able to provide safeguard against infrequent ground motions as there were considerable drift accommodations observed. Briefly, increasing building height, or the total number of stories, clearly has a negative impact on the seismic performance of SCBFs since the collapse probability is found to be directly proportional with the building height in general. Apart from that, the aspect ratio of the frame, which is another parameter considered in archetype selection, does not affect the seismic performance as sharp as the building height. BF1 archetypes with shorter period have slight less collapse probability for both SDCs. This can be attributed to the additional axial stiffness effect of bigger cross sections selected for the beams to

resist the increasing gravity load proportional with the span length. However, the effect of aspect ratio disappears for higher archetypes, in which collapse probabilities are observed to be independent from the aspect ratio.

As a general outcome, collapse performances of SDC D_{max} type SCBF designs considered in this study are not found satisfactory under MCE level seismic actions since the collapse performance are obtained to be more than 20% on average. Likewise, the building height was observed to have a negative impact on seismic performance of CBFs designed to TBEC 2018 since the collapse probability is clearly parallel with the increasing building height.



Figure 8. Collapse probabilities calculated for both SDCs by adopting τ as 1.40 and 1.75

CONCLUSION

In this study, it is aimed to investigate the dynamic structural response and to assess the seismic performance of SCBF systems designed per the recently revised and

published national seismic design code of Turkey, TBEC 2018, which came into force with various changes in seismic design and detailing of steel structures. To that end, the methodology defined in FEMA P-695 was adopted as the fundamental path of this study, which includes the design, modeling and nonlinear static and dynamic analyses procedures. Initially, a nonlinear numerical model was developed to represent the structural behavior of SCBFs. Several material models and modeling techniques were implemented into the model in order to achieve accurate simulations and account for different structural responses. A set of index archetypes were designed considering 6 story heights ranging from 7 meters to 42 meters as it is defined as the maximum allowed height for SCBFs in TBEC 2018, 3 bay widths as 3.75 m, 5.00 m and 7.5 m, and 2 Seismic Design Categories as D_{max} and C_{max} of ASCE/SEI-07, which yielded to a total of 36 SCBF design. Nonlinear static and dynamic analyses were performed using the numerical model that is developed for the purposes of this study. Results showed lateral structural behaviors of the archetype set, and they provided the parameters required to construct the collapse fragilities as well. To carry out the assessment, several τ ratios were derived by processing the data retrieved from the Turkish Seismic Hazard Maps, to perform the transformation between DBE and MCE level ground motion intensities since it is not specified in TBEC 2018. Accordingly, collapse probabilities of each system under MCE level seismic effects were calculated to provide an insight into the collapse performance of the SCBFs considered. In general, SDC C_{max} type archetypes are found adequate to resist the seismic actions under corresponding MCE level GM intensity, while SDC D_{max} type ones are not able to prevent a potential undesired collapse. Both individual and group average collapse probabilities of SDC D_{max} archetypes are deemed unacceptable despite a few exceptions. Especially, long period archetypes are failed to exhibit satisfactory performance, since their collapse

probabilities are significantly high as it is observed from the fragility curves, and they reached to a collapse state in earlier stages of the IDA procedure with regards to NDA results. In brief, the probabilistic assessment performed in this study revealed that the collapse performance of SDC $D_{\max}$ category highly-ductile SCBFs designed to TBEC 2018 is unsatisfactory under the effect of MCE level GMs. On the contrary, SDC $C_{\max}$ category archetypes are deemed safe in terms of collapse capacity under corresponding MCE level GMs.

ACKNOWLEDGEMENTS

The first author was financially supported by Ozyegin University. The cyberinfrastructure provided by the Texas Advanced Computing Center (TACC), DesignSafe, is appreciated. Any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

REFERENCES

- [1] S.C. Goel, Cyclic Post Buckling Behavior of Steel Bracing Members, in: Stability and Ductility of Steel Structures under Cyclic Loading, CRC Press, Boca Raton, FL, 1992: pp. 75–104.
- [2] S.C. Goel, Earthquake Resistant Design of Ductile Braced Steel Structures, in: Stability and Ductility of Steel Structures under Cyclic Loading, CRC Press, Boca Raton, FL, 1992: pp. 297–308.
- [3] Turkish Standards Institution (TSE), Building Code for Steel Structures, TS 648. (1980).

- [4] Republic of Turkey Ministry of Environment and Urbanisation, Turkish Specification for the Design, Calculation and Construction of Steel Structures, SDCCSS 2016. (2016).
- [5] American Institute of Steel Construction, Specification for Structural Steel Buildings, 2010.
- [6] Disaster and Emergency Management Agency (AFAD), Turkish Building Earthquake Code 2018, TBEC 2018. (2018).
- [7] Republic of Turkey Ministry of Public Works and Housing, Turkish Earthquake Code 2007, TEC 2007. (2007).
- [8] American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2016.
- [9] American Institute of Steel Construction, Seismic Provisions for Structural Steel Buildings, 2016.
- [10] Disaster and Emergency Management Agency (AFAD), Turkey Earthquake Risk Map, (2018).
- [11] E. Kor, Y. Ozelik, Seismic Performance Evaluation of Steel X-Braced Frames per the Turkish Building Earthquake Code 2018, in: 9th Turkish Conference on Earthquake Engineering, Istanbul, 2021.
- [12] E. Kor, Y. Ozelik, Incremental Dynamic Analysis of Concentric X-Braced Frames Designed to the TBEC 2018, in: Eurosteel 2021, Ernst&Sohn, Sheffield, 2021.
- [13] M. Adali, Y. Ozelik, Seismic Performance Evaluation of Steel Chevron Braced Frames Per Turkish Building Earthquake Code 2018, in: 2019.

- [14] Y. Ozcelik, M. Adali, Collapse Assessment of Concentrically Braced Frame Designed to Turkish Building Earthquake Code 2018 by Using Incremental Dynamic Analysis, in: 2nd International Conference on Seismic Design of Industrial Facilities, 2020. <https://www.researchgate.net/publication/346520989>.
- [15] Federal Emergency Management Agency, Quantification of Building Seismic Performance Factors, FEMA P-695. (2009). www.ATCouncil.org.
- [16] P. Uriz, F.C. Filippou, S.A. Mahin, Model for Cyclic Inelastic Buckling of Steel Braces, *Journal of Structural Engineering*. 134 (2008). <https://doi.org/10.1061/ASCE0733-94452008134:4619>.
- [17] P.C. Hsiao, D.E. Lehman, C.W. Roeder, Improved analytical model for special concentrically braced frames, *Journal of Constructional Steel Research*. 73 (2012) 80–94. <https://doi.org/10.1016/j.jcsr.2012.01.010>.
- [18] D.E. Lehman, C. Roeder, D. Herman, S. Johnson, B. Kotulka, Improved Seismic Performance of Gusset Plate Connections, *Journal of Structural Engineering*. 134 (2008) 890–901. <https://doi.org/10.1061/ASCE0733-94452008134:6890>.
- [19] C.W. Roeder, E.J. Lumpkin, D.E. Lehman, A balanced design procedure for special concentrically braced frame connections, *Journal of Constructional Steel Research*. 67 (2011) 1760–1772. <https://doi.org/10.1016/j.jcsr.2011.04.016>.
- [20] E. Kor, Seismic Performance of Concentric X-Braced Steel Frames Designed to the Turkish Building Earthquake Code 2018, 2021.
- [21] Fell B V, Kanvinde A M, ASCE A M, Deierlein G G, ASCE F, Myers A T, Experimental Investigation of Inelastic Cyclic Buckling and Fracture of Steel Braces,

- Journal of Structural Engineering. 135 (2009) 19–32.
<https://doi.org/10.1061/ASCE0733-94452009135:119>.
- [22] E. Karamanci, D.G. Lignos, Computational Approach for Collapse Assessment of Concentrically Braced Frames in Seismic Regions, *Journal of Structural Engineering*. 140 (2014). [https://doi.org/10.1061/\(asce\)st.1943-541x.0001011](https://doi.org/10.1061/(asce)st.1943-541x.0001011).
- [23] Mazzoni S, McKenna F, Scott M H, Fenves G L, Open System for Earthquake Engineering Simulation User Command-Language Manual, (2006).
- [24] K.D. Palmer, C.W. Roeder, D.E. Lehman, T. Okazaki, C.K. Shield, J. Powell, Concentric X-braced frames with HSS bracing, *International Journal of Steel Structures*. 12 (2012) 443–459. <https://doi.org/10.1007/s13296-012-3012-8>.
- [25] B. Qu, F. Sanchez-Zamora, M. Pollino, Mitigation of inter-story drift concentration in multi-story steel Concentrically Braced Frames through implementation of Rocking Cores, *Engineering Structures*. 70 (2014). <https://doi.org/10.1016/j.engstruct.2014.03.032>.
- [26] A.Y. Ozcelik, A. Saritas, Comparison of Chevron and Suspended Zipper Braced Steel Frames, in: 9th International Congress on Advances in Civil Engineering, 2010: pp. 27–30.
- [27] E.M. Rathje, C. Dawson, J.E. Padgett, J.-P. Pinelli, D. Stanzione, A. Adair, P. Arduino, S.J. Brandenburg, T. Cockerill, C. Dey, M. Esteva, F.L. Haan, M. Hanlon, A. Kareem, L. Lowes, S. Mock, G. Mosqueda, DesignSafe: New Cyberinfrastructure for Natural Hazards Engineering, *Natural Hazards Review*. 18 (2017) 06017001. [https://doi.org/10.1061/\(asce\)nh.1527-6996.0000246](https://doi.org/10.1061/(asce)nh.1527-6996.0000246).

- [28] B. Judy Liu, A. Astaneh-Asl, Cyclic Testing Of Simple Connections Including Effects Of Slab, Journal Of Structural Engineering. 130 (2000).
<https://doi.org/10.1061/~ASCE!0733-9445~2004!130:9~1371!>



APPENDIX D

SECTION & GUSSET PLATE TABLES

Table- 2. I section table used for the beams and columns

Sct No	Section	W (N/m)	W (kg/m)	A (mm ²)	Sct No	Section	W (N/m)	W (kg/m)	A (mm ²)
1	IPE100	79.5	8.1	1032	55	HE260B	912.3	93.0	11840
2	IPE120	102.0	10.4	1321	56	HE320A	957.5	97.6	12440
3	IPN120	108.9	11.1	1420	57	HE200M	1010.4	103.0	13130
4	IPE140	126.5	12.9	1643	58	HE280B	1010.4	103.0	13140
5	IPN140	140.3	14.3	1830	59	HE340A	1030.1	105.0	13350
6	IPE160	155.0	15.8	2009	60	IPE550	1039.9	106.0	13440
7	HE100A	163.8	16.7	2124	61	HE360A	1098.7	112.0	14280
8	IPN160	175.6	17.9	2280	62	IPN450	1128.2	115.0	14700
9	IPE180	184.4	18.8	2395	63	HE220M	1147.8	117.0	14940
10	HE120A	195.2	19.9	2534	64	HE300B	1147.8	117.0	14910
11	HE100B	200.1	20.4	2604	65	IPE600	1196.8	122.0	15600
12	IPN180	214.8	21.9	2790	66	HE400A	1226.3	125.0	15900
13	IPE200	219.7	22.4	2848	67	HE320B	1245.9	127.0	16130
14	HE140A	242.3	24.7	3142	68	HE340B	1314.5	134.0	17090
15	IPE220	257.0	26.2	3337	69	HE450A	1373.4	140.0	17800
16	IPN200	257.0	26.2	3340	70	IPN500	1383.2	141.0	17900
17	HE120B	261.9	26.7	3401	71	HE360B	1393.0	142.0	18060
18	HE160A	298.2	30.4	3877	72	HE400B	1520.6	155.0	19780
19	IPE240	301.2	30.7	3912	73	HE500A	1520.6	155.0	19750
20	IPN220	305.1	31.1	3950	74	HE240M	1540.2	157.0	19960
21	HE140B	330.6	33.7	4296	75	HE550A	1628.5	166.0	21180
22	HE180A	348.3	35.5	4525	76	IPN550	1628.5	166.0	21200
23	IPE270	354.1	36.1	4595	77	HE450B	1677.5	171.0	21800
24	IPN240	355.1	36.2	4610	78	HE260M	1687.3	172.0	21960
25	HE100M	410.1	41.8	5324	79	HE600A	1746.2	178.0	22650
26	IPN260	411.0	41.9	5330	80	HE500B	1834.5	187.0	23860
27	IPE300	414.0	42.2	5381	81	HE280M	1854.1	189.0	24020
28	HE200A	415.0	42.3	5383	82	HE650A	1863.9	190.0	24160
29	HE160B	417.9	42.6	5425	83	HE550B	1952.2	199.0	25410
30	IPN280	469.9	47.9	6100	84	HE700A	2001.2	204.0	26050
31	IPE330	481.7	49.1	6261	85	HE600B	2079.7	212.0	27000
32	HE220A	495.4	50.5	6434	86	HE800A	2197.4	224.0	28580
33	HE180B	502.3	51.2	6525	87	HE650B	2207.3	225.0	28630
34	HE120M	511.1	52.1	6641	88	HE300M	2334.8	238.0	30310
35	IPN300	531.7	54.2	6900	89	HE700B	2364.2	241.0	30640
36	IPE360	560.2	57.1	7273	90	HE320M	2403.5	245.0	31200
37	HE240A	591.5	60.3	7684	91	HE340M	2432.9	248.0	31580

Sct No	Section	W (N/m)	W (kg/m)	A (mm ²)	Sct No	Section	W (N/m)	W (kg/m)	A (mm ²)
38	IPN320	598.4	61.0	7770	92	HE360M	2452.5	250.0	31880
39	HE200B	601.4	61.3	7808	93	HE900A	2472.1	252.0	32050
40	HE140M	620.0	63.2	8056	94	HE400M	2511.4	256.0	32580
41	IPE400	650.4	66.3	8446	95	HE800B	2570.2	262.0	33420
42	IPN340	667.1	68.0	8670	96	HE450M	2580.0	263.0	33540
43	HE260A	669.0	68.2	8682	97	HE500M	2648.7	270.0	34430
44	HE220B	701.4	71.5	9104	98	HE1000A	2668.3	272.0	34680
45	IPN360	746.5	76.1	9700	99	HE550M	2727.2	278.0	35440
46	HE160M	747.5	76.2	9705	100	HE600M	2795.9	285.0	36370
47	HE280A	749.5	76.4	9726	101	HE900B	2854.7	291.0	37130
48	IPE450	761.3	77.6	9882	102	HE650M	2874.3	293.0	37370
49	HE240B	816.2	83.2	10600	103	HE700M	2952.8	301.0	38300
50	IPN380	824.0	84.0	10700	104	HE1000B	3080.3	314.0	40000
51	HE300A	866.2	88.3	11250	105	HE800M	3109.8	317.0	40430
52	HE180M	872.1	88.9	11330	106	HE900M	3266.7	333.0	42360
53	IPE500	889.8	90.7	11550	107	HE1000M	3423.7	349.0	44420
54	IPN400	906.4	92.4	11800					

Table- 3. SHS section table used for the braces

Type	No	Section	No	Section	No	Section
Square	1	SHS 20X1.5	57	SHS 90X5	113	SHS 250X6.3
Square	2	SHS 20X2	58	SHS 110X4	114	SHS 260X6.3
Square	3	SHS 25X1.5	59	SHS 80X6	115	SHS 180X10
Square	4	SHS 20X2.5	60	SHS 80X6.3	116	SHS 280X6
Square	5	SHS 30X1.5	61	SHS 120X4	117	SHS 220X8
Square	6	SHS 25X2	62	SHS 100X5	118	SHS 300X6
Square	7	SHS 35X1.5	63	SHS 90X6	119	SHS 200X10
Square	8	SHS 25X2.5	64	SHS 90X6.3	120	SHS 300X6.3
Square	9	SHS 30X2	65	SHS 110X5	121	SHS 320X6
Square	10	SHS 40X1.5	66	SHS 80X8	122	SHS 250X8
Square	11	SHS 25X3	67	SHS 140X4	123	SHS 180X12.5
Square	12	SHS 35X2	68	SHS 100X6	124	SHS 260X8
Square	13	SHS 30X2.5	69	SHS 100X6.3	125	SHS 220X10
Square	14	SHS 50X1.5	70	SHS 120X5	126	SHS 350X6
Square	15	SHS 40X2	71	SHS 150X4	127	SHS 200X12
Square	16	SHS 30X3	72	SHS 110X6	128	SHS 280X8
Square	17	SHS 35X2.5	73	SHS 90X8	129	SHS 200X12.5
Square	18	SHS 40X2.5	74	SHS 110X6.3	130	SHS 300X8
Square	19	SHS 35X3	75	SHS 140X5	131	SHS 250X10
Square	20	SHS 50X2	76	SHS 120X6	132	SHS 400X6
Square	21	SHS 40X3	77	SHS 100X8	133	SHS 220X12
Square	22	SHS 60X2	78	SHS 120X6.3	134	SHS 260X10

Type	No	Section	No	Section	No	Section
Square	23	SHS 50X2.5	79	SHS 180X4	135	SHS 220X12.5
Square	24	SHS 70X2	80	SHS 150X5	136	SHS 320X8
Square	25	SHS 40X4	81	SHS 160X5	137	SHS 400X6.3
Square	26	SHS 50X3	82	SHS 110X8	138	SHS 280X10
Square	27	SHS 60X2.5	83	SHS 140X6	139	SHS 350X8
Square	28	SHS 80X2	84	SHS 140X6.3	140	SHS 250X12
Square	29	SHS 70X2.5	85	SHS 100X10	141	SHS 250X12.5
Square	30	SHS 60X3	86	SHS 150X6	142	SHS 300X10
Square	31	SHS 50X4	87	SHS 120X8	143	SHS 260X12
Square	32	SHS 80X2.5	88	SHS 180X5	144	SHS 260X12.5
Square	33	SHS 70X3	89	SHS 150X6.3	145	SHS 320X10
Square	34	SHS 50X5	90	SHS 160X6	146	SHS 400X8
Square	35	SHS 60X4	91	SHS 110X10	147	SHS 280X12.5
Square	36	SHS 90X2.5	92	SHS 160X6.3	148	SHS 300X12
Square	37	SHS 80X3	93	SHS 200X5	149	SHS 350X10
Square	38	SHS 100X2.5	94	SHS 140X8	150	SHS 300X12.5
Square	39	SHS 50X6	95	SHS 120X10	151	SHS 320X12.5
Square	40	SHS 50X6.3	96	SHS 180X6	152	SHS 400X10
Square	41	SHS 70X4	97	SHS 220X5	153	SHS 280X16
Square	42	SHS 90X3	98	SHS 180X6.3	154	SHS 350X12.5
Square	43	SHS 60X5	99	SHS 150X8	155	SHS 300X16
Square	44	SHS 110X2.5	100	SHS 200X6	156	SHS 400X12
Square	45	SHS 100X3	101	SHS 160X8	157	SHS 320X16
Square	46	SHS 80X4	102	SHS 200X6.3	158	SHS 400X12.5
Square	47	SHS 60X6	103	SHS 250X5	159	SHS 500X10
Square	48	SHS 60X6.3	104	SHS 140X10	160	SHS 350X16
Square	49	SHS 70X5	105	SHS 220X6	161	SHS 320X20
Square	50	SHS 110X3	106	SHS 220X6.3	162	SHS 400X16
Square	51	SHS 90X4	107	SHS 150X10	163	SHS 500X12.5
Square	52	SHS 120X3	108	SHS 180X8	164	SHS 350X20
Square	53	SHS 80X5	109	SHS 160X10	165	SHS 400X20
Square	54	SHS 70X6	110	SHS 250X6	166	SHS 500X16
Square	55	SHS 70X6.3	111	SHS 200X8	167	SHS 500X20
Square	56	SHS 100X4	112	SHS 260X6		

Table- 4. Required plate thicknesses vs. plate thicknesses that are available in the market and used for the gusset plate connections

Plate Thickness (<i>mm</i>)					
Required	Available	Required	Available	Required	Available
4	4	13	13	22	25
5	5	14	14	23	25
6	6	15	15	24	25
7	7	16	16	25	25
8	8	17	18	26	30
9	9	18	18	27	30
10	10	19	20	28	30
11	12	20	20	29	30
12	12	21	25	30	30

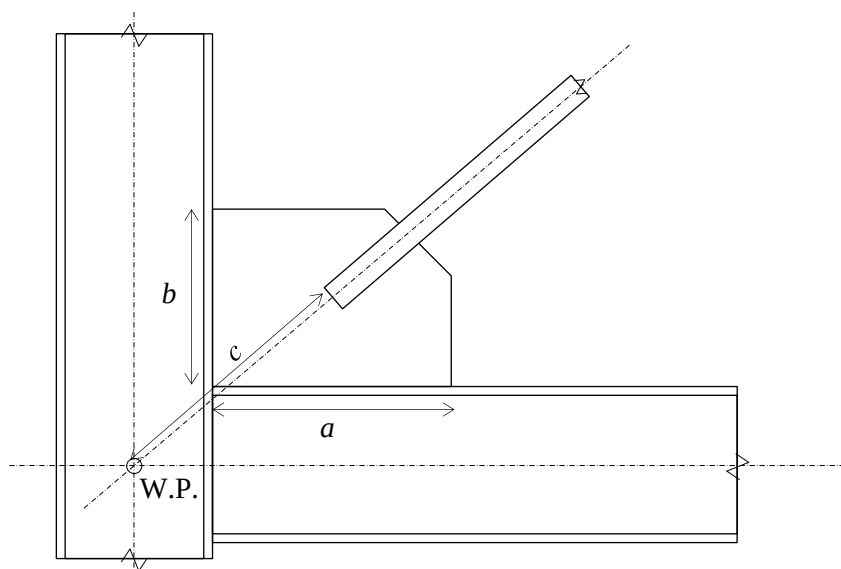


Figure 21. An illustration regarding the gusset plate dimensions where a and b are the horizontal and vertical dimensions on beam and column faces, respectively, and c is the distance from the work point of the connection to the brace end

Table- 5. Gusset plate dimensions of the SDC D_{max} archetypes (see Figure 21 for dimensions a , b and c)

Model	S	t	a	b	c	Model	S	t	a	b	c	Model	S	t	a	b	c
S12-BF1-D	1	9	1085	490	1062	S10-BF1-D	1	8	1190	460	982	S8-BF1-D	1	8	1225	460	974
	2	9	1085	490	1062		2	8	1190	460	982		2	8	1075	390	923
	3	9	1085	490	1062		3	8	1190	460	982		3	8	1075	390	923
	4	8	1310	440	933		4	8	1195	380	889		4	8	1180	380	891
	5	8	1180	380	891		5	8	1195	380	889		5	8	1085	360	827
	6	8	1180	380	891		6	8	1195	380	889		6	6	1105	370	806
	7	8	1215	380	886		7	8	1125	350	813		7	7	940	280	704
	8	8	1100	350	817		8	6	1145	360	791		8	7	845	270	624
	9	6	1140	370	800		9	7	920	280	706	S8-BF2-D	1	8	720	550	733
	10	6	1155	360	790		10	7	855	260	615		2	8	720	550	733
	11	7	930	280	705	S10-BF2-D	1	10	675	520	772		3	8	720	550	733
	12	7	835	270	625		2	10	675	520	772		4	6	775	480	618
S12-BF2-D	1	12	615	580	899		3	8	700	560	754		5	7	605	360	565
	2	12	615	580	899		4	6	790	480	613		6	7	540	330	526
	3	12	615	580	899		5	6	790	480	613		7	7	560	290	482
	4	8	760	450	646		6	7	620	360	560	S8-BF3-D	8	6	455	260	393
	5	6	775	480	618		7	7	650	350	543		1	12	440	580	687
	6	6	775	480	618		8	7	585	320	505		2	12	440	580	687
	7	6	740	430	579		9	7	540	290	486		3	12	440	580	687
	8	7	625	350	550	S10-BF3-D	10	6	485	260	385		4	8	410	410	477
	9	7	560	320	511		1	10	425	510	596		5	8	400	410	469
	10	7	605	320	500		2	10	425	510	596	S6-BF1-D	6	7	355	370	440
	11	6	525	280	446		3	10	425	510	596		7	6	385	330	367
	12	6	455	260	393		4	8	455	430	476		8	7	300	270	323
S12-BF3-D	1	12	260	580	869		5	6	500	470	474		1	8	1150	380	896
	2	12	260	580	869	S4-BF1-D	6	8	425	400	460		2	8	1150	380	896
	3	12	260	580	865		7	7	430	380	419		3	8	1150	380	896
	4	9	455	450	506		8	6	375	340	380		4	8	1125	350	813
	5	9	455	450	506		9	6	340	310	363		5	6	1115	370	749
	6	9	455	450	506		10	6	315	260	297	S6-BF2-D	6	7	825	270	627
	7	8	425	390	452	S4-BF1-D	1	8	1230	380	883		1	7	730	520	684
	8	8	425	400	446		2	8	1115	350	815		2	7	730	520	684
	9	7	370	350	413		3	6	1105	370	751		3	8	545	390	623

Model	S	t	a	b	c	Model	S	t	a	b	c	Model	S	t	a	b	c
	10	6	385	330	367		4	7	855	260	615		4	8	570	390	613
	11	6	350	300	350	S4-BF2-D	1	6	755	430	575		5	7	495	350	564
	12	6	300	270	315		2	7	640	350	546		6	6	415	340	498
S2-BF1-D	1	6	1165	360	788		3	7	575	320	507	S6-BF3-D	1	9	445	470	531
	2	7	885	290	636		4	6	530	290	404		2	9	445	470	531
S2-BF2-D	1	7	605	320	500	S4-BF3-D	1	7	425	370	434		3	8	400	430	503
	2	6	500	290	412		2	7	425	370	434		4	7	430	380	419
S2-BF3-D	1	6	395	330	377		3	7	385	350	406		5	6	375	340	380
	2	6	340	300	340		4	6	360	290	324		6	6	330	320	348

*S: story, all dimensions are in millimeters

Table- 6. Gusset plate dimensions of the SDC C_{max} archetypes (see Figure 21 for dimensions a , b and c)

Model	S	t	a	b	c	Model	S	t	a	b	c	Model	S	t	a	b	c
S12-BF1-C	1	6	925	360	772	S10-BF1-C	1	7	805	300	747	S8-BF1-C	1	6	925	310	716
	2	7	800	300	748		2	7	805	300	747		2	7	840	270	707
	3	7	800	300	748		3	7	805	300	747		3	7	840	270	707
	4	7	870	290	721		4	7	765	270	719		4	7	870	260	696
	5	7	830	270	709		5	7	765	270	719		5	7	870	260	696
	6	7	830	270	709		6	7	765	270	719		6	6	780	240	624
	7	7	840	270	707		7	7	680	250	665		7	6	800	230	615
	8	7	840	270	707		8	6	680	250	653		8	7	670	200	543
	9	7	755	250	652		9	6	680	250	653	S8-BF2-C	1	6	570	360	531
	10	6	790	230	616		10	7	530	210	576		2	7	485	300	510
	11	6	750	210	606	S10-BF2-C	1	7	500	360	569		3	7	485	300	510
	12	7	660	200	544		2	7	500	360	569		4	6	510	290	459
S12-BF2-C	1	7	540	330	526		3	6	530	380	568		5	6	510	290	459
	2	7	540	330	526		4	7	495	300	507		6	6	470	260	443
	3	7	540	330	526		5	7	495	300	507		7	6	445	230	395
	4	7	565	330	519		6	6	475	300	479		8	5	410	220	360
	5	7	520	300	500		7	6	480	260	441	S8-BF3-C	1	6	430	450	496
	6	7	520	300	500		8	6	480	260	441		2	7	355	380	475
	7	6	520	290	456		9	5	415	230	392		3	7	355	380	475
	8	6	520	290	456		10	6	400	200	352		4	6	405	370	403

Model	S	t	a	b	c	Model	S	t	a	b	c	Model	S	t	a	b	c
S12-BF3-C	9	6	480	260	441	S10-BF3-C	1	10	395	490	597	S6-BF1-C	5	6	375	340	394
	10	7	465	230	426		2	10	395	490	597		6	7	300	280	360
	11	5	430	220	380		3	7	410	500	573		7	5	320	260	304
	12	6	380	200	356		4	7	365	360	450		8	4	255	220	255
	1	8	405	410	487		5	7	365	360	450		1	7	870	260	696
	2	8	405	410	487		6	6	355	370	432		2	7	870	260	696
	3	8	405	410	487		7	7	320	270	357		3	7	870	260	696
	4	8	420	390	461		8	6	310	270	339		4	7	820	240	633
	5	7	375	350	432		9	5	300	270	321		5	6	800	230	615
	6	7	375	350	432		10	4	265	220	251		6	6	715	220	544
	7	6	385	340	389	S4-BF1-C	1	7	880	260	695	S6-BF2-C	1	7	530	290	488
	8	6	385	340	389		2	7	880	260	695		2	7	530	290	488
	9	7	310	270	361		3	6	790	240	623		3	6	510	290	459
	10	6	320	260	327		4	6	715	210	538		4	6	495	250	429
S2-BF1-C	11	6	275	230	299	S4-BF2-C	1	6	520	290	456	S6-BF3-C	5	7	465	230	426
	12	4	255	230	263		2	6	520	290	456		6	5	410	220	360
	1	7	820	240	633		3	6	480	260	441		1	7	390	340	416
S2-BF2-C	2	6	760	240	560	S4-BF3-C	4	5	430	220	355		2	7	390	340	416
	1	6	495	250	429		1	6	385	340	389		3	6	375	340	394
S2-BF3-C	2	6	420	230	371		2	6	385	340	389		4	7	330	260	346
	1	5	310	250	315		3	6	300	270	343		5	6	320	260	327
	2	5	295	260	297		4	5	275	220	269		6	4	255	230	263

*S: story, all dimensions are in millimeters

BIBLIOGRAPHY

- [1] R. Tremblay, M.-H. Archambault, A. Filiatrault, Seismic Response of Concentrically Braced Steel Frames Made with Rectangular Hollow Bracing Members, *Journal of Structural Engineering*. 129 (2003).
<https://doi.org/10.1061/ASCE0733-94452003129:121626>.
- [2] K.D. Palmer, C.W. Roeder, D.E. Lehman, T. Okazaki, C. Shield, Experimental Performance of Steel Braced Frames Subjected to Bidirectional Loading, *Journal of Structural Engineering*. 139 (2013) 1274–1284.
[https://doi.org/10.1061/\(asce\)st.1943-541x.0000624](https://doi.org/10.1061/(asce)st.1943-541x.0000624).
- [3] B. Judy Liu, A. Astaneh-Asl, Cyclic Testing Of Simple Connections Including Effects Of Slab, *Journal Of Structural Engineering*. 130 (2000).
<https://doi.org/10.1061/~ASCE!0733-9445~2004!130:9~1371!>
- [4] Fell B V, Kanvinde A M, ASCE A M, Deierlein G G, ASCE F, Myers A T, Experimental Investigation of Inelastic Cyclic Buckling and Fracture of Steel Braces, *Journal of Structural Engineering*. 135 (2009) 19–32.
<https://doi.org/10.1061/ASCE0733-94452009135:119>.
- [5] S.C. Goel, Cyclic Post Buckling Behavior of Steel Bracing Members, in: *Stability and Ductility of Steel Structures under Cyclic Loading*, CRC Press, Boca Raton, FL, 1992: pp. 75–104.
- [6] S.C. Goel, Earthquake Resistant Design of Ductile Braced Steel Structures, in: *Stability and Ductility of Steel Structures under Cyclic Loading*, CRC Press, Boca Raton, FL, 1992: pp. 297–308.
- [7] P.C. Hsiao, D.E. Lehman, C.W. Roeder, Improved analytical model for special concentrically braced frames, *Journal of Constructional Steel Research*. 73 (2012) 80–94. <https://doi.org/10.1016/j.jcsr.2012.01.010>.
- [8] P. Uriz, F.C. Filippou, S.A. Mahin, Model for Cyclic Inelastic Buckling of Steel Braces, *Journal of Structural Engineering*. 134 (2008).
<https://doi.org/10.1061/ASCE0733-94452008134:4619>.

- [9] M.C.H. Yam, J.J.R. Cheng, Behavior and design of gusset plate connections in compression, *Journal of Constructional Steel Research*. 58 (2002) 1143–1159. [https://doi.org/10.1016/S0143-974X\(01\)00103-1](https://doi.org/10.1016/S0143-974X(01)00103-1).
- [10] R. Bjorhovde, S.K. Chakrabarti, Tests of Full-Size Gusset Plate Connections, *Journal of Structural Engineering*. 111 (1985) 667–684. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1985\)111:3\(667\)](https://doi.org/10.1061/(ASCE)0733-9445(1985)111:3(667)).
- [11] D.E. Lehman, C. Roeder, D. Herman, S. Johnson, B. Kotulka, Improved Seismic Performance of Gusset Plate Connections, *Journal of Structural Engineering*. 134 (2008) 890–901. <https://doi.org/10.1061/ASCE0733-94452008134:6890>.
- [12] J.J.R. Cheng, M.C.H. Yam, S.Z. Hu, Elastic Buckling Strength of Gusset Plate Connections, *Journal of Structural Engineering*. 120 (1994) 538–559. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1994\)120:2\(538\)](https://doi.org/10.1061/(ASCE)0733-9445(1994)120:2(538)).
- [13] Disaster and Emergency Management Agency (AFAD), Turkish Building Earthquake Code 2018, TBEC 2018. (2018).
- [14] Republic of Turkey Ministry of Public Works and Housing, Turkish Earthquake Code 2007, TEC 2007. (2007).
- [15] American Institute of Steel Construction, Specification for Structural Steel Buildings, 2010.
- [16] American Institute of Steel Construction, Seismic Provisions for Structural Steel Buildings, 2016.
- [17] American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2016.
- [18] Disaster and Emergency Management Agency (AFAD), Turkey Earthquake Risk Map, (2018).
- [19] C.W. Roeder, E.J. Lumpkin, D.E. Lehman, A balanced design procedure for special concentrically braced frame connections, *Journal of Constructional Steel Research*. 67 (2011) 1760–1772. <https://doi.org/10.1016/j.jcsr.2011.04.016>.

- [20] Republic of Turkey Ministry of Environment and Urbanisation, Turkish Specification for the Design, Calculation and Construction of Steel Structures, SDCCSS 2016. (2016).
- [21] P.-C. Hsiao, D.E. Lehman, C.W. Roeder, A model to simulate special concentrically braced frames beyond brace fracture, *Earthquake Engineering & Structural Dynamics*. 42 (2013) 183–200. <https://doi.org/10.1002/eqe.2202>.
- [22] E. Karamanci, D.G. Lignos, Computational Approach for Collapse Assessment of Concentrically Braced Frames in Seismic Regions, *Journal of Structural Engineering*. 140 (2014). [https://doi.org/10.1061/\(asce\)st.1943-541x.0001011](https://doi.org/10.1061/(asce)st.1943-541x.0001011).
- [23] D.G. Lignos, E. Karamanci, Predictive Equations for Modeling Cyclic Buckling and Fracture of Steel Braces, in: 10th International Conference on Urban Earthquake Engineering, 10th International Conference on Urban Earthquake Engineering, Tokyo, 2013.
- [24] Mazzoni S, McKenna F, Scott M H, Fenves G L, Open System for Earthquake Engineering Simulation User Command-Language Manual, (2006).
- [25] P. Bazzurro, C.A. Cornell, Seismic Hazard Analysis of Nonlinear Structures. I: Methodology, *Journal of Structural Engineering*. 120 (1994) 3320–3344. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1994\)120:11\(3320\)](https://doi.org/10.1061/(ASCE)0733-9445(1994)120:11(3320)).
- [26] P. Bazzurro, C.A. Cornell, Seismic Hazard Analysis of Nonlinear Structures. II: Applications, *Journal of Structural Engineering*. 120 (1994) 3345–3365. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1994\)120:11\(3345\)](https://doi.org/10.1061/(ASCE)0733-9445(1994)120:11(3345)).
- [27] I.N. Psycharis, D.Y. Papastamatiou, A.P. Alexandris, Parametric investigation of the stability of classical columns under harmonic and earthquake excitations, *Earthquake Engineering & Structural Dynamics*. 29 (2000) 1093–1109. [https://doi.org/10.1002/1096-9845\(200008\)29:8<1093::AID-EQE953>3.0.CO;2-S](https://doi.org/10.1002/1096-9845(200008)29:8<1093::AID-EQE953>3.0.CO;2-S).
- [28] Federal Emergency Management Agency, Recommended seismic design criteria for new steel moment-frame buildings, Washington DC, 2000.

- [29] D. Vamvatsikos, C. Allin Cornell, Incremental dynamic analysis, *Earthquake Engineering and Structural Dynamics*. 31 (2002) 491–514.
<https://doi.org/10.1002/eqe.141>.
- [30] Federal Emergency Management Agency, Quantification of Building Seismic Performance Factors, FEMA P-695. (2009). www.ATCouncil.org.
- [31] K.D. Palmer, C.W. Roeder, D.E. Lehman, T. Okazaki, C.K. Shield, J. Powell, Concentric X-braced frames with HSS bracing, *International Journal of Steel Structures*. 12 (2012) 443–459. <https://doi.org/10.1007/s13296-012-3012-8>.
- [32] E.M. Rathje, C. Dawson, J.E. Padgett, J.-P. Pinelli, D. Stanzione, A. Adair, P. Arduino, S.J. Brandenberg, T. Cockerill, C. Dey, M. Esteva, F.L. Haan, M. Hanlon, A. Kareem, L. Lowes, S. Mock, G. Mosqueda, DesignSafe: New Cyberinfrastructure for Natural Hazards Engineering, *Natural Hazards Review*. 18 (2017) 06017001. [https://doi.org/10.1061/\(asce\)nh.1527-6996.0000246](https://doi.org/10.1061/(asce)nh.1527-6996.0000246).
- [33] American Society of Civil Engineers, Seismic Evaluation and Retrofit of Existing Buildings, ASCE/SEI 41-17. (2017).
- [34] E. Kor, Y. Ozcelik, Seismic Performance Evaluation of Steel X-Braced Frames per the Turkish Building Earthquake Code 2018, in: 9th Turkish Conference on Earthquake Engineering, Istanbul, 2021.
- [35] E. Kor, Y. Ozcelik, Incremental Dynamic Analysis of Concentric X-Braced Frames Designed to the TBEC 2018, in: Eurosteel 2021, Ernst&Sohn, Sheffield, 2021.
- [36] E. Kor, Y. Ozcelik, Seismic performance assessment of concentrically braced steel frames designed per the Turkish Building Earthquake Code 2018, *Structures*. (2021).

VITA

Efecan Kor, graduated from Çerkezköy Hacı Fahri Zümbül Anatolian High School in 2013 and began the prep school of Özyeğin University, which was followed by the Bachelor of Science degree in Civil Engineering. He obtained his bachelor's degree in early 2019. He continued to the academic path by starting to the Master of Science program in Civil Engineering. In his research, the primary objective is to evaluate the seismic behavior and collapse performance of concentrically braced steel frames. Besides the research, he also took place as research assistant in the Civil Engineering Department of Özyeğin University.