SEPTEMBER 2019 M.Sc. in Civil Engineering MESUT OĞURLU

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GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCES

INVESTIGATION OF SEISMIC BEHAVIOR OF STEEL STRUCTURES USING RESPONSE SPECTRUM ANALYSIS

M.Sc. THESIS

IN

CIVIL ENGINEERING

BY

MESUT OĞURLU

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M.Sc. Thesis

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Supervisor

Assoc. Prof. Dr. Talha EKMEKYAPAR

by

Mesut OĞURLU

September 2019

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This is to certify that we have read this thesis and that in our consensus/majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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Mesut OĞURLU

ABSTRACT

INVESTIGATION OF SESISMIC BEHAVIOR OF STEEL STRUCTURES USING RESPONSE SPECTRUM ANALYSIS

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Seismic behavior of steel structures in earthquake zones should be well known. This study investigates the seismic behavior of steel structures with different types of steel bracing using response spectrum analysis. To find and search the best performance, we use several concentrically bracing types in different locations. In this study, 30 and 50 storey moment resisting frame models and structures retrofitted with different type of bracing in different locations discussed, their behavior under the dynamic loads were analyzed and their lateral displacements were compared. Data were collected and evaluated by using the Sap 2000 computer program and by using the response spectrum analysis according to TEC 2007 (Turkish Earthquake Code 2007) principles. The results show that the position and type of steel braces members significantly affect the displacement and base reactions of the structure under dynamic loads. The results of this study will assist engineers in investigating the best steel brace type and it’s positioning in structural geometry to keep lateral displacements of high-rise steel structures under dynamic loads at low levels using the response spectrum method.

Key Words: Response spectrum analysis, Steel structures, Steel braces, Seismic behavior of steel structures

ÖZET

TEPKİ SPEKTRUM YÖNTEMİ KULLANARAK ÇELİK YAPILARIN SİSMİK DAVRANIŞININ İNCELENMESİ

OĞURLU, Mesut
Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Doç. Dr. Talha EKMEKYAPAR
Eylül 2019
64 sayfa

Deprem bölgelerindeki çelik yapıların sismik davranışları iyi bilinmelidir. Bu çalışmanın amacı, tepki spektrum yöntemi kullanarak farklı tiplerde çelik çapraz elemanlarla güçlendirilmiş çelik yapıların sismik davranışlarını incelemektir. En iyi performansın elde edilmesi için birkaç merkezi çelik çapraz çeşidi yapının farklı yerlerinde konumlandırılmıştır. Bu çalışmada 30 ve 50 katlı moment aktaran çerçeveli sistem modelleri ile bu modellerin merkezi çelik çaprazlarla güçlendirilmiş modellerinin dinamik yükler altındaki davranışları incelenip yanal deplasmanları karşılaştırılmıştır. Bu araştırma yapılırken Sap 2000 bilgisayar programı kullanılarak TDY 2007 (Türk Deprem Yönetmeliği 2007) ilkelerine bağlı kalarak tepki spektrum metoduyla veriler elde edilip değerlendirilmiştir. Sonuçlar karşılaştırıldığında çelik çapraz elemanlarının konumu ve tipinin dinamik yükler altında yapının deplasman ve taban kesme kuvvetlerini önemli ölçüde etkilediğini göstermektedir. Bu çalışmanın sonuçları tepki spektrum yöntemi kullanılarak yüksek katlı çelik yapıların dinamik yükler altında yanal deplasmanlarını düşük seviyelerde tutmak için en iyi çelik çapraz çeşidini ve yapı geometrisindeki konumlandırılmasını araştırmada mühendislere yardımcı olacaktır.

Anahtar Kelimeler: Tepki spektrum analizi, Çelik yapılar, Çelik çaprazlar, Çelik yapı sismik davranışları

This thesis is dedicated to my wife for her love,

endless support and encouragement.

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LIST OF SYMBOLS

M	The mass matrix
C	The damping matrix
f	The storey force vector
C	is a viscous damping matrix
K	The static stiffness matrix for the system of structural elements
$\boldsymbol{S_{A\,max}}$	Maximum spectral acceleration of the earthquake
$\boldsymbol{S_{V\,max}}$	Maximum speed spectral of the earthquake
$\boldsymbol{A}(\boldsymbol{T})$	The spectral acceleration coefficient
$\boldsymbol{S_{ae}}(T)$	The elastic spectral acceleration
$\boldsymbol{A_0}$	The effective ground acceleration coefficient
$\boldsymbol{S}(\boldsymbol{T})$	The spectrum coefficient
$\boldsymbol{T_A}$	Spectrum characteristic periods
T	The building natural period
$\boldsymbol{R_a}(\boldsymbol{T})$	Seismic load reduction factor
R	Structural system behavior factor
$\boldsymbol{I}$	The building importance factor

LIST OF ABBREVIATIONS

MRF	Moment Resisting Frame
CBF	Concentrically Braced Frame
EBF	Eccentrically Braced Frame
OCBF	Ordinary Concentrically Braced Frame
SCBF	Special Concentrically Braced Frame
RS	Response Spectrum
CQC	Complete Quadratic Combination
SRSS	Square Root of the Sum of their Squares
TEC	Turkish Earthquake Code
MDOF	Multi Degree of Freedom
SDOF	Single Degree of Freedom

CHAPTER 1

INTRODUCTION

1.1. General Introduction

With the developing technology, the number of high-rise steel structures is increasing day by day. High-rise steel structures are exposed to horizontal and vertical loads. Especially the seismic behavior of high-rise steel structures in earthquake zones under horizontal loads should be well known.

In order to design high-rise steel structures with sufficient safety in seismic areas, it is very important to choose the most suitable structure system. The main principle of the design of earthquake resistant buildings according to Turkish Earthquake Code 2007:

1. No damage to structural and non-structural system elements in buildings in weak earthquakes.
2. Limited and repairable damage to structural and non-structural elements in moderate earthquakes.
3. Limiting the occurrence of permanent structural damage without collapse in order to ensure life safety in high intensity earthquakes (TEC, 2007).

Due to the rigid behavior under horizontal loads and the limitation of the horizontal displacements, braced steel frame systems are commonly used in high-rise steel structures. The type of steel braces and their position in the structure significantly affect the behavior of the structure under horizontal loads.

1.2. Steel as a Building Material

The steel has been used as building material since 19th century. In recent years the use of steel in high-rise structures is increasing. Therefore, the use of steel as a building material has gained importance. The steel has some basic qualities that will make the use advantageous compared to other building materials, which are (Yardımcı, 2005):

1. High strength and the ratio of self-weight per carrying load is very small.
2. The modulus of elasticity is very high compared to that of other building materials for that reason, it exhibits a behavior consistent with dynamic load and stability problems and more economical sections are obtained for the dimensioning of bearing elements with deflection problems.
3. It has a large deformation capacity because it is ductile, therefore, it becomes very important in unexpected extraordinary load situations and earthquake regions.
4. The processing of the bearing elements in the workshops causes the construction not to be affected much more by the weather conditions during the installation phase and therefore the construction time is shortened.
5. The building elements are easy to change and can be used elsewhere when needed.

Like other building materials, steel also has some disadvantages as a building material, such as:

1. It is not flammable, but as the heat rises, there is a rapid decline in its strength and modulus of elasticity. So, precautions should be taken to protect steel structures from fire.
2. In case of contact of steel with water or other chemicals, initiates corrosion and reduces the strength of the material. Therefore, steel structures must be periodically maintained.

1.3. Steel Structures

In recent year, steel structure is used for almost every type of structure including heavy industrial buildings, high-rise buildings, infrastructure, bridges, towers, etc. As the number of storeys increases in multi-storey steel structures, the effect of horizontal loads such as earthquakes increases. For this reason, depending on the height of the building, different steel structure systems used with the choice of the designer.

Configuration of some steel structural systems for high-rise buildings shown in Figure 1.1 like Braced, moment resisting frame, frame-shear truss roof and framed tube construction as shown. The main elements for steel structures are beams, columns, ties and cross section components. All structural elements like beam, column and braces can rolled from several sections such as H, I, L, pipe, tube or box as shown in Figure 1.2.

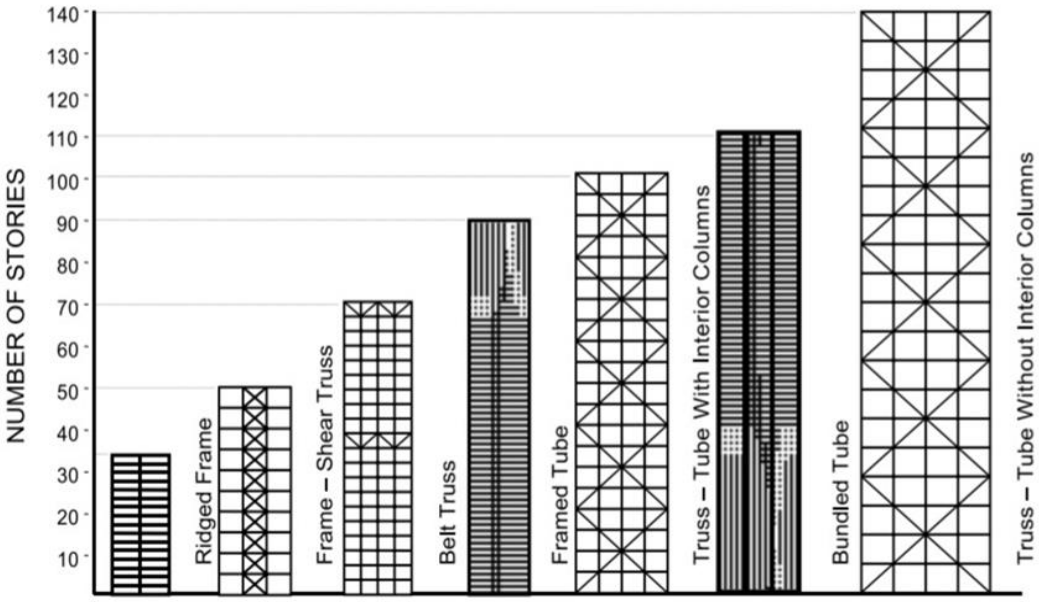


Figure 1.1 Types of steel structural systems for high-rise buildings (Khan, 2002).

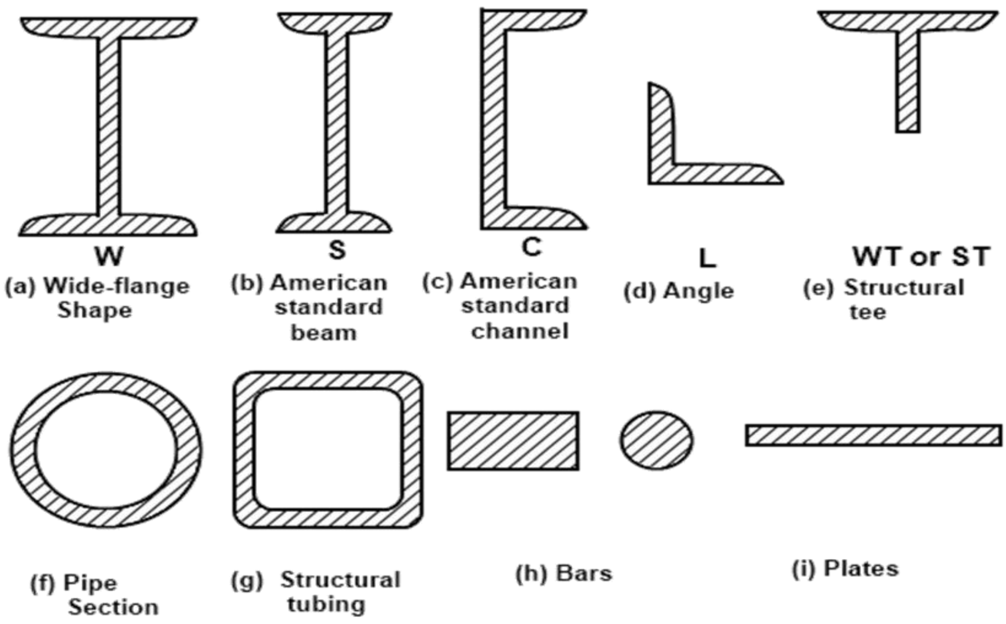


Figure 1.2 Rolled structural steel sections

Overall share of the steel construction sector in Turkey is estimated at 5%. Number of multi-storey steel structure in Turkey is quite low. However, in Istanbul and Izmir, there are several steel structures over 30 floors.

1.4. Objectives and Research Significance

Earthquake caused building damages or loss of life around the world. In recent years number of high-rise steel structures rises so earthquake effects become more significant. For that reason the selection and analyzing of structures very important. The purpose of this study is investigating seismic behavior of steel structures with different types of steel bracing using response spectrum analysis. To find and search the best performance, we use several concentrically bracing types in different locations of structures. In this study an equivalent planar geometry with a 30 and 50-storey moment frame system models with different type of bracing in different locations discussed, their behavior under the dynamic loads were analyzed and their lateral displacements were compared. Data were collected and evaluated by using the Sap 2000 computer program and by using the response spectrum analysis according to TEC 2007 (Turkish Earthquake Code 2007) principles. As a result, every detail of the lateral movements will be investigated and the methods of controlling this behavior will be developed from the point of view of civil engineering. Result of the thesis studies will help engineers about the using for response spectrum method and significant of bracing of high-rise steel structures.

1.5. Thesis Layout

This thesis consists of five chapters:

In Chapter 1, the aim and significant of the thesis are summarized.

In Chapter 2, the literature review for steel bracing systems and response spectrum was presented. In addition to the previous studies made about steel bracing systems and response spectrum analysis.

In Chapter 3, in methodology, explain methods of structural analysis using by computer program. Describe the analytical model of existing structures and approach response spectrum analysis.

In Chapter 4, discuss the results of the structures analyzed in the Sap 2000 program. As a result of the analysis of all structures, displacements in X and Y directions, periods and base reactions compared graphically.

In Chapter 5, conclusions are presented according to the findings in the analysis results for further research.

CHAPTER 2

LITERATURE RIVIEW

2.1. Moment Resisting Frame

Moment Resisting Frame (MRF) consist of columns and beams rigidly connected to each other. That permits moment transfer across the joints. Horizontal loads such as earthquakes and the live and dead loads in the frame system are carried by columns and beams. A typical moment resistive beam-to-column steel framed connection includes transferring horizontal loads along the beam flanges directly to the column flanges using angles and column web hardening plates. A typical MRF example is shown in Figure 2.1.

Figure 2.1 Moment-Resisting Frame (Popov, 1988)

A view of a frame analysis using moment-resisting frame (MRF) as shown in Figure 2.2

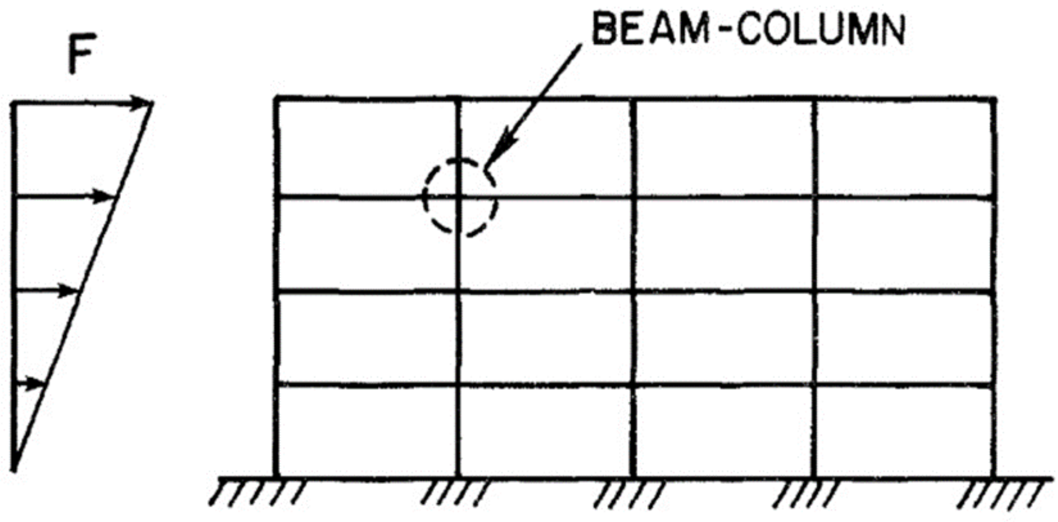


Figure 2.2 Elevation view of a Moment Resisting Frame

In a MRF during a major earthquake, the energy distribution is mainly achieved by the non-elastic effect in the beam-column joints, and if the beams and columns are proportionate to meet the design concept of the strong column-weak beam, these frames usually have a significant ductility, and appropriate details are applied in beam-column connections (Kasai and Popov, 1986). In Figure 2.3 a typical MRF beam-column connection is shown.

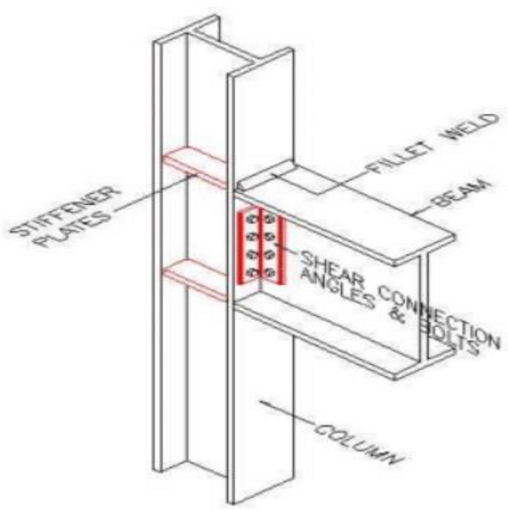


Figure 2.3 Typical MRF beam-column connection

Shear and axial forces in beams are usually much smaller than the angle of repose is small compared to the bending moment and should be considered in the design. The beams typically exhibit high bending moments, with the maximum moments occurring at the ends of the members (Bruneau et al., 1998). The internal forces in a MRF under lateral loads shown in Figure 2.4.

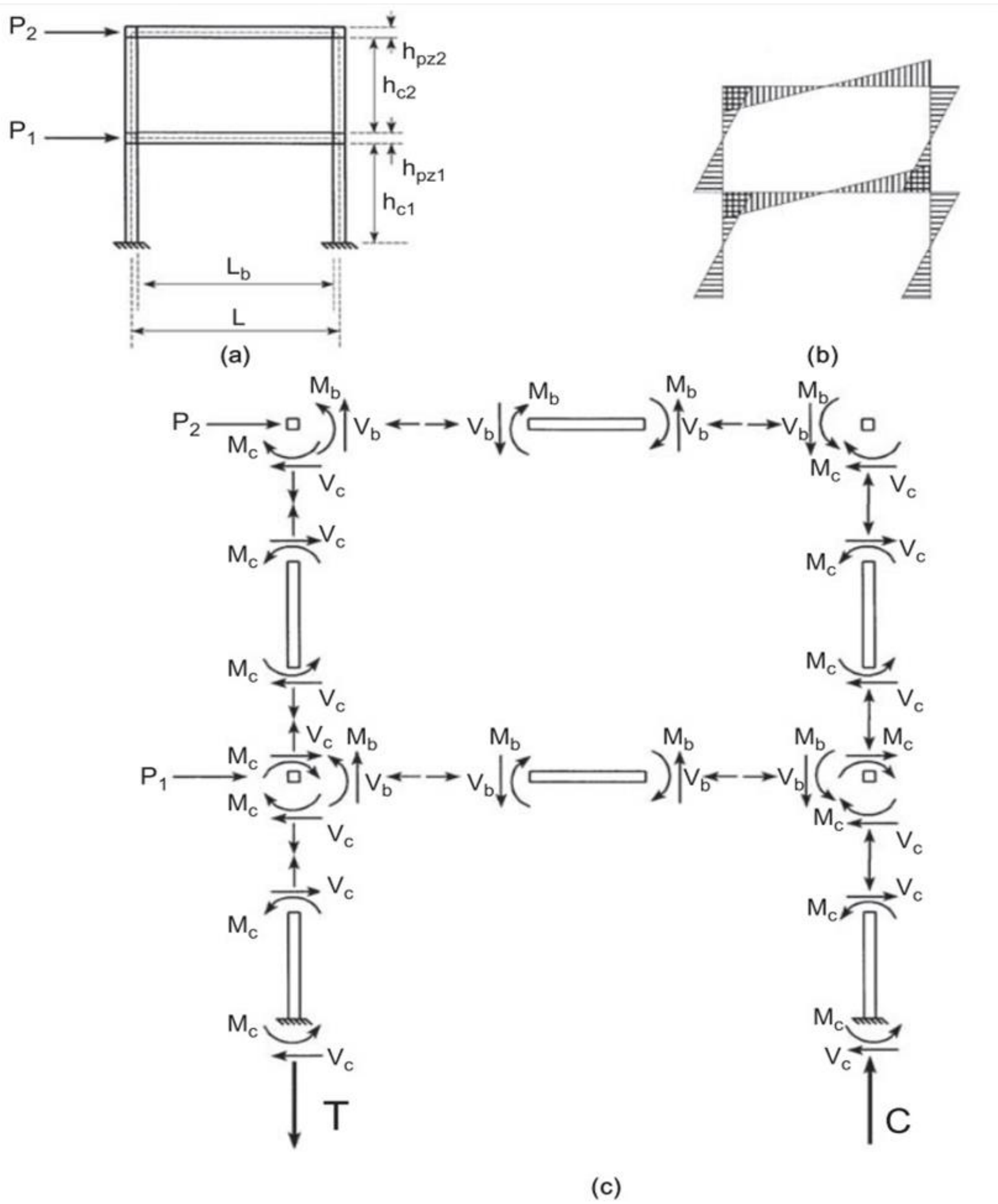


Figure 2.4 MRF: (a) geometry of MRF, (b) moment diagram of the structure under lateral load and (c) member forces on beams, columns, and panel zones (Bruneau et al., 1998)

2.2. Steel Braced Frame

In the earthquake areas moment resisting frame structures are not effective against horizontal loads. Braces are increasing the horizontal load bearing capacity of steel structures. The most economical way to increase lateral stability is using braces. Due to the rigid behavior under horizontal loads and the limitation of the horizontal displacements, braced steel frame systems are commonly used in high-rise steel structures. The type of steel braces and their position in the structure significantly affect the behavior of the structure under horizontal loads.

A typical Steel Braced Frame example is shown in Figure 2.5

Figure 2.5 Steel Braced Frame

Steel braced frames allow getting a huge increment of firmness with a slight increment in weight, so it's considered as healthy solutions for existing structures that suffer from weakness in the lateral load. The bracing improve in rigidity and stabilization of structures under the horizontal loads, just as to decrease the impact of lateral displacement significantly (Rashid, 2015). Typical arrangements for braced frames as shown in Fıgure 2.6.

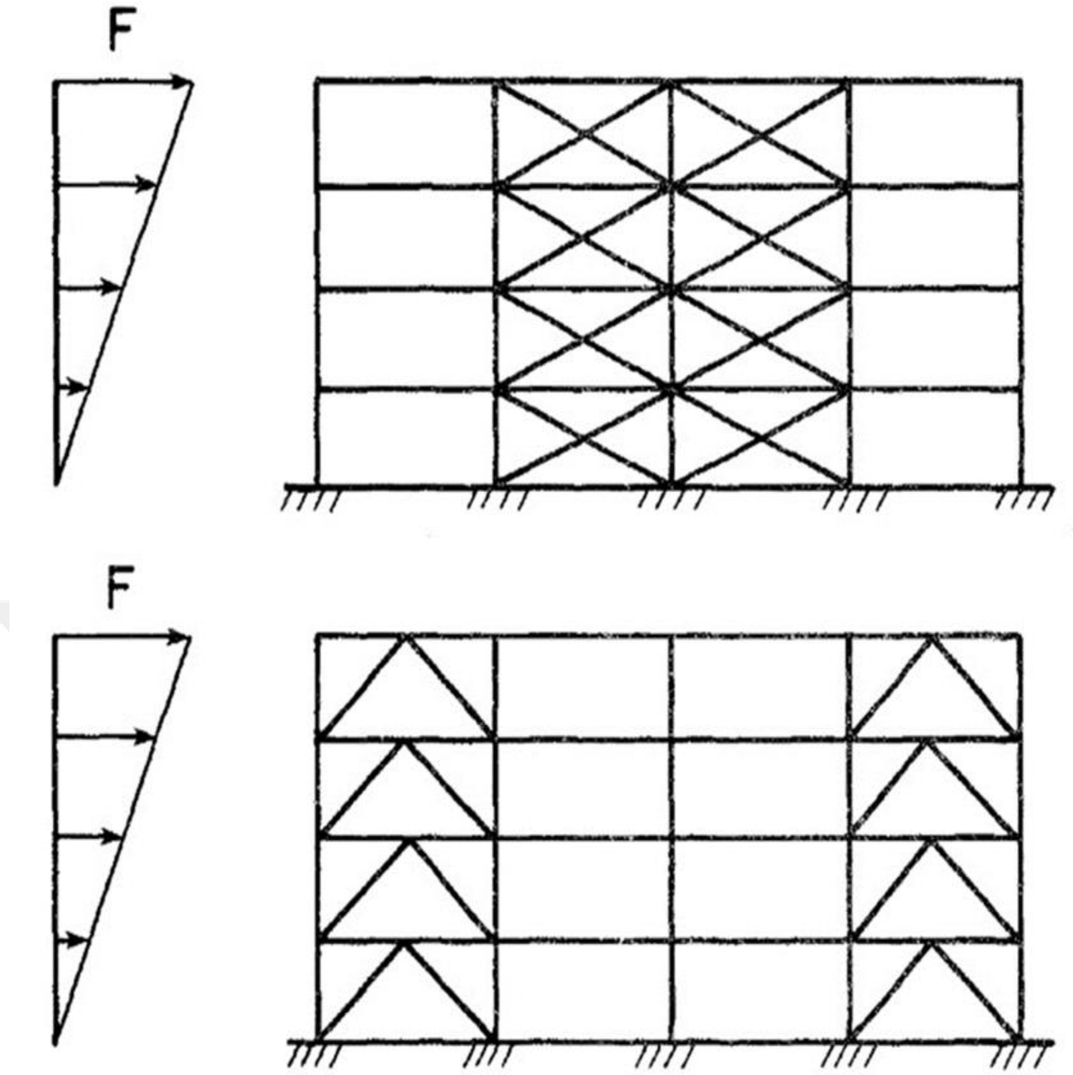


Figure 2.6 Typical arrangements for braced frames (Kasai and Popov, 1986)

Under earthquakes load, braced frames are relied upon to yield and scatter energy through postbuckling hysteretic behavior of their bracing members. For drift in one specific direction, this is accomplished by buckling of the braces in compression, followed by yielding of the braces in tension, as schematically showed in Figure 2.7 To avoid an earthquake, the braces must be able to maintain large non-elastic displacement reversals without significant loss of strength and stiffness (Bruneau et al., 1998). After earthquake demonstration of buckling of brace is shown in Figure 2.8.

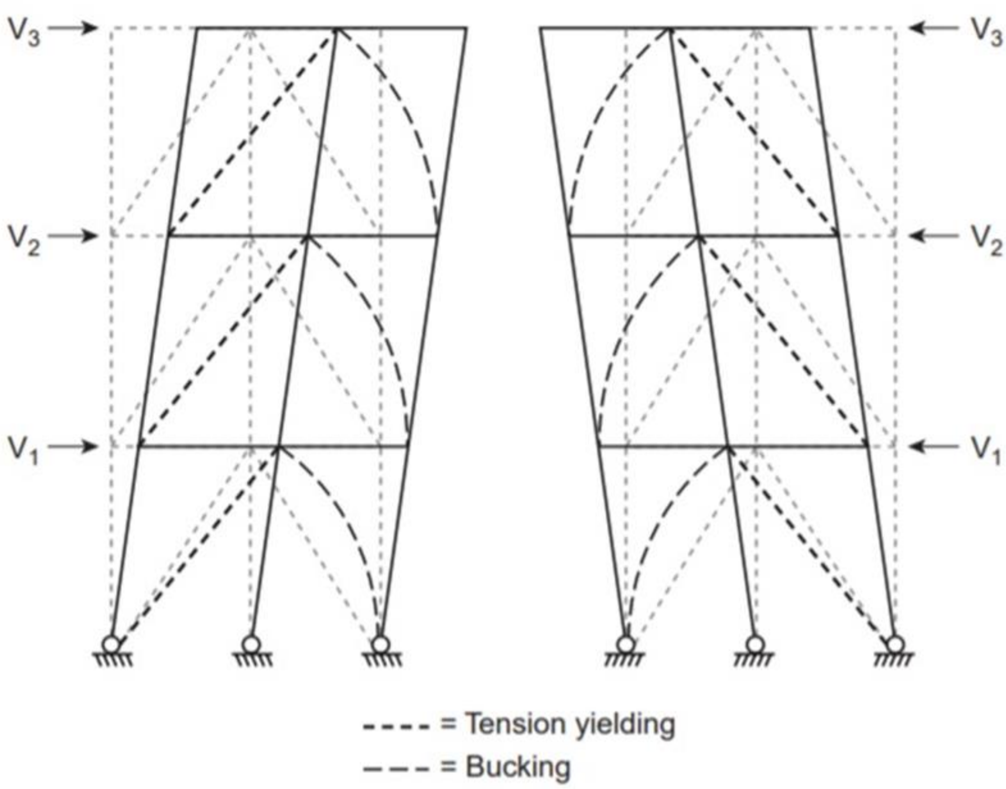


Figure 2.7 Schematic of braced frames inelastic behavior (Bruneau et al., 1998)

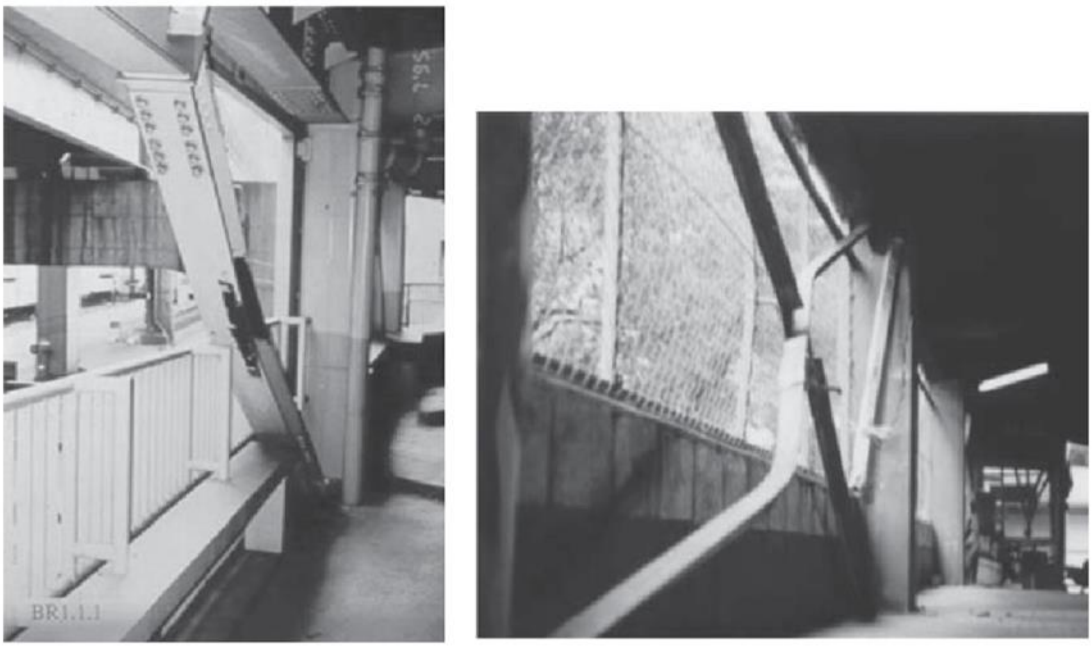


Figure 2.8 After earthquake demonstration of buckling of brace (Bruneau et al., 1998)

2.2.1. Concentric Braced Frames

Concentric Braced Frames (CBFs) are used as utmost effective earthquake load carrier system for to buildings. CBF can be designed and constructed as high, medium and normal ductility level. The main advantages of the CBFs are that they provide stiffness and strength requirements very easily with minimum weight. Although CBF easily meet the rigidity and strength requirements required for the design, nonlinear behavior under severe earthquake ground motions is not the most preferred carrier system among the expected systems (Günday, 2017).

The CBF maintains a horizontal load in the elastic spacing mainly during axial forces in the members for truss systems. Braced frames in the non-elastic range may be related to the bending of the frame elements, yet it can be predicted that the inelastic shift will function as a base as a result of the divergent axial deformation and with the exception of some unforeseen formations. The flexibility ratio of the braced elements is an important parameter regulation factor in braced frame structures. Estimation of excessive strength, ductility and response modifying factors of chevron-type concentric braced frames with different layers and lengths of propagation, using pushover analyses (Rashid, 2015).

With the help of transverse support members based on lateral forces, the system can achieve high rigidity using higher internal axial movements and relatively lower bending movements. It is a diagonal support that generates the main units that form the lateral stiffness. The brackets may be made of different shapes and sections, as follows: I-shaped sections, circular or rectangular tubes, double-angled, solid T-shaped sections, single angles, channels and tension only rods and angles connected together to form a T-shaped section. Generally, the reinforcements are joined by gusset plates bolted or welded to other members of the framing system. The focus in the CBF design approach is generally directed to the distribution of energy in the reinforcements, so that in accordance with the design, the connection remains elastic at all stages during loading (Nourbakhsh, 2011).

In MRF systems, beams, columns and crosses are put in ordered to form a vertical cage. Horizontal forces are counteracted by cage movement. Ductility is achieved by non-elastic movement in the diagonals. Elastic horizontal stiffness of MRF systems is considerably higher than Moment resisting frame systems. In order for the MRF system to exhibit ductile behavior, the diagonal elements must be able to perform large deformation during repeated inelastic loading without significantly losing their strength and rigidity (Çırpan, 2017).

In a review of experimental work on the non-elastic response of diagonal steel reinforcement elements, it has been proposed to cyclic inelastic loading to collect data for the seismic design of concentric reinforced steel structures when a ductile reaction is required under earthquakes. The different parameters observed are the bending resistance of the bracing elements, the post-stress compressive strength after the support at different levels of ductility, the maximum tensile strength including the tensile hardening effects and the lateral deformations bracing under stress. Equations are established for each of these parameters (Tremblay, 2002).

CBFs divided into two primary groups, special concentrically braced systems (SCBF) and ordinary concentrically braced systems (OCBF). SCBF is often used in areas with high earthquake risk; the design purpose of CBFs provides satisfactory ductility. OCBF does not have comprehensive qualifications related to elements or connections, and is often used in areas with low seismic risk. Seismic design needs for supported frameworks, observing the flaws of the OCBF, have been modified and the idea of special concentric frames has been created (Sabelli et al., 2003).

It has been perceived for quite a while that these frameworks have natural commitments in the post-elastic range, since it is known that most of the efficiency and consequently the ability to absorb energy concentrate on the support members, ranging from tension and compression. The tension yield of the joint results in reduced compression capacity and stiffness of the braces in each successive cycle. Causes ever-increasing deviations and possible failure. The above-mentioned weakness, which indicates that this system was previously described arbitrarily in the Uniform Building Code, required that the connections of the elements and connections be designed for forces greater than 25% greater than those obtained from the seismic analysis code. Due to the increased use of braced frames in larger, heavier, and more important structures, improved design requirements were felt to be necessary (Rashid, 2015).

The nonlinear dynamic analysis is performed in a triple CBF and, in the first storey, a drift concentration is confirmed to be predictable, especially when the CBF is subjected to a series of large ground motions with a chance of doubling. Two conditions are measured: ideally fixed gravity columns and base-fixed gravity columns. It is well known that when the gravity columns are substantially fixed to the base, the shear concentration can be significantly alleviated. Nonlinear time-history analysis was performed to count the stiffness / strength demands of gravity columns to confirm theoretical explanations and avoid drift concentration (Ji et al., 2009).

Inventive structure ideas can be utilized with the perfect opportunity to select the type of support system that is connected to an efficient and economical support system for construction, offering the building system engineer. However, a suitable depth for the support cages can often be an important consideration. Scissors vary as stiffness, square of depth and a preliminary guide for a reasonably efficient supporting system. There is always the possibility of disrupting architectural planning because a multi-storey depth-of-field and optimum height-width truss bracing has therefore forced the structural engineer to use less efficient support systems in construction (Mccormick et al., 2007).

Different types of concentric braces can also be ordered according to their configuration. Examples of concentric braces are type V, type X, type K, and so on. It is possible to use I, L profiles, tubular, square or rectangular box profiles and multipart sections for the diagonal elements. In Figure 2.9 different types of CBFs are shown.

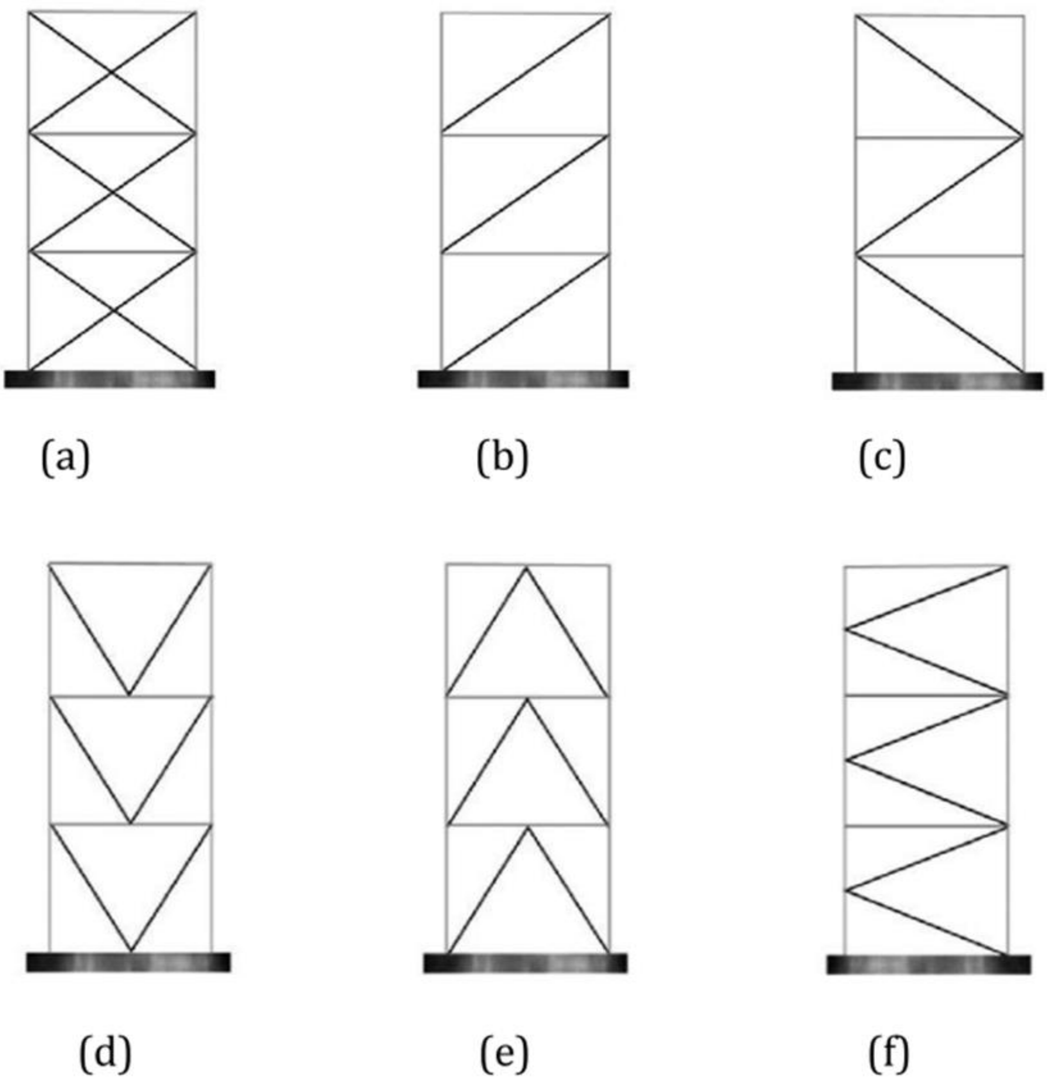


Figure 2.9 Example of concentrically braced frame: (a) X-braced; (b) diagonally braced; (c) alternative diagonally braced; (d) V-braced; (e) inverted V-braced; and (f) K-braced (Tejesh and Pasha, 2018)

An X-braced frame (Figure 2.9 (a)) has fasteners that are tensioned in an equal amount of loading direction, if they are sufficiently large to be renewed before the beam, the beam becomes short and ductility may develop.

Diagonal support (Figures 2.9 (b) and (c)) react to the loading path. Configuration (b) is likely to be substantially weak with the elastic in the path, since configuration (c) will be as weak as the extra elastic on the storey with the compression brace, which is most important for soft-storey formations.

The V-braced supply of Figures 2.9 (d) and (e) experiences the ability of the buckling compression braces to be substantially less than the ability to dispense with the tensile braces. Therefore, inevitably with the exception of the equilibrium loads on the horizontal beams, their capability must withstand the bending of the level piece. They limit the amount of elastic from which the bonds can accumulate and are thus taken to be completely ductile. Hysteretic presentation of the V-braced system has been improved in cases where the horizontal joints have high flexural strength that can emerge from equilibrium loads (Tejesh and Pasha, 2018).

2.2.2. Eccentrically Braced Frames

Eccentrically Braced Frames (EBFs) are a lateral load-resistant system for steel constructions that may be well-respected a hybrid between (MRFs) and (CBFs). EBFs are popular systems because they combine the rigidity advantages of the braced frame with flexible energy performance and flexibility advantages that can be distributed over ductile steel MRFs. EBFs are defined as a supported frame such that at least one end of each support frame enters only one beam and at least one fixed soft connection is formed at each beam. For framed lateral force-resistant systems, it would represent the entire sequence between the ends of the moment frame, which essentially depends on the bending and shear strength of the frame elements, the continuity of the normal square frame, the eccentric frame, which is essentially dependent on the axial force of the diagonal elements (Rashid, 2015).

The distinctive feature of an EBF is that at least one end each support is linked in such a way that the connecting force is transmitted to a column by sliding and bending in a beam part connected to either a connecting part or a column. The connection lengths in Figure 2.10 are assimilated by the letter e. Despite the fact that eccentric bracing has long been known for its wind resistance, its application to the seismic resistant structure is very new. The bracing members in EBFs provide high elastic stiffness, which allows the CBFs to have high characteristics and economically meet code deflection requirements. Nevertheless, with the extremely severe seismic loading conditions, well-designed and detailed EBFs produce the ductility and energy dissipation capacity of MRFs (Popov and Engelhardt, 1988).

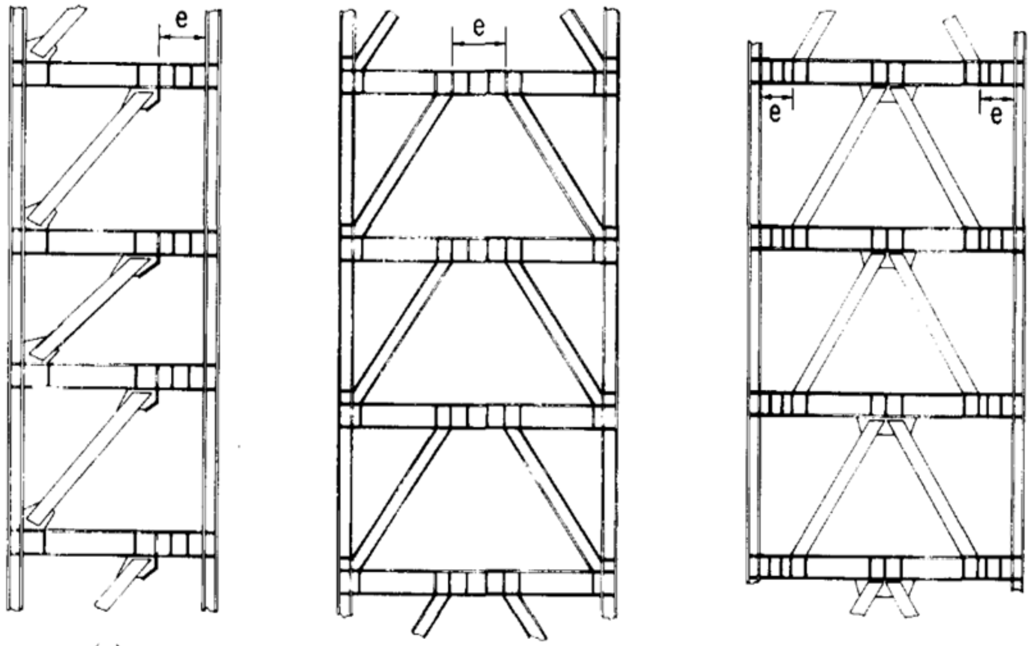


Figure 2.10 Typical bracing arrangements for EBFs. (Popov and Engelhardt, 1988)

An EBF is a frame system in which the axial forces induced by the brace are transmitted by sliding and bending a small portion of a beam onto a column or other beam. Typical EBF configurations are shown in Figure 2.11. The basic beam section is called "link" and is indicated by *e* in the figure. Connections in EBFs serve as structural fuses to distribute earthquake-induced energy in a stable manner in a building. In practical applications, horizontal joints have been widely used; see Figure 2.12 for examples. Figure 2.11 (e) shows an EBF with no connection on each floor. The connections in Figures 2.11 (b) and c are connected to the columns. (Bruneau et al., 1998)

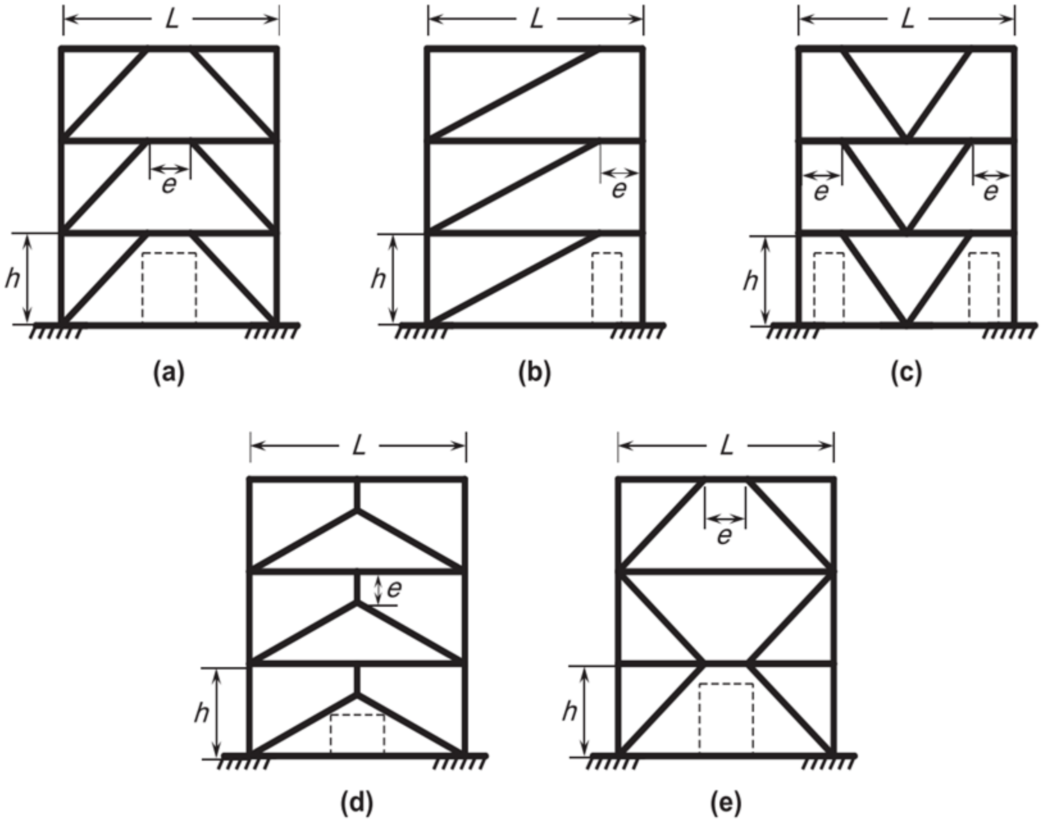


Figure 2.11 Typical EBF configurations. (Bruneau et al., 1998)

(a) EBF with interior links **(b)** EBF with exterior links

Figure 2.12 Examples of EBF construction.

Yield mechanism of EBF as illustrated in Figure 2.13. Figure 2.14 shows a comparison of the expected plasticity mechanisms between SCBF and EBF. In the SCBF, the bracket is designed and described in detail as a structural fuse. However, for EBF, the chain links need to be properly designed and detailed to achieve sufficient strength and ductility. All other structural components are proportioned according to capacity design principles to maintain basic flexibility during design earthquakes. (Bruneau et al., 1998)

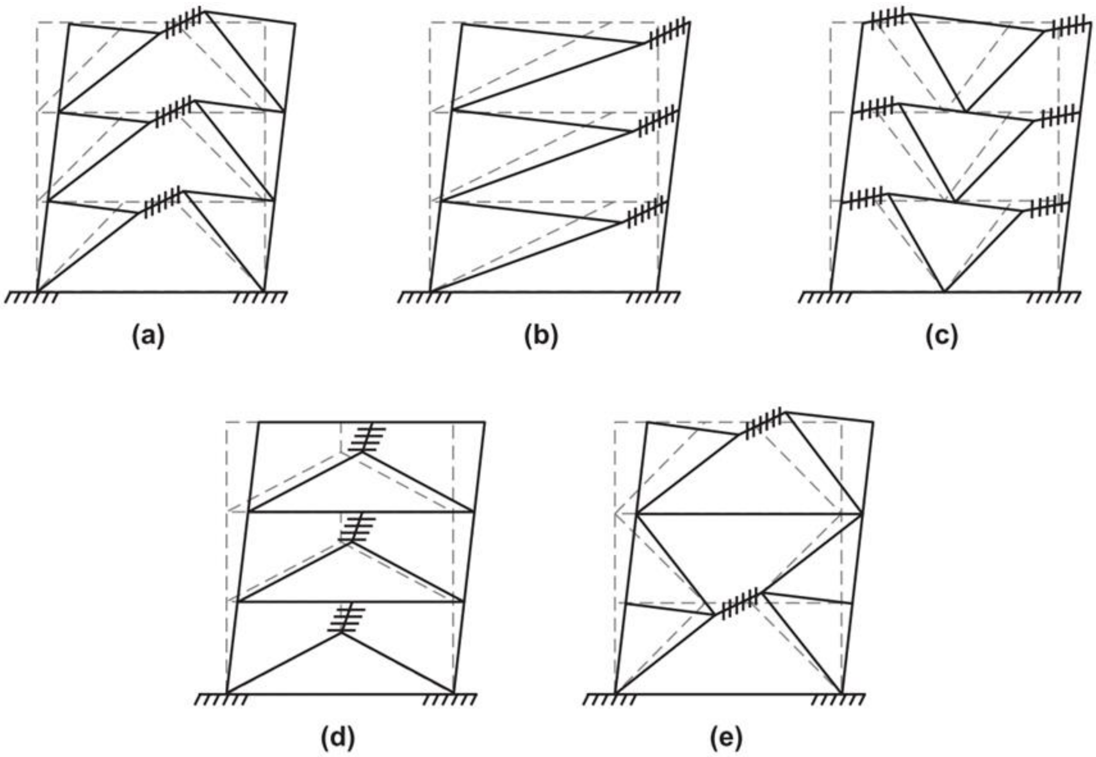


Figure 2.13 Yield mechanism of EBF

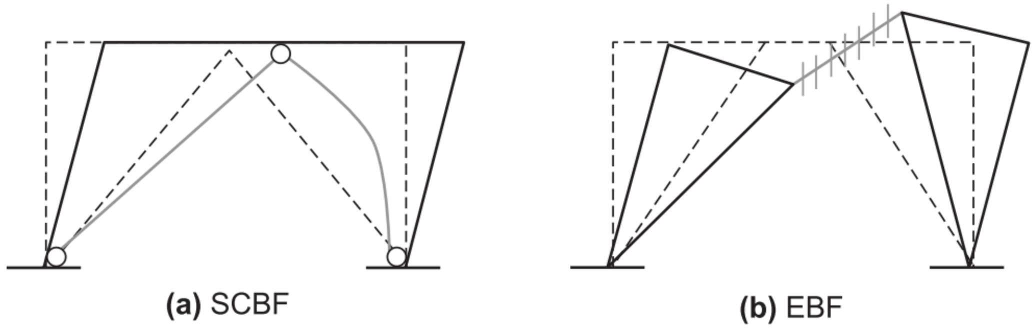


Figure 2.14 Deformed configuration of SCBF and EBF (Bruneau et al., 1998)

2.3. Previous Studies

Günday (2007) investigated the effects of strengthening on structure dynamic behavior of steel structures with steel bracing. He compared MRF and their reinforced with X type steel bracing. The result of him study show effect of bracing on rigidity of structures.

Kul (2010) analyzed and compared different type of steel structures such as MRF and various types of steel braced frame. He investigated the behavior of the models having different steel brace elements under horizontal load.

Khatib et al. (1988) applied optimization techniques the design of frames with brackets to improve seismic responses. They investigated behavior of MRF, X braced frame and V braced frame. Each of these approaches has been researched and each has advantages in certain situations.

Siddiqi et al. (2014) selected six storey building and analyzed under horizontal loads. They investigated five different types of braced systems for use in high-rise buildings to provide lateral stiffness, and as a result optimized design with lower structural weight and less lateral displacement has been introduced.

Çavdar (2017) compared different types of braces on a selected three different building. She analyzed and compared the behavior of MRF and V braces under earthquakes load and compare periods, displacement and base reactions of each structures. As a result of the study, she was obtained that the V crossed element had better earthquake performance.

Tansel (2010) modeled a building under the same conditions as a MRF, with a CBF and EBF. He analyzed the structures according to the TEC 2007 and with the principles of the capacity design principles. And he compared and interpreted the results.

Rashid (2015) investigated seismic behavior of 10, 20, 30 storey steel circular and rectangular structures with CBF and EBF and searched for the best model to resist deformation in structural members. As a result he obtains different performance for the structures under horizontal loads.

CHAPTER 3

METHODOLOGY

3.1. Introductıon

As it is known, earthquakes cause changes in structure and time depending on time. In turn, internal forces vary depending on time. The purpose of earthquake resistant construction design is to limit possible damage caused by these internal forces. The main principle of the design of earthquake resistant buildings according to Turkish Earthquake Code 2007:

No harm to the structural and non-structural elements of the system in buildings during weak earthquakes, limited and repairable damage to structural and non-structural elements in moderate earthquakes, limiting the occurrence of permanent structural damage without collapse in order to ensure life safety in high intensity earthquakes. The design earthquake to be based on the design of buildings, the probability of exceeding over a period of 50 years is 10 % (TEC, 2007).

Civil engineering structures are designed to withstand static loads. Most of all, the effect of dynamic loads on the structure is not taken into account. This property of ignoring dynamic forces is sometimes catastrophic, especially in the case of an earthquake. The seismic load, one of the most important dynamic loads, should be taken into consideration in the structural analysis (Gottala et al., 2015).

The methods for dynamic analysis of structures are: equivalent static lateral force method, response spectrum (RS) method and time history function method. The equivalent static lateral force method is acceptable for normal, low and middle-rise buildings not available for all type buildings. Time history function methods requires huge amount of calculations and the analysis has to be repeated different types of ground motion with these method. RS method is progress a method of procedure the equivalent static lateral load analysis. RS method provides a simple method for finding design forces that affect the structural elements under the seismic load. Therefore, engineer prefer RS method for dynamic analysis.

3.2. Equivalent Static Lateral Load Analysis

The equivalent static method is a preliminary method for finding subsequent loads that are applied to the build. The basic shear force is calculated using the seismic weight of the building and the seismic horizontal acceleration coefficient. Utilizing the code base equation, this base shear the stature appropriation along the structure. This method is usually used in relatively regular buildings, although some modifications have been implemented to achieve good results even for high-rise buildings. Compared with other methods, the biggest advantage of this method is that it is simple to calculate. Therefore, it is the most commonly used design procedure (Reddy and Rao, 2016).

Seismic loads have different effects, which are not proportional to the exposed area of the building, but proportional to the mass distribution of the building, higher than the specific level being considered. Earthquakes indirectly apply loads to the structure. The ground is displaced and they are subject to sudden movements due to the structure being connected to the ground. These movements produce acceleration in the building, resulting in different movements of the building level. These deformations result in horizontal shear. The base shear force is an estimate of the maximum expected lateral force that occurs due to seismic action at the bottom of the structure. The calculation of the foundation shear depends mainly on the soil conditions of the site, the potential seismic activity source, the ductility level, the total weight of the structure and its basic natural vibration period. The equivalent lateral force program determines the base shear based on empirical formulas and then distributes it to each floor as it tries to balance the seismic loads. There are many equations for determining the underlying cuts, which can be found in the code. Each formula calculates the seismic force as part of the weight of the building (Sokoli and Yardim, 2012).

Obviously, the earthquake does not exert an external force on the ground, but an inertial force that changes with time, and these inertial forces are replaced by the equivalent static force applied to each floor as shown in Figure 3.1. The equivalent lateral force procedure is based on the fact that the first mode is dominant for earthquakes and some structures undergo some inelastic deformation during an earthquake. When excited by an earthquake, the free body map of the structure is considered to be a simple vertical cantilever with lumped mass. In this method, the design seismic force is determined by linear elastic static analysis of the structure. In this method, the total equivalent seismic load is distributed as static lateral loads to floor levels. This distribution is triangle and the distributed equivalent load is applied at the mass center at each floor level (Sokoli and Yardim, 2012).

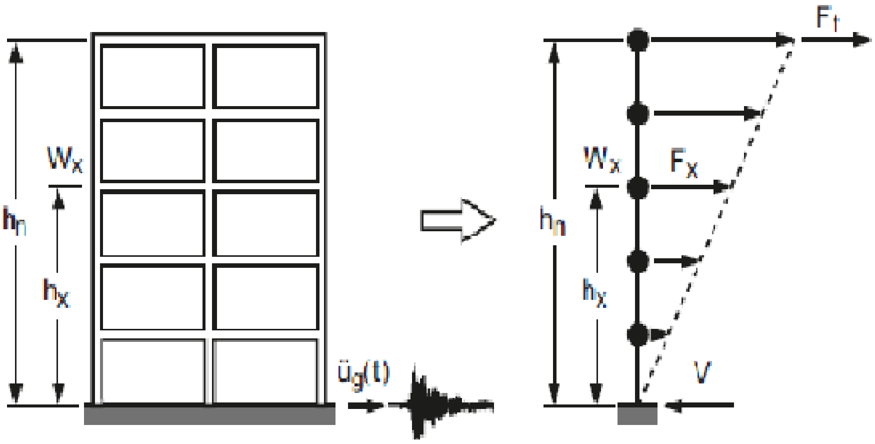


Figure 3.1 static lateral seismic load distributions.

3.3. Time History Analysis

Time history analysis is a dynamic analysis that is performed by taking the data on incremental steps as a function of acceleration, force, moment or displacement. The closer the time interval, the more accurate the solution will be. The background of this time history analysis depends on the feature values generated for the structure based on the response to the time history. Compared to response spectrum analysis, considerations are more realistic. Most suitable for very long or very high structures (Reddy and Rao, 2016).

Time history analysis is a step-by-step analysis of the dynamic behavior of a structure over a given load, which may change over time. By means of chronological analysis, the seismic response of a structure under the dynamic load of a representative earthquake is determined (Katti and Balapgol, 2014).

In special cases, prerecorded or artificially simulated ground movements can be used for linear or nonlinear analysis of structures and structures similar to buildings in the time domain.

At least three acceleration records, pre-recorded or generated, shall be used for time domain seismic analysis and the unfavorable response quantities shall be considered in the design according to the following specifications: The duration of the intense acceleration protocol movements where the acceleration envelopes should not be less than ± 0.05 g should not be less than five times the first natural vibration period of the building nor less than 15 seconds.

Spectral acceleration values recalculated for each recorded or simulated acceleration record with 5% damping ratio shall not be less than 90% of the spectral acceleration coefficient, A(T), defined in 6.4 times the acceleration of gravity, g , for the whole period range. In the case where linear elastic analysis is performed in the time domain, spectral acceleration values to be considered for the reduced ground motion shall be calculated by Eq. (3.1).

In the case where nonlinear analysis is performed in the time domain, dynamic stress-strain relationships defining the nonlinear behavior of the structural system shall be determined by theoretically or experimentally proven methods in line with the overall philosophy of this Specification (TEC, 2007).

$$S_{ae}(T) = \frac{A(T)g}{R_a(T)} \tag{3.1}$$

3.4. Nonlinear Static (Pushover) Analysis

It is also known as Non-linear Static Analysis. This is a practical way to analyze in a permanent situation. Vertical loads and increasing lateral loads are used to calculate the deformation and failure modes of the structure. Pushover analysis is a seismic analysis method in which the behavior of a structure is characterized by a capacity curve that represents the relationship between the base shear force and the roof displacement (Katti and Balapgol, 2014).

The pushover static analysis can provide sufficient information about the seismic loads established by the design ground motion on the structural system and its components. The purpose of the pushover analysis is to calculate the strength and deformation requirements of the design earthquake by static unyielding analysis, and to estimate the predictable performance of the structural system by evaluating the performance of the structural system. Nonlinear static pushover analysis can be used as a technique for predicting seismic forces and deformation levels. It considers internal forces that cannot be resisted by elastic behavior in an estimated internal force rearrangement method. Pushover curve of a structure as shown in Figure 3.2 (Oğuz, 2005).

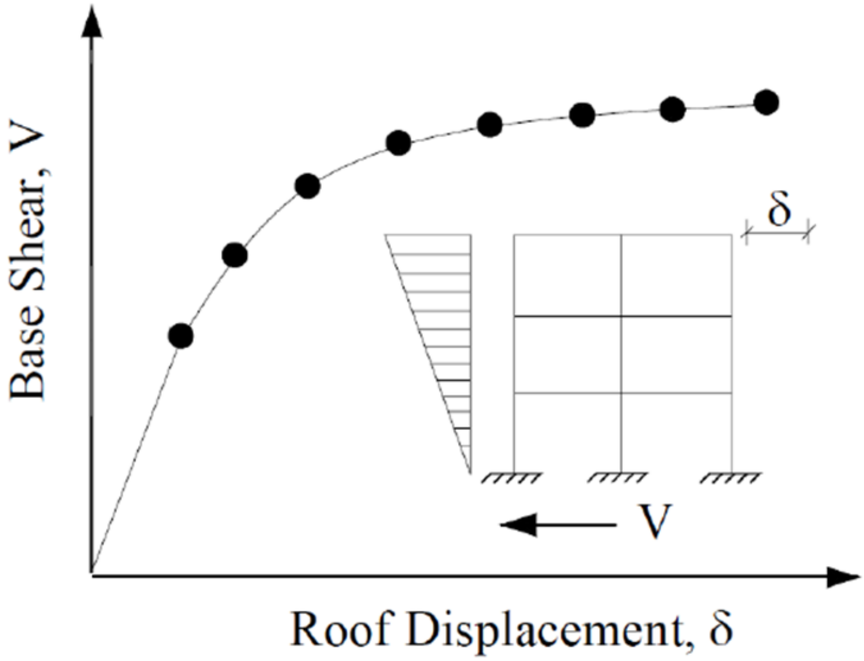


Figure 3.2 Pushover curve of a structure

The static pushover analysis method does not have a strict theoretical basis. It is primarily based on the assumption that the response of the structure is controlled by the first vibration mode and the modal shape, or by the first few vibration models that remain constant throughout the elastic and inelastic response of the structure. This provides the basis for translating dynamic problems into static problems that are theoretically flawed. In addition, the response of a multi-degree of freedom (MDOF) structure is related to the response of an equivalent single degree of freedom (SDOF) system. The previous concept is shown in Figure 3.3 (Rashid, 2015).

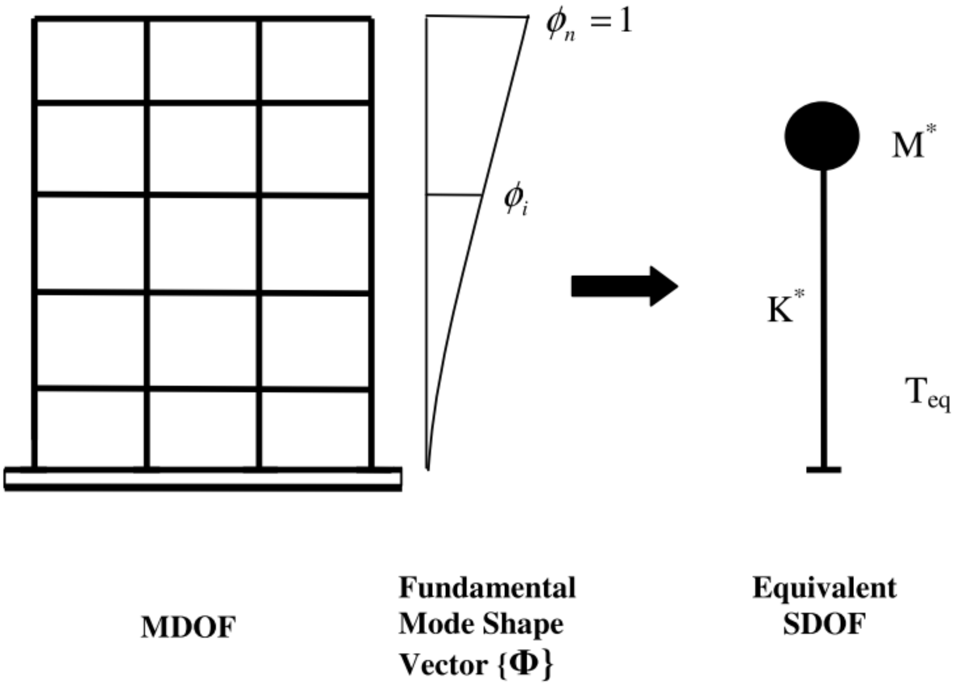


Figure 3.3 Conceptual diagrams for transformation of MDOF to SDOF system (Themelis, 2008)

3.5. Response Spectrum Analysis

This method entails the calculation of only the maximum values of the member forces and displacements using the proper design spectra obtained as the mean of many earthquake movements in each mode. According to a given earthquake in the earthquake design spectrum by considering the maximum values of the behavior of each vibration mode is calculated by the modal calculation method. Approximate values of the maximum behavior magnitudes are obtained by combining the maximum modal behavior magnitudes that are calculated and non-simultaneous for sufficient vibration mode. The following procedure is usually used for spectrum analysis;

Chose the design spectrum, determining the mode shapes and vibration periods to be included in the analysis, read the response level in the spectrum for the period of each of the modes considered. Calculate the probability that each mode corresponds to a degree of freedom read on the curve. Add the effect of the modes for a combined maximum response. Convert the combined maximum response into shears and moments used to construct the structure (Hassaballa et al., 2013).

The equation gives the mechanical balance approval of the response of the structures to the earthquake as follow:

$$\boldsymbol{M}\boldsymbol{u}''(t) + \boldsymbol{C}\boldsymbol{u}'(t) + \mathbf{Ku}(\mathrm{t}) = m_x\boldsymbol{u}''_{gx}(t) + m_y\boldsymbol{u}''_{gy}(t) + m_z\boldsymbol{u}''_{gz}(t) \qquad (3.2)$$

where:

- $(m_x, m_y, m_z\ )$, Loads in the global axis
- $(u''_{gx}, u''_{gy}\ , u''_{gz})$ Acceleration of gravy at (t) in the global direction
- **M,** is the mass matrix (lumped or consistent).
- **C**, is a viscous damping matrix
- **K**, is the static stiffness matrix for the system of structural elements
- $\boldsymbol{u}(t), \boldsymbol{u}'(t)\ and\ \boldsymbol{u}''(t)$, are the time-dependent vectors for the absolute node displacements, velocities and accelerations, respectively.

After finding maximum responses and acceleration, the analysis gives internal forces, moments and stress results in this way. Therefore, the program can perform any number of spectral analysis conditions in a single analysis process and give a specific name for each analysis case, and the analysis of each case is different in the spectrum acceleration used and also in the results of the composition (Rashid, 2015).

The RS curve represents the graphical function that connects the vibration period of structures and the so-called spectrum response acceleration. The RS curve of different type of soil showed In Figure 3.4.

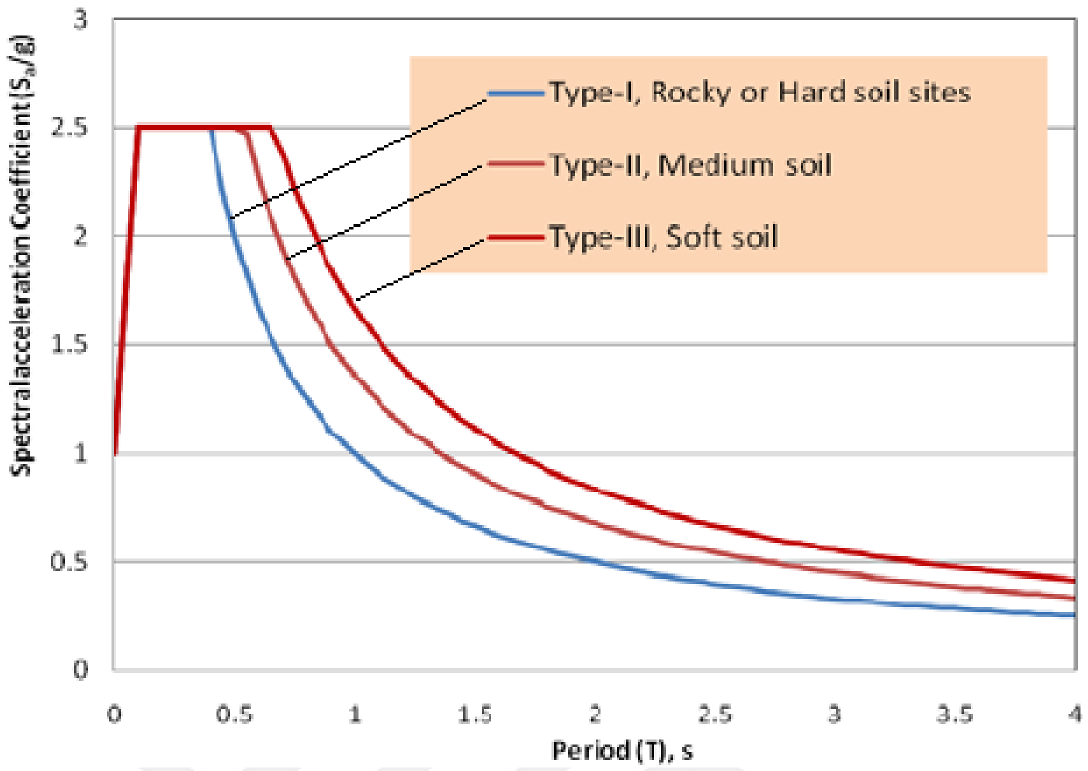


Figure 3.4 The RS curve of different type of soil

Examination of the spectrum curve given in Figure 3.4:

Maximum acceleration of hard soil is slightly higher than soft soil. However, the period range in which the maximum acceleration is observed on hard soil is short, maximum acceleration occurs in the range of 0.1 second to 0.5 second and decelerates rapidly after 0.5 period. This can prevent the building from collapsing without major damage from the earthquake.

3.5.1. Modal Combination Rules

The most commonly used methods for determining the size of the response of interest for structures are the Square Root of the Sum of Squares (SRSS) and the Complete Quadratic Combination (CQC) method.

3.5.1.1. Complete Quadratic Combination Method

The most protective method used to estimate the highest displacement or force value within a structure is to use the absolute sum of the modal response values. This methodology expect that the greatest modal values for all modes happen at a similar point in time.

CQC method is depends on random vibration theories and is widely accepted by most engineers and included as an option in most modern computer programs for seismic analysis (Wilson, 2002).

Peak value of a typical force calculated from maximum modal values using the CQC method by applying the following equation:

$$S_1 = \sqrt{\sum_{J=1}^{N} S_{1.j}^2 + 2\sum_{j=1}^{n} \left| \; S_{1,i} \, , S_{1,j} \; \right|} \qquad i \neq j \tag{3.3}$$

Here, it is important to note that, when the seismic force is parallel to a global axis, the spectral coordinates ($S_{a,i}$) are taken when generating the equations of forces or inertial loads ($P_{i,j}$) take values as fallow:

1. The vertical coordinates for the spectrum design ($S_{ax,j}$) and the associated natural vibration period ($T = T_1$) for all model compounds (J) have alternate values on the axis (X).
2. All other transitions to modal (J) take a value equal to zero ($S_{ay,j} = 0$) and deformation takes torsion or compounds for torsion ($S_{aw,j} = 0$) on axis (Y) (Rashid, 2015).

3.5.1.2. Square Root of the Sum of Squares Method

The commonly used combination rule is the Square Root of the Sum of Squares (SRSS) is the maximum structural response is evaluated as the square root of the sum of squared modal maximum responses. The rule is based on expressing the variance of the structural response as the sum of the variance of the modal response, thus ignoring the cross-covariance between the modal responses (Heredia, 2011). The SRSS method assumes that all maximum modal values are statistically independent.

$$S_1 = \sqrt{S_{1,1}^2 + S_{1,2}^2 + S_{1,3}^2 + \cdots + S_{1,j}^2} \tag{3.4}$$

So, the bending moments in turn become the previous relationship as fallow:

$$M_s = \sqrt{M_{s.1}^2 + M_{s.2}^2 + M_{s.3}^2 + \cdots + M_{s.j}^2} \tag{3.5}$$

This comes in the form of a composition idea, which is based on the fact that not all models reach maximum values at the same time. Therefore, they can evaluate the typical resonances such as transitions and torsion independently (Rashid, 2015).

The application of the rules for the statistical combination of the non-simultaneous maximum contributions of the response quantities calculated for each vibration mode, such as base shear, storey shear, internal force components, displacement and the storey drift of the stage, are specified below, provided that they are applied independently for each response quantity:

1. In cases where the natural periods of $T_m < T_n$ and the two modes of vibration always correspond to $\frac{T_m}{T_n} < 0.8$, the SRSS rule may be used for the combination of maximum modal additives.
2. In cases where the above condition is not fulfilled, the CQC rule is used for the combination of maximum modal contributions. When calculating the cross-correlation coefficients to be used in applying the rule, modal damping factors for all modes should be assumed to be 5%. (TEC, 2007).

3.6. Computational Methods

14 different steel structures were modeled and analysis of the models was performed using SAP2000 v20 software. Models are equivalent planar geometry with a 30 and 50-storey MRF models with different type of bracing in different locations of structures. All steel braces are concentrically (X) bracing with 1,2 and 5 storey. Steel braces are positioned symmetrically in the corner and in the middle of the outer axis of structures. Structures are 8 bays in the X direction, 4 bays in the Y direction with 6 meters between bays and the floor height is 4.0 m. The dimensions of all columns are HE 500B, the dimensions of all beams are IPE 400. All supports of structures are fixed. Pipe section was used for brace members. The dimensions of all bracing pipe was (TUBO-D219.1x4) according to EUROCODE one of default code in the program. Since the frames represent typical office buildings, the design live load of the building is taken as 2.00 kN/m².

3.7. Dynamic Methods

Earthquakes are sudden and accelerated vibration movements that cause the formation of inertial forces in the structure. Earthquake effects are highly complex impacts that can reach significant levels due to weight loads.

In TEC 2007 three methods are proposed for the linear elastic calculation of earthquake effects during the design phase; equivalent static lateral force method, RS method and time history function method. In this study, RS method is preferred because the analysis of the systems will be quite difficult with other analysis methods. In this case, the spectral responses obtained for each mode should be combined in some way. However, since these values are not simultaneous, some modal combination rules are applied, such as CQC and SRSS. The SRSS method gives incorrect results if the modal responses are close to the frequency. Due to the correct results, CQC modal combination method was used.

3.7.1. Determination of Earthquake Loads

Data were collected and evaluated by using the Sap 2000 software and by using the RS analysis according to TEC 2007 principles. The main purpose of the earthquake load calculations:

To determine earthquake loads against the possibility of earthquake hazard above a certain level in a region and the possibility of a life and economic loss due to the earthquake risk expected in the region.

The building structural system resisting seismic loads as a whole as well as each structural element of the system shall be provided with sufficient stiffness, stability and strength to ensure an uninterrupted and safe transfer of seismic loads down to the foundation soil. It is essential that floor systems possess sufficient stiffness and strength to ensure the safe transfer of lateral seismic loads between the elements of the structural system. In insufficient cases, appropriate transfer elements shall be arranged on floors (TEC, 2007).

The earthquake loads to be taken into account are explained in TEC 2007 as follows:

In order to determine seismic loads acting on buildings, Spectral Acceleration Coefficient and Seismic Load Reduction Factor shall be based on.

Seismic charges are not considered simultaneously along the two vertical axes of the building in the horizontal plane. The load factors used to calculate design forces under the combined effects of earthquake loads and other loads according to the shear strength theory will be derived from the corresponding static data. It can be assumed that the effects of wind and seismic charges do not move at the same time and that the most negative reaction variable caused by wind or earthquakes should be considered when designing each component.

The Spectral Acceleration Coefficient, $A(T)$, which shall be considered as the basis for the determination of seismic loads is given by Eq. (3.6). Elastic Spectral Acceleration, $S_{ae}(T)$ is given by Eq. which is the ordinate of Elastic Acceleration Spectrum defined for 5 % damped rate is derived by multiplying Spectral Acceleration Coefficient with gravity, g.

$$A(T) = A_0 * I * S(T) \tag{3.6}$$

$$S_{ae}(T) = A(T) * g \tag{3.7}$$

The Effective Ground Acceleration Coefficient, A_0, appearing in Eq. (3.6) is specified in Table 3.1.

Table 3.1 Effective ground acceleration coefficient (A_0) (TEC, 2007)

Sesismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

The design earthquake considered in the 2007 TEC corresponds to the high-density earthquake. The probability of exceeding the planned earthquake for buildings over a 50-year period is 10%. Different earthquake probabilities are taken into account in the evaluation and reinforcement of existing buildings. Seismic zones cited in the TEC 2007 are the first, second, third and fourth seismic zones depicted in Seismic Zoning Map of Turkey as shown in Figure 3.5.

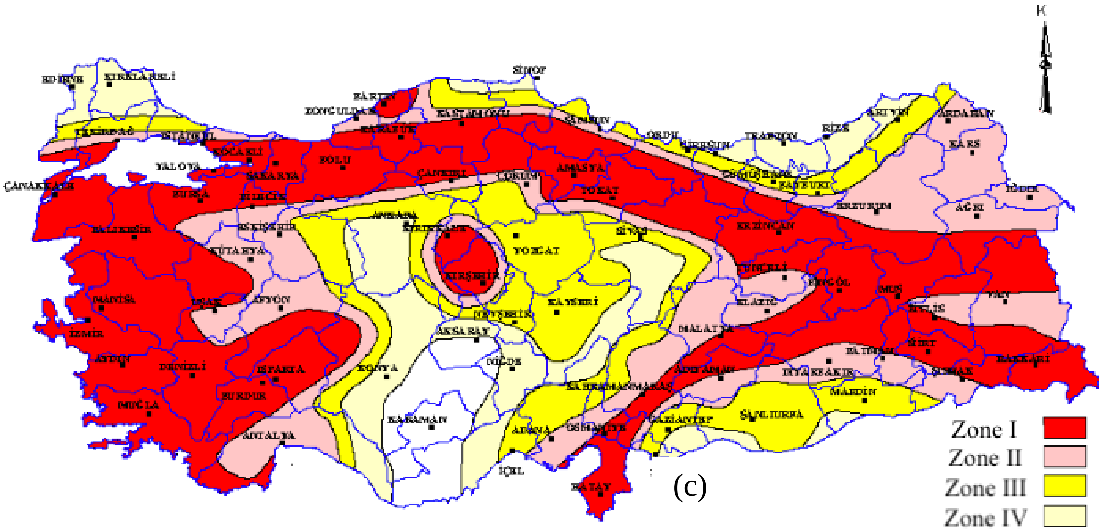


Figure 3.5 Seismic Zoning Map of Turkey

In our study, the seismic zone was chosen as 1st , so the value of A_0 is equal to 0.4 according to Table 3.1.

The Building Importance Factor, I, appearing in Equation (3.6) is specified in Table 3.2.

Table 3.2 Building importance factor (I)

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods **a)** Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	1.2
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

Since our models in this study are office buildings, it is classified in other buildings category according to table 3.2 and building importance factor I is equal to 1.

The Spectrum Coefficient, $S(T)$, appearing in Eq. (3.6) shall be determined by Eq. (3.8), depending on the local site conditions and the building natural period, T (Table 3.3).

$$\begin{aligned} S(T) &= 1 + 1{,}5\,\frac{T}{T_A} && (0 \le T \le T_A) \\ S(T) &= 2{,}5 && (T_A < T \le T_B) \\ S(T) &= 2{,}5\left[\frac{T_B}{T}\right]^{0.8} && (T_B < T) \end{aligned} \tag{3.8}$$

Spectrum Characteristic Periods, T_A and T_B , appearing in Eq. (3.8) are specified in Table 3.3, depending on Local Site Classes defined in Table 3.4.

Table 3.3 Spectrum characteristic periods (T_A, T_B)

Local Site Class	T_A (second)	T_B (second)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Our local site class is Z4 which T_A = 0.20 and T_B = 0.90 second.

Soil groups and local site classes to be considered as the bases of determination of local soil conditions are given in Table 3.4 and Table 3.5, respectively. Values concerning soil parameters in Table 3.4 are to be considered as standard values given for guidance in determining the soil groups. Soil investigations based on required site and laboratory tests are mandatory for below given buildings with related reports prepared and attached to design documents. Soil groups and local site classes defined in accordance with Table 3.4 and Table 3.5 (TEC, 2007).

Table 3.4 Soil groups

Soil Group	*Description of Soil Group*	*Standard Penetration* **(N/30)**	*Relative Density* (%)	*Unconfined. Compressive Strength* **(kPa)**	*Drift Wave Velocity* **(m / s)**
(A)	**1.** Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks **2.** Very dense sand, gravel... **3.** Hard clay and silty clay…	— > 50 > 32	— 85 – 100 —	> 1000 — > 400	> 1000 > 700 > 700
(B)	**1.** Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity…… **2.** Dense sand, gravel.......... **3.** Very stiff clay, silty clay…	— 30 – 50 16 – 32	— 65 – 85 —	500 – 1000 — 200 – 400	700 – 1000 400 – 700 300 – 700
(C)	**1.** Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity **2.** Medium dense sand and gravel.......................... **3.** Stiff clay and silty clay.....	— 10 – 30 8 – 16	— 35 – 65 —	< 500 — 100 – 200	400 – 700 200 – 400 200 – 300
(D)	**1.** Soft, deep alluvial layers with high ground water level **2.** Loose sand..................... **3.** Soft clay and silty clay…..	— < 10 < 8	— < 35 —	— — < 100	< 200 < 200 < 200

Table 3.5 Local site classes

Local Site Class	*Soil Group according to Table 6.1 and Topmost Soil Layer Thickness* ($\mathbf{h_1}$)
Z1	Group **(A)** soils Group **(B)** soils with $h_l \leq 15$ m
Z2	Group **(B)** soils with $h_l > 15$ m Group **(C)** soils with $h_l \leq 15$ m
Z3	Group **(C)** soils with 15 m < $h_l \leq 50$ m Group **(D)** soils with $h_l \leq 10$ m
Z4	Group **(C)** soils with $h_l > 50$ m Group **(D)** soils with $h_l > 10$ m

When necessary, elastic design acceleration spectrum can be determined by special researches by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by Eq. (3.6) based on relevant characteristic periods specified in Table 3.3.

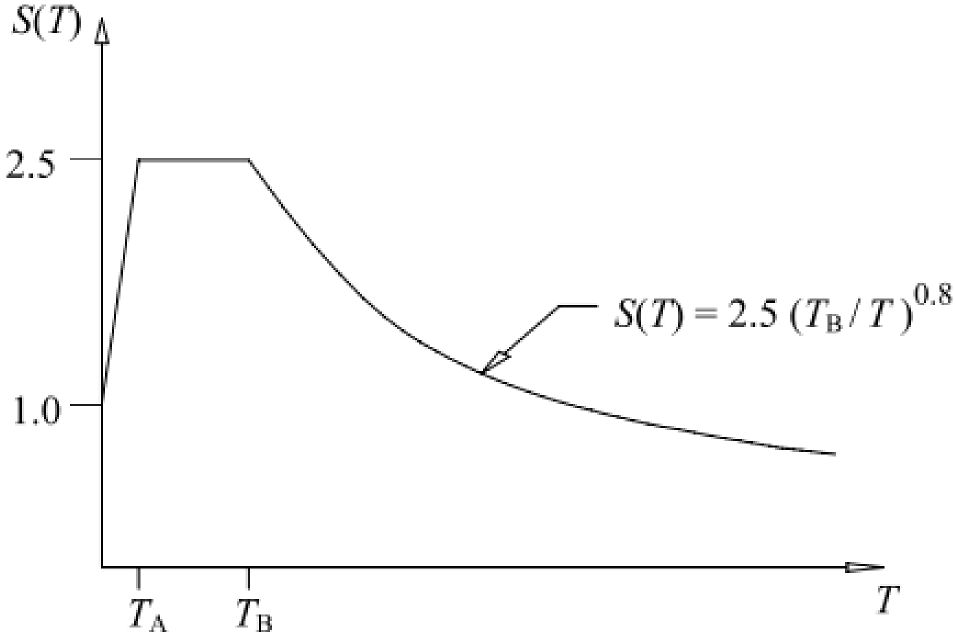


Figure 3.6 Relationship between the building natural period (T) and the spectrum coefficient $S(T)$

In order to take into account the nonlinear behavior of the structure during the earthquake, the elastic seismic loads to be determined by the spectral acceleration coefficient are taken into account by dividing the seismic charge reduction factor described below. The reduction factor of the seismic charge should be determined by the equation. (3.9) according to structural system behavior factor R defined in Table 3.6 for different structural systems and natural release period T.

$$R_a(T) = 1{,}5 + \left(R - 1{,}5\ \frac{T}{T_A}\right) \qquad (0 \leq T \leq T_A) \qquad (3.9)$$

$$R_a(T) = R \qquad (T_A < T)$$

In this study all models are structural systems of nominal ductility level in both directions. So according to the Table 3.6 Structural behavior factors (R) is equal to 5. All RS function parameters are entered into the Sap2000 program in accordance with TEC 2007 and the function graph is shown in Figure 3.7. The sum of the effective masses calculated in the x and y directions perpendicular to each other cannot be less than 90% of the total building mass.

Table 3.6 Structural behavior factors (R) for structural steel buildings

BUILDING STRUCTURAL SYSTEM	*Nominal Ductility Level*	*High Ductility Level*
(1) STRUCTURAL STEEL BUILDINGS		
(1.1) Buildings in which seismic loads are fully resisted by frames........	5	8
(1.2) Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top..............	—	4
(1.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*..............	4	5
(b) *Eccentrically braced frames*..............	—	7
(c) *Reinforced concrete structural walls*..............	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*..............	5	6
(b) *Eccentrically braced frames*..............	—	8
(c) *Reinforced concrete structural walls*..............	4	7

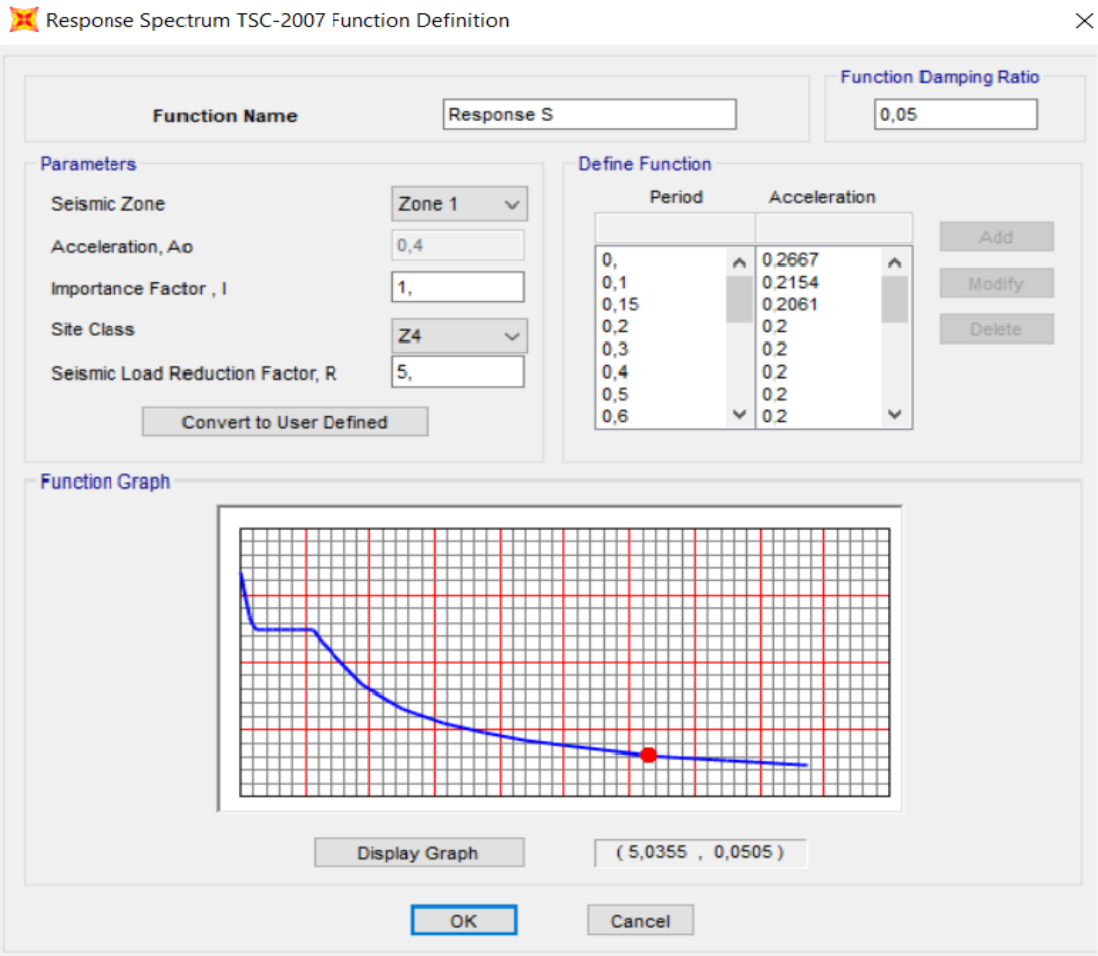


Figure 3.7 Response spectrum function parameters and function graph

3.8. Description of Buildings

In order to investigate the effectiveness of steel braces used to increase the horizontal load capacity of steel structures, 14 models were selected in this study. All models analyzed by using SAP2000 software. Models are equivalent planar geometry with a 30 and 50-storey MRF models and structures retrofitted with different type of bracing in different locations. Structures are 8 bays in the X direction, 4 bays in the Y direction with 6.0 meters between bays and the floor height is 4.0 m. The dimensions of all columns are HE 500B, the dimensions of all beams are IPE 400. St37 quality steel material is used as a structural element, for slab 4000 Psi concrete and 12 cm membrane thickness was used.

For the retrofitted structures, all steel braces are concentrically (X) bracing with 1,2 and 5 storey. Steel braces are positioned symmetrically in the corner and in the middle of the outer axis of structures. Pipe section was used for brace members. The dimensions of all bracing pipe was (TUBO-D219.1x4) according to EUROCODE. The typical floor plan for all models is given in Figure 3.8.

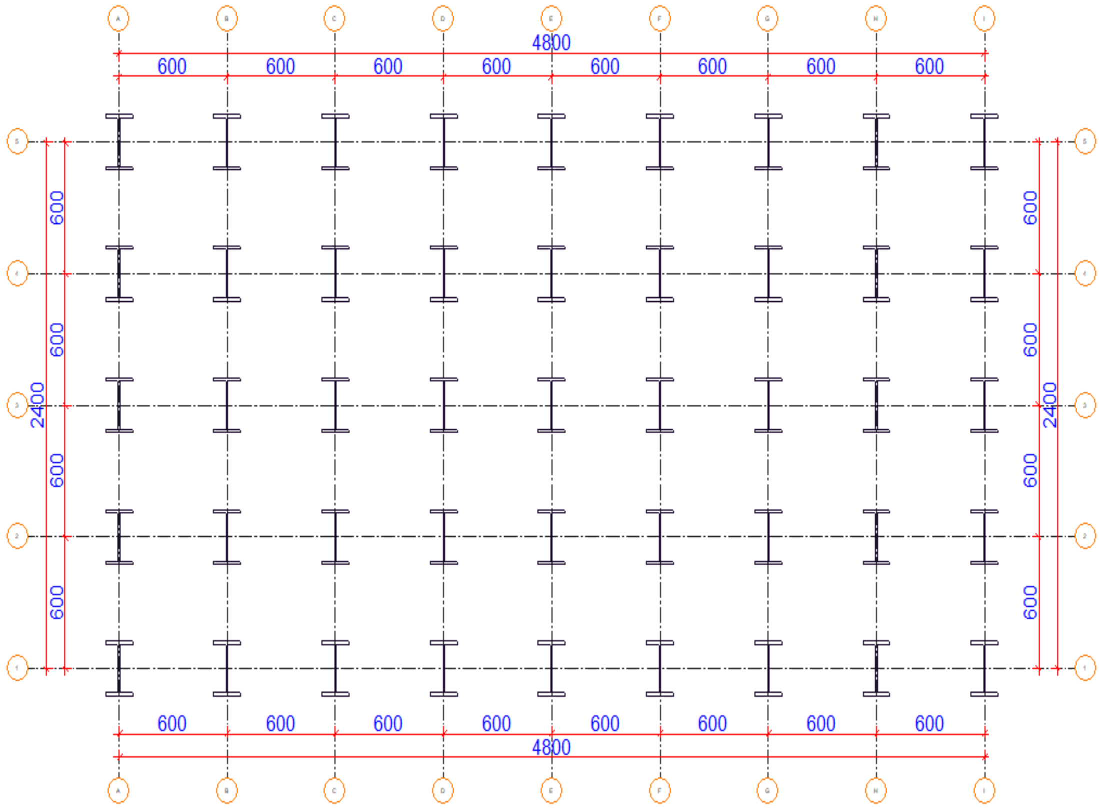
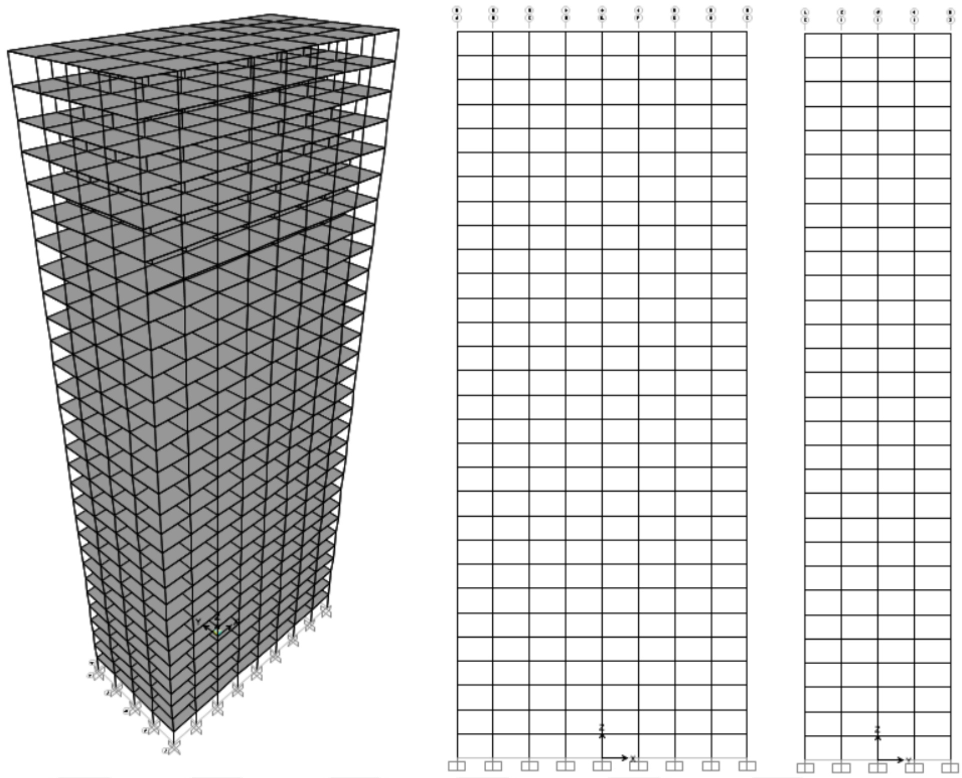


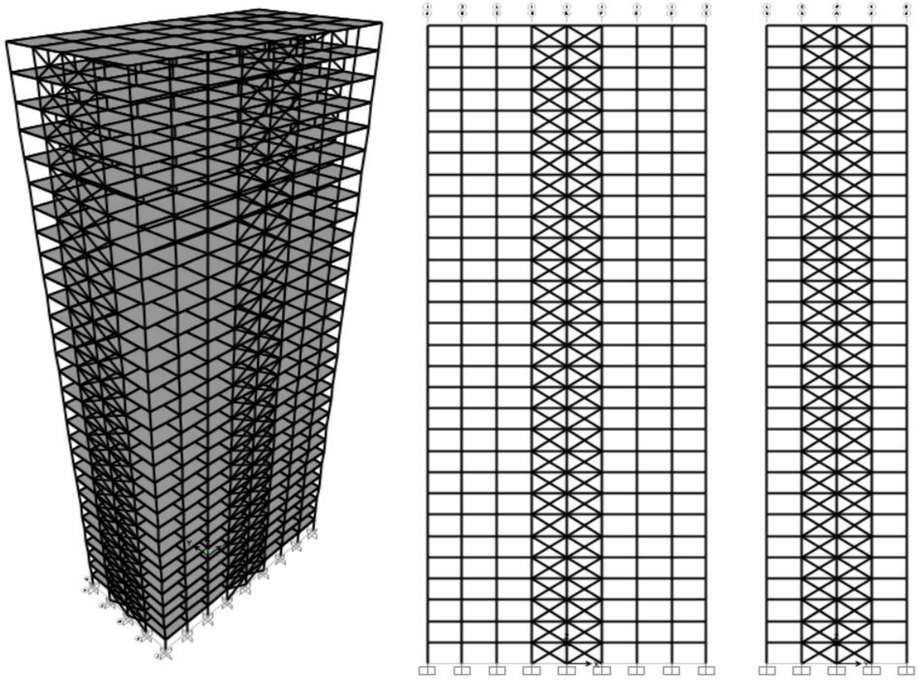
Figure 3.8 Typical floor plan for all models (units in mm)

The following figures show the elevation and 3-dimensional view of all 30 and 50 storey models.



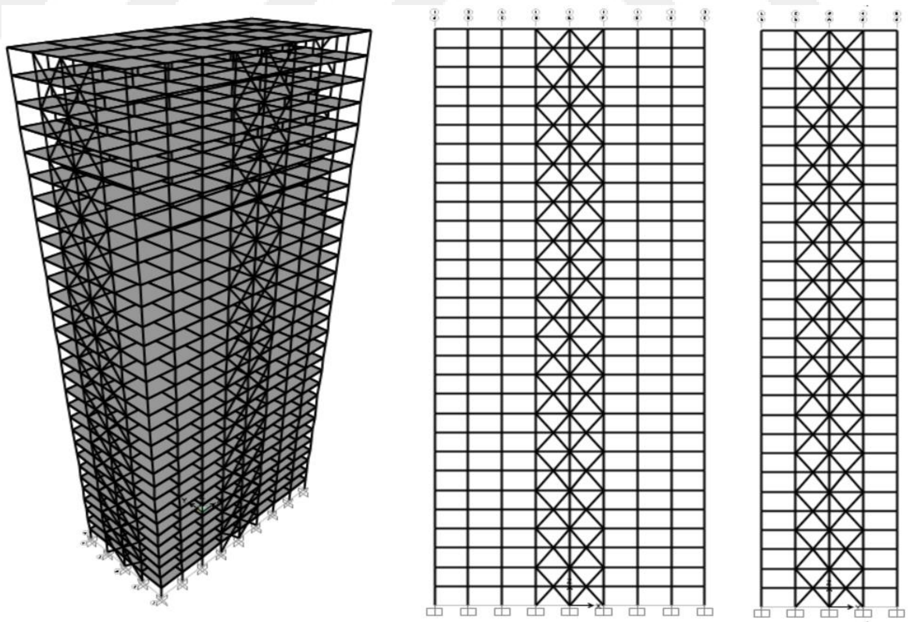
a) b) c)

Figure 3.9 30-strorey moment resistant frame (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



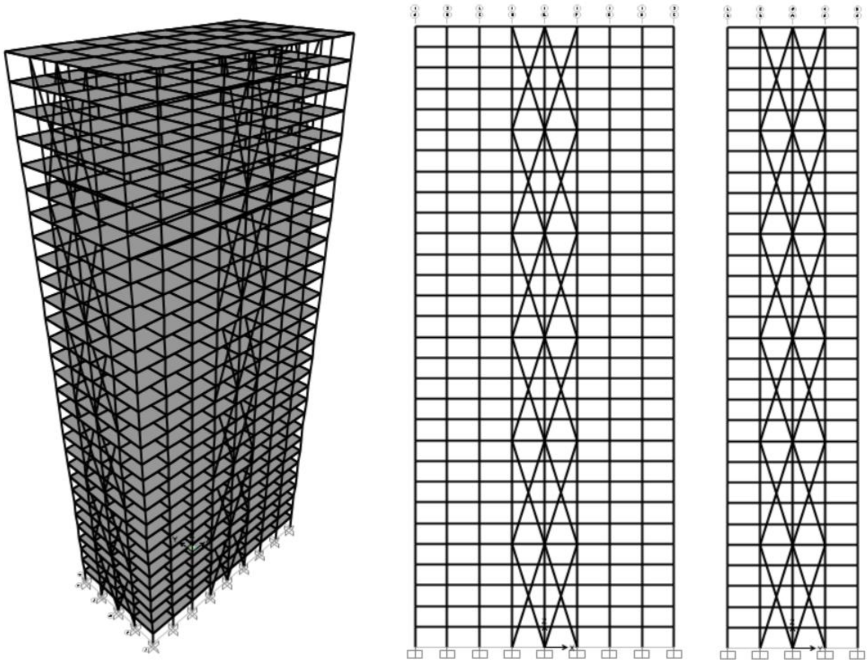
b) b) c)

Figure 3.10 30-storey MRF retrofitted with inner X brace every 1 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



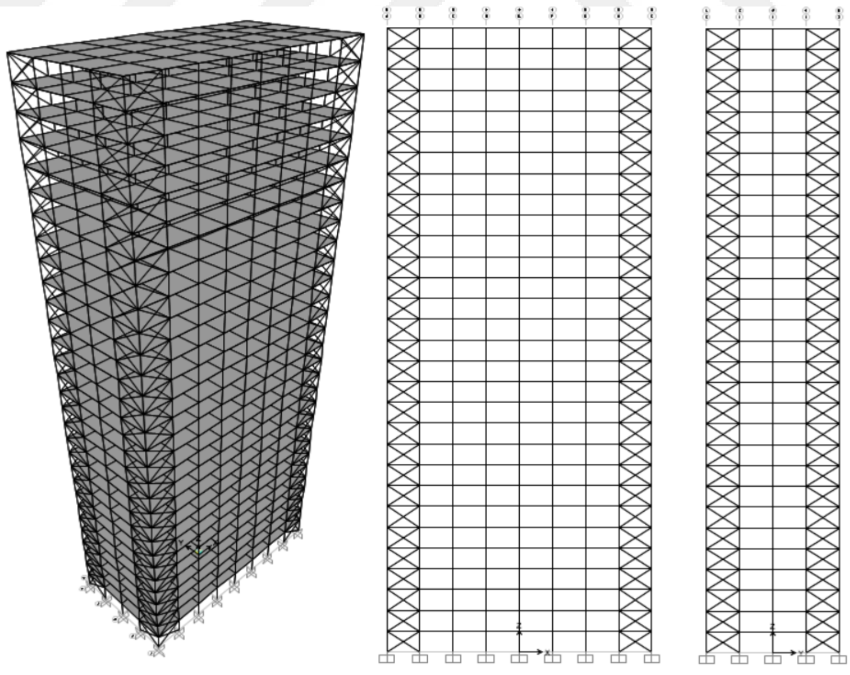
a) b) c)

Figure 3.11 30-storey MRF retrofitted with inner X brace every 2 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



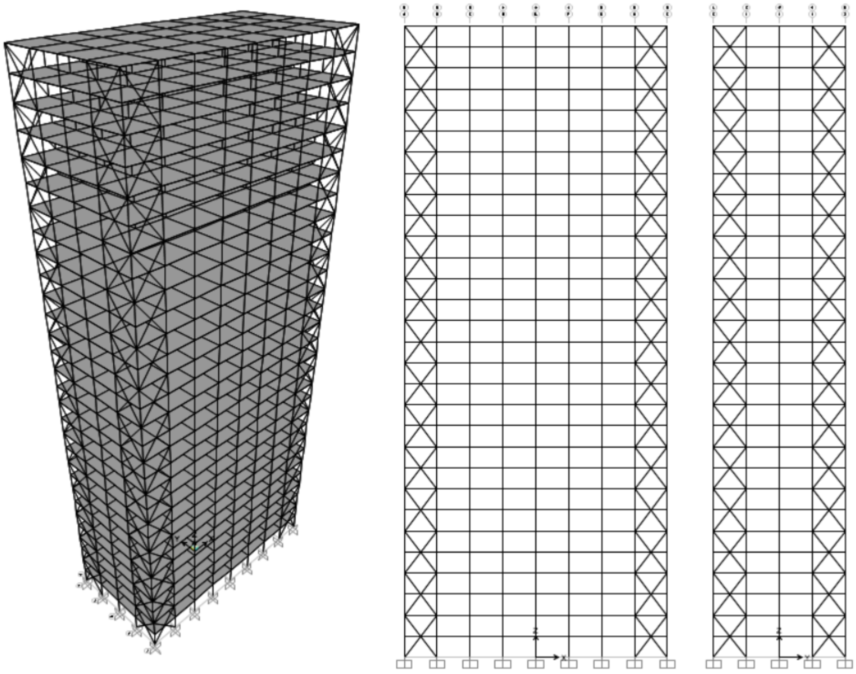
a) b) c)

Figure 3.12 30-storey MRF retrofitted with inner X brace every 5 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



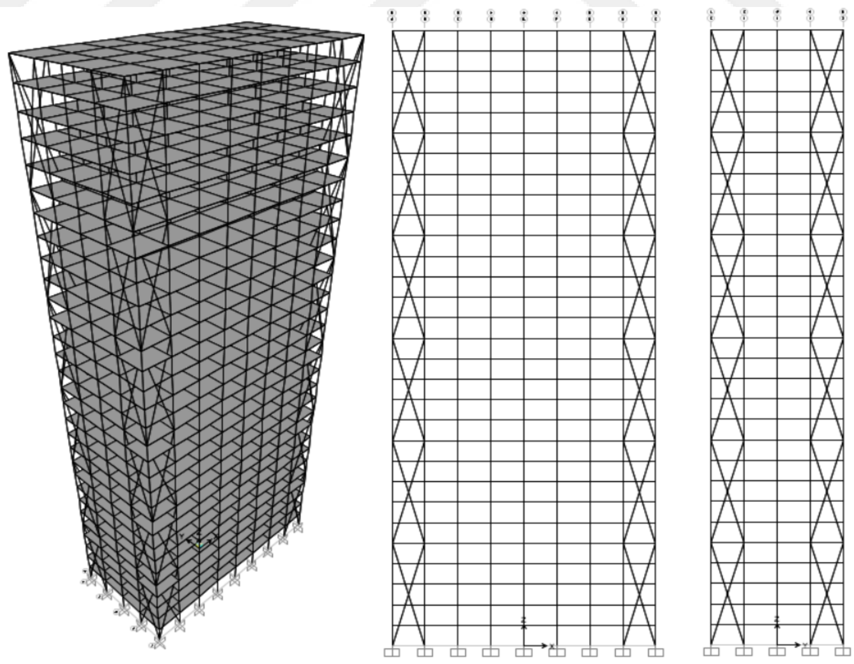
a) b) c)

Figure 3.13 30-storey MRF retrofitted with outer X brace every 1 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



a) b) c)

Figure 3.14 30-storey MRF retrofitted with outer X brace every 2 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane



a) b) c)

Figure 3.15 30-storey MRF retrofitted with outer X brace every 5 storey (a) 3 dimensional view, (b) elevation view on X-Z plane, (c) elevation view on Y-Z plane

All building models were analyzed with 30 and 50-storey. Examined steel structure models and model designations are given in Table 3.7.

Table 3.7 Examined steel structure models and model designation

Examined steel structure models	Model designation	Number of Frame Elements
30-storey Moment Resistant Frame	30-MRF	3630
30-storey MRF retrofitted with inner brace every 1 storey	30-inner-1	4110
30-storey MRF retrofitted with inner brace every 2 storey	30-inner-2	3870
30-storey MRF retrofitted with inner brace every 5 storey	30-inner-5	3726
30-storey MRF retrofitted with outer brace every 1 storey	30-outer-1	4110
30-storey MRF retrofitted with outer brace every 2 storey	30-outer-2	3870
30-storey MRF retrofitted with outer brace every 5 storey	30-outer-5	3726
50-storey Moment Resistant Frame	50-MRF	6050
50-storey MRF retrofitted with inner brace every 1 storey	50-inner-1	6850
50-storey MRF retrofitted with inner brace every 2 storey	50-inner-2	6450
50-storey MRF retrofitted with inner brace every 5 storey	50-inner-5	6210
50-storey MRF retrofitted with outer brace every 1 storey	50-outer-1	6850
50-storey MRF retrofitted with outer brace every 2 storey	50-outer-2	6450
50-storey MRF retrofitted with outer brace every 5 storey	50-outer-5	6210

CHAPTER 4

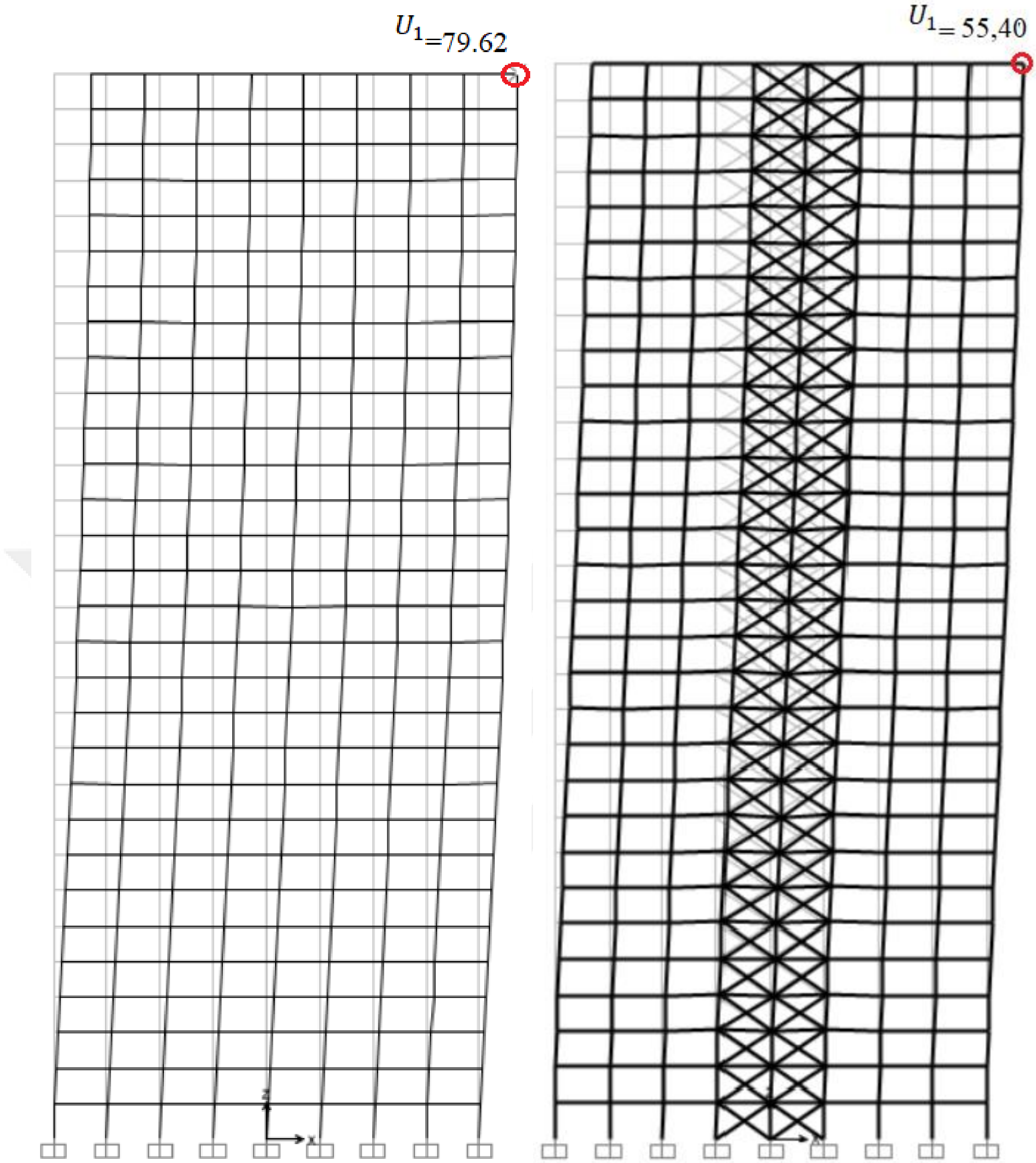
RESULTS AND DISCUSSIONS

4.1. Computational Results

In this section, horizontal displacements, natural periods and base shear of MRF steel structures and retrofitted structures were given as graphs and tables comparatively. With the data obtained, the effects of the location and type of the steel braces on the structure dynamic behavior were observed.

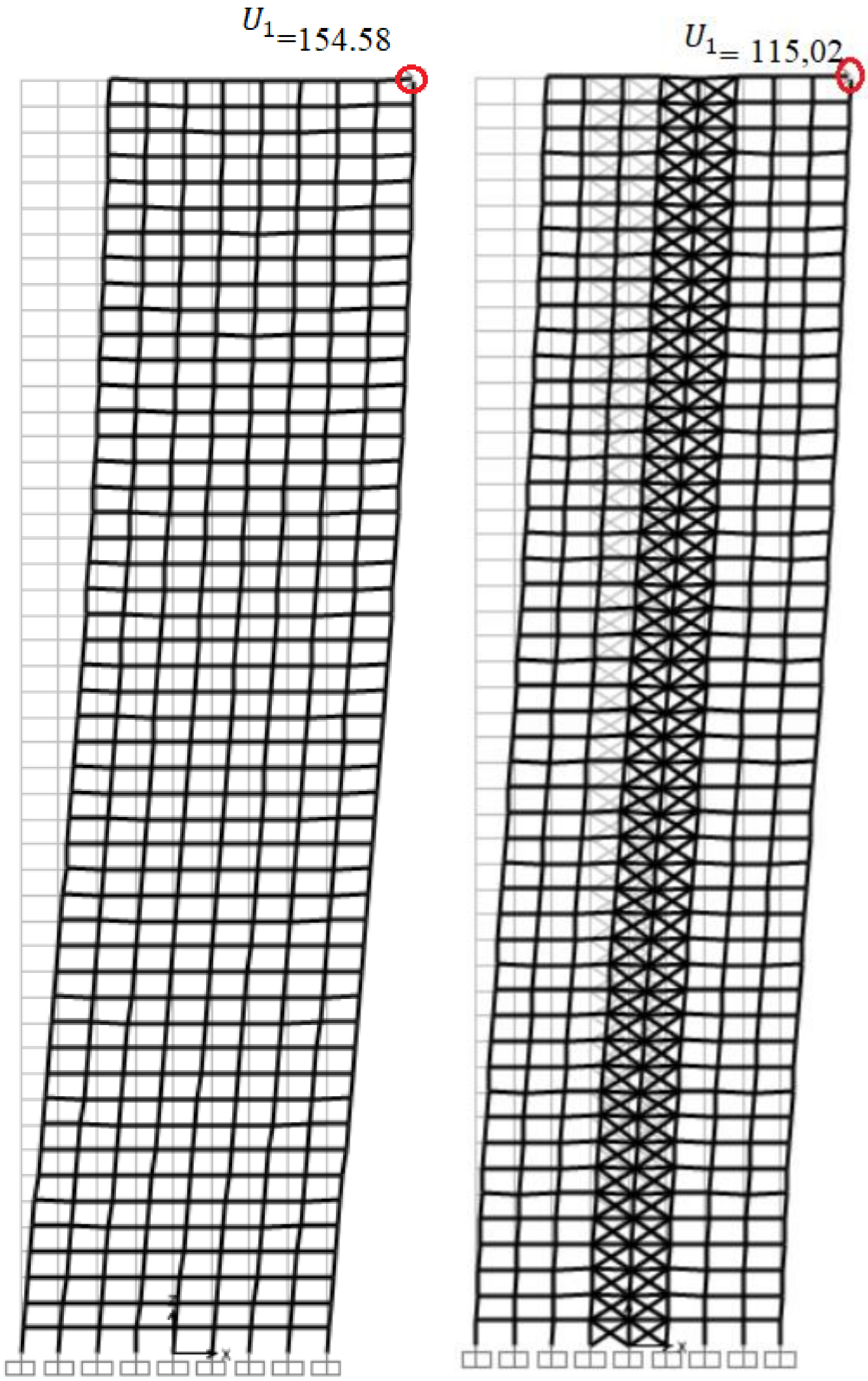
The following figures and tables show the variation of horizontal displacements, natural periods and base shear of MRF steel structures and retrofitted structures of all 30 and 50 storey models. As seen from the figures and tables, there is a noticeable difference between the MRF steel structures and retrofitted structures. Low natural periods are obtained in the structure by using steel brace elements and greatly reduced in displacement due to horizontal loads.

It has been observed that all structures studied are severely affected by specific earthquakes. In addition, the use of brace members for retrofitting significantly reduces the value of the maximum floor displacement compared to the MRF, especially in the case of modifications with bracing systems. The maximum storey displacements, natural periods and base shears were affected by the bracing location, number of stories and bracing type. In Figure 4.1 in the case of 30 storey MRF retrofitted with inner X brace every 1 storey, the maximum storey displacement was smaller than MRF, by increasing numbers of stories the maximum storey displacements were also increased.



(a) (b)

Figure 4.1 The bracing performance for 30 storey building under seismic load, showing the reduced in maximum displacement (a) Without bracing, (b) With internal bracing



(a) (b)

Figure 4.2 The bracing performance for 50 storey building under seismic load, showing the reduced in maximum displacement (a) Without bracing, (b) With internal bracing

Table 4.1 Variation of displacement of the 30 storey MRF and retrofitted buildings with X-bracing in X direction

	30-MRF	30-outer-1	30-inner-1	30-outer-2	30-inner-2	30-outer-5	30-inner-5
Height (m)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)
4	3,31	1,31	1,35	1,52	1,49	2,50	2,53
8	7,35	2,99	2,98	3,37	3,42	5,51	5,58
12	11,36	4,91	4,77	5,53	5,34	8,48	8,56
16	15,31	7,03	6,69	7,69	7,48	11,38	11,46
20	19,20	9,30	8,71	10,13	9,60	14,26	14,31
24	23,02	11,69	10,81	12,54	11,86	17,41	17,35
28	26,77	14,17	12,97	15,14	14,10	20,51	20,31
32	30,43	16,71	15,18	17,69	16,42	23,51	23,15
36	34,01	19,30	17,41	20,37	18,71	26,42	25,88
40	37,50	21,92	19,66	22,98	21,04	29,25	28,53
44	40,89	24,55	21,91	25,67	23,35	32,26	31,34
48	44,18	27,18	24,15	28,28	25,65	35,19	34,05
52	47,36	29,80	26,38	30,94	27,92	37,99	36,62
56	50,43	32,38	28,58	33,50	30,15	40,66	39,06
60	53,38	34,93	30,75	36,08	32,35	43,23	41,38
64	56,21	37,44	32,87	38,55	34,49	45,90	43,82
68	58,91	39,89	34,94	41,02	36,59	48,46	46,14
72	61,47	42,27	36,96	43,36	38,61	50,86	48,29
76	63,90	44,59	38,92	45,69	40,59	53,11	50,28
80	66,18	46,83	40,81	47,88	42,47	55,22	52,12
84	68,32	48,99	42,63	50,05	44,30	57,41	54,08
88	70,29	51,07	44,38	52,07	46,02	59,46	55,88
92	72,10	53,06	46,05	54,06	47,69	61,32	57,49
96	73,75	54,96	47,64	55,89	49,24	62,98	58,90
100	75,21	56,77	49,15	57,70	50,73	64,48	60,13
104	76,49	58,49	50,57	59,33	52,10	66,08	61,49
108	77,58	60,12	51,91	60,95	53,40	67,50	62,67
112	78,47	61,65	53,17	62,37	54,59	68,69	63,60
116	79,14	63,10	54,34	63,83	55,69	69,64	64,29
120	79,61	64,45	55,40	65,01	56,66	70,33	64,72

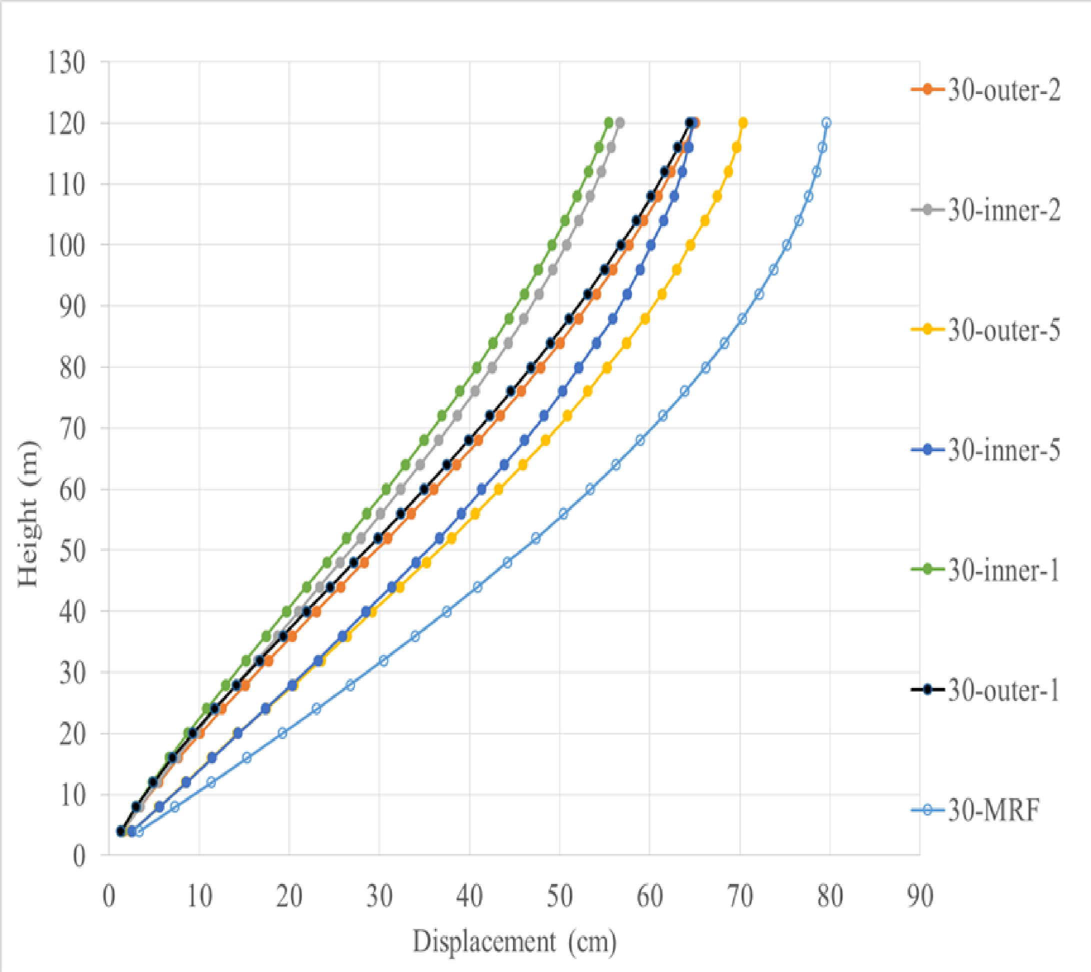


Figure 4.3 Distribution of displacement of the 30 storey MRF and retrofitted buildings with X-bracing in X direction

Table 4.2 Variation of displacement of the 30 storey MRF and retrofitted buildings with X-bracing in Y direction

	30-MRF	30-outer-1	30-inner-1	30-outer-2	30-inner-2	30-outer-5	30-inner-5
Height (m)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)
4	1,24	0,84	0,84	0,87	0,89	1,08	1,09
8	3,41	2,22	2,18	2,41	2,37	2,98	2,99
12	5,81	3,80	3,67	4,11	3,96	5,07	5,09
16	8,27	5,51	5,25	5,88	5,66	7,22	7,23
20	10,73	7,32	6,91	7,76	7,40	9,39	9,37
24	13,18	9,21	8,63	9,68	9,20	11,59	11,53
28	15,62	11,16	10,39	11,67	11,01	13,79	13,66
32	18,02	13,15	12,19	13,68	12,87	15,96	15,77
36	20,40	15,17	14,02	15,72	14,73	18,10	17,84
40	22,75	17,21	15,86	17,77	16,61	20,23	19,90
44	25,07	19,25	17,71	19,82	18,48	22,38	21,97
48	27,34	21,29	19,57	21,86	20,36	24,50	24,01
52	29,57	23,32	21,42	23,90	22,22	26,58	26,01
56	31,76	25,34	23,26	25,91	24,07	28,61	27,96
60	33,90	27,33	25,08	27,90	25,91	30,61	29,87
64	35,98	29,29	26,88	29,85	27,72	32,59	31,77
68	38,00	31,22	28,66	31,77	29,50	34,53	33,63
72	39,96	33,10	30,41	33,64	31,24	36,40	35,43
76	41,85	34,94	32,12	35,47	32,96	38,20	37,15
80	43,67	36,73	33,79	37,25	34,62	39,95	38,82
84	45,41	38,47	35,42	38,97	36,25	41,66	40,47
88	47,07	40,15	37,01	40,63	37,82	43,29	42,04
92	48,64	41,76	38,54	42,24	39,34	44,84	43,53
96	50,12	43,32	40,02	43,77	40,80	46,30	44,92
100	51,50	44,80	41,44	45,23	42,20	47,67	46,23
104	52,77	46,22	42,80	46,62	43,53	48,99	47,49
108	53,93	47,56	44,09	47,94	44,80	50,21	48,66
112	54,97	48,83	45,32	49,17	45,98	51,31	49,71
116	55,88	50,02	46,47	50,33	47,09	52,28	50,62
120	56,68	51,12	47,53	51,36	48,10	53,13	51,41

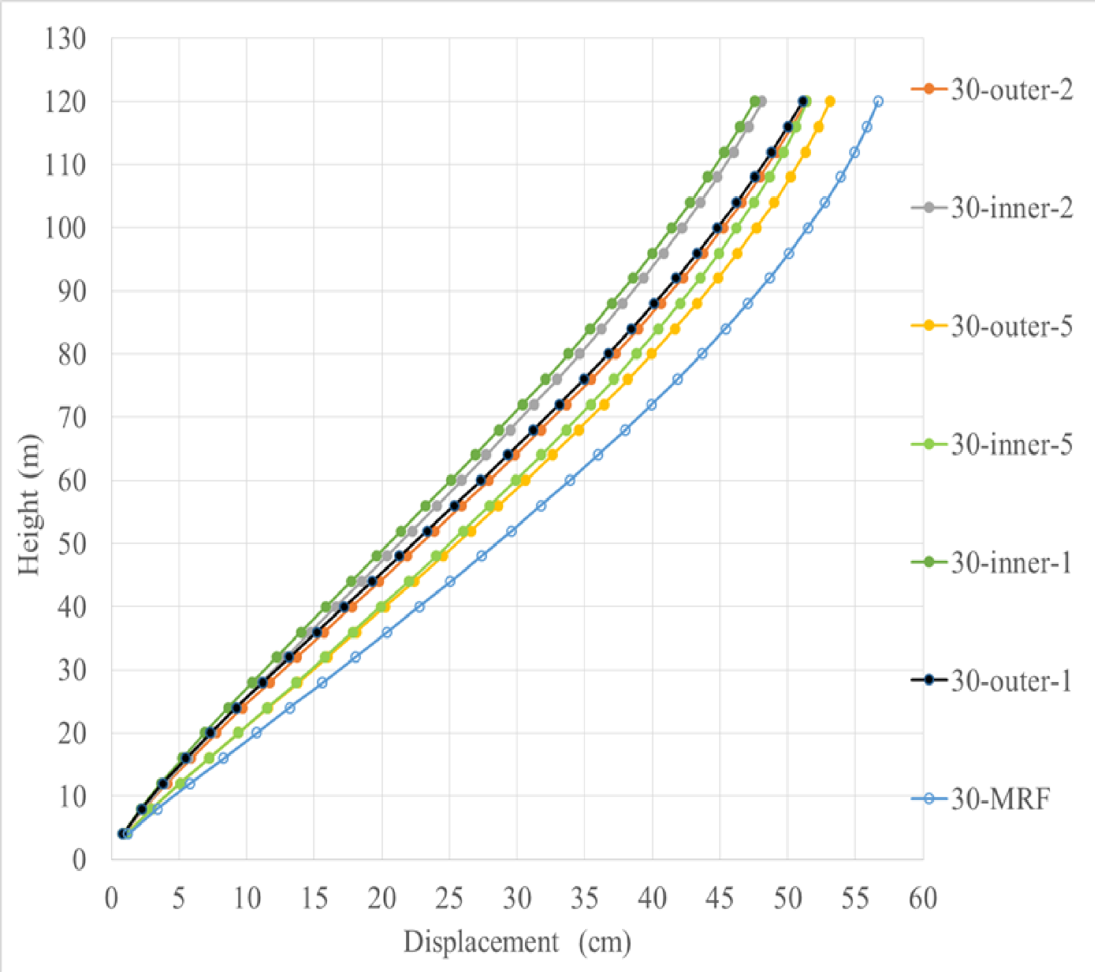


Figure 4.4 Distribution of displacement of the 30 storey MRF and retrofitted buildings with X-bracing in Y direction

Table 4.3 Variation of displacement of the 50 storey MRF and retrofitted buildings with X-bracing in X direction

	50-MRF	50-outer-1	50-inner-1	50-outer-2	50-inner-2	50-outer-5	50-inner-5
Height (m	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)
4	3,60	1,36	1,41	1,59	1,56	2,66	2,71
8	8,00	3,15	3,15	3,56	3,61	5,90	5,99
12	12,42	5,23	5,09	5,89	5,69	9,13	9,25
16	16,82	7,57	7,21	8,28	8,04	12,33	12,46
20	21,21	10,11	9,47	11,00	10,41	15,56	15,67
24	25,56	12,83	11,87	13,75	12,98	19,11	19,08
28	29,90	15,70	14,37	16,75	15,56	22,67	22,46
32	34,20	18,69	16,97	19,75	18,30	26,19	25,80
36	38,48	21,78	19,65	22,94	21,03	29,67	29,08
40	42,72	24,96	22,39	26,12	23,87	33,14	32,34
44	46,93	28,21	25,18	29,43	26,71	36,81	35,76
48	51,09	31,51	28,02	32,73	29,62	40,46	39,14
52	55,21	34,85	30,88	36,12	32,52	44,06	42,45
56	59,29	38,23	33,77	39,48	35,46	47,60	45,71
60	63,31	41,62	36,68	42,90	38,39	51,11	48,92
64	67,29	45,03	39,59	46,29	41,34	54,73	52,23
68	71,21	48,43	42,50	49,71	44,28	58,31	55,50
72	75,08	51,83	45,40	53,09	47,21	61,82	58,69
76	78,89	55,22	48,30	56,49	50,12	65,27	61,80
80	82,64	58,59	51,17	59,84	53,01	68,66	64,86
84	86,33	61,94	54,03	63,19	55,89	72,12	67,99
88	89,95	65,26	56,86	66,48	58,73	75,51	71,06
92	93,50	68,54	59,66	69,77	61,54	78,83	74,04
96	96,98	71,79	62,42	72,98	64,31	82,07	76,94
100	100,40	74,99	65,15	76,18	67,05	85,24	79,76
104	103,73	78,15	67,84	79,31	69,74	88,44	82,65
108	106,99	81,26	70,49	82,41	72,40	91,57	85,45
112	110,17	84,32	73,09	85,44	74,99	94,61	88,16
116	113,27	87,32	75,64	88,43	77,55	97,56	90,77
120	116,29	90,26	78,14	91,34	80,04	100,42	93,30
124	119,22	93,14	80,59	94,21	82,50	103,31	95,88
128	122,06	95,95	82,98	96,99	84,87	106,10	98,36
132	124,81	98,70	85,31	99,72	87,21	108,79	100,74
136	127,46	101,38	87,58	102,37	89,46	111,38	103,01
140	130,02	103,99	89,79	104,96	91,68	113,88	105,18
144	132,47	106,53	91,94	107,46	93,80	116,38	107,39
148	134,82	108,99	94,02	109,91	95,89	118,78	109,51
152	137,07	111,38	96,04	112,25	97,88	121,07	111,50
156	139,21	113,69	97,99	114,55	99,83	123,23	113,36
160	141,23	115,92	99,88	116,73	101,68	125,28	115,11
164	143,14	118,07	101,69	118,87	103,49	127,36	116,92
168	144,93	120,14	103,44	120,89	105,20	129,31	118,60
172	146,60	122,14	105,11	122,87	106,86	131,14	120,15
176	148,15	124,06	106,72	124,72	108,43	132,82	121,55
180	149,56	125,90	108,26	126,55	109,94	134,39	122,83
184	150,84	127,67	109,74	128,25	111,37	136,00	124,18
188	151,99	129,37	111,15	129,92	112,74	137,48	125,40
192	152,99	131,01	112,50	131,47	114,03	138,81	126,46
196	153,85	132,58	113,80	133,04	115,27	139,97	127,34
200	154,58	134,09	115,02	134,42	116,42	140,97	128,07

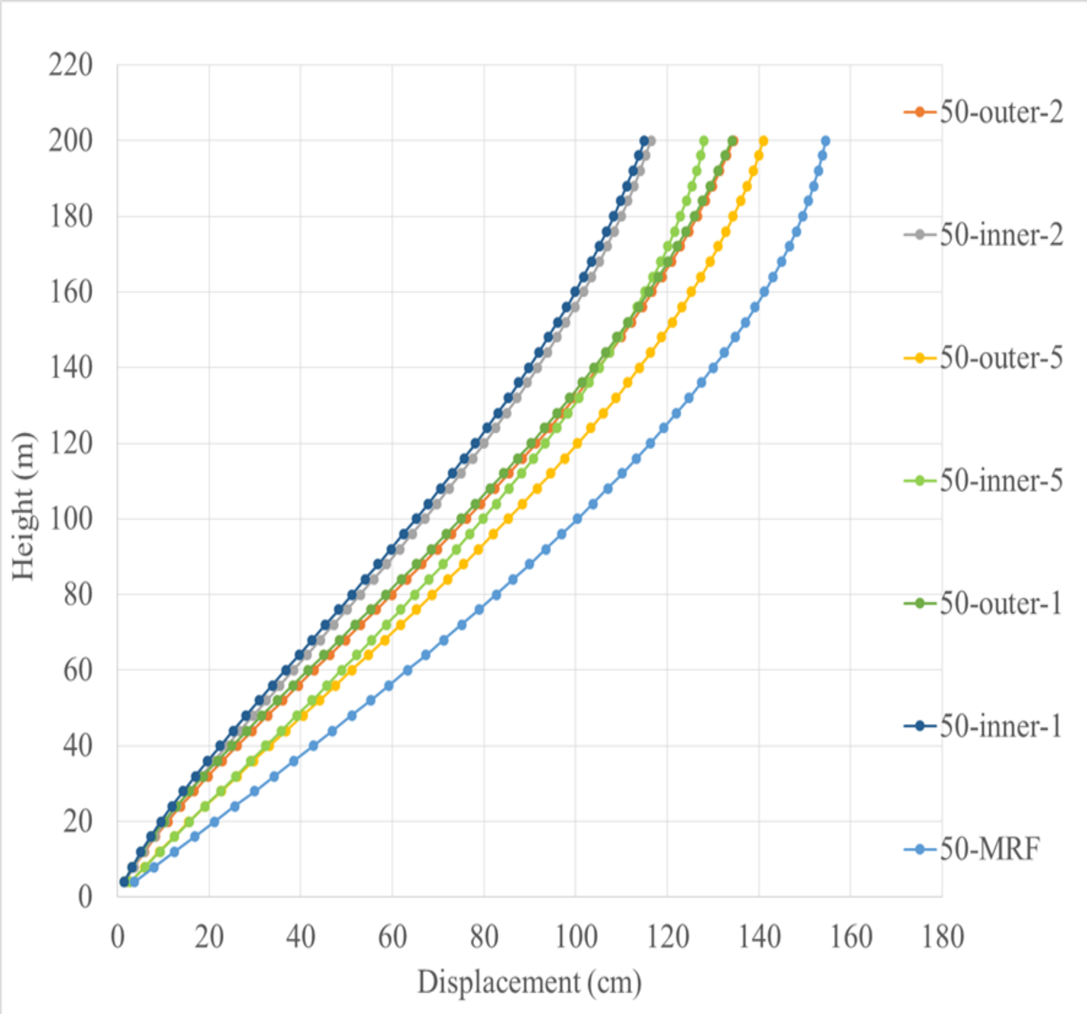


Figure 4.5 Distribution of displacement of the 50 storey MRF and retrofitted buildings with X-bracing in X direction

Table 4.4 Variation of displacement of the 50 storey MRF and retrofitted buildings with X-bracing in Y direction

	50-MRF	50-outer-1	50-inner-1	50-outer-2	50-inner-2	50-outer-5	50-inner-5
Height (m)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)	Disp.(cm)
4	1,26	0,85	0,85	0,88	0,90	1,11	1,11
8	3,51	2,27	2,21	2,45	2,41	3,07	3,06
12	6,02	3,92	3,76	4,22	4,06	5,27	5,25
16	8,63	5,75	5,45	6,11	5,86	7,56	7,52
20	11,28	7,73	7,25	8,15	7,74	9,91	9,83
24	13,97	9,83	9,16	10,28	9,73	12,35	12,20
28	16,69	12,04	11,16	12,52	11,78	14,82	14,59
32	19,43	14,35	13,25	14,83	13,92	17,32	17,00
36	22,20	16,73	15,40	17,23	16,11	19,85	19,43
40	25,00	19,18	17,63	19,67	18,37	22,42	21,90
44	27,81	21,69	19,91	22,18	20,67	25,04	24,42
48	30,64	24,25	22,24	24,73	23,02	27,70	26,97
52	33,49	26,85	24,62	27,33	25,41	30,38	29,53
56	36,35	29,49	27,04	29,96	27,85	33,07	32,10
60	39,21	32,17	29,49	32,62	30,31	35,78	34,69
64	42,09	34,87	31,98	35,30	32,81	38,52	37,32
68	44,97	37,60	34,50	38,00	35,32	41,28	39,96
72	47,86	40,34	37,04	40,72	37,87	44,04	42,61
76	50,75	43,10	39,60	43,46	40,43	46,80	45,26
80	53,64	45,87	42,18	46,20	43,01	49,57	47,91
84	56,53	48,65	44,78	48,95	45,60	52,35	50,59
88	59,42	51,43	47,39	51,71	48,21	55,13	53,28
92	62,29	54,22	50,00	54,47	50,82	57,91	55,95
96	65,17	57,00	52,63	57,22	53,44	60,68	58,62
100	68,03	59,78	55,26	59,97	56,06	63,44	61,29
104	70,88	62,56	57,89	62,72	58,68	66,20	63,97
108	73,71	65,33	60,52	65,46	61,30	68,95	66,64
112	76,53	68,09	63,15	68,18	63,92	71,68	69,29
116	79,33	70,72	65,77	70,90	66,54	74,40	71,93
120	82,11	72,46	68,39	73,60	69,14	77,09	74,56
124	84,87	75,17	71,00	76,28	71,74	79,78	77,18
128	87,60	77,86	73,59	78,94	74,32	82,44	79,79
132	90,30	80,53	76,17	81,58	76,89	85,07	82,37
136	92,98	83,18	78,74	84,19	79,44	87,67	84,92
140	95,61	85,80	81,29	86,77	81,97	90,24	87,44
144	98,21	88,39	83,81	89,32	84,48	92,79	89,95
148	100,77	90,94	86,31	91,85	86,96	95,30	92,43
152	103,29	93,46	88,79	94,33	89,42	97,76	94,87
156	105,76	95,95	91,23	96,78	91,84	100,18	97,26
160	108,18	98,39	93,65	99,19	94,24	102,56	99,61
164	110,55	100,80	96,03	101,56	96,60	104,90	101,94
168	112,86	103,16	98,38	103,88	98,92	107,19	104,22
172	115,11	105,47	100,69	106,16	101,21	109,42	106,44
176	117,30	107,74	102,96	108,38	103,45	111,59	108,60
180	119,42	109,97	105,20	110,57	105,66	113,70	110,70
184	121,46	112,14	107,39	112,69	107,81	115,77	112,78
188	123,44	114,26	109,54	114,78	109,92	117,77	114,78
192	125,33	116,34	111,64	116,80	111,99	119,70	116,71
196	127,15	118,37	113,70	118,79	114,00	121,54	118,56
200	128,89	120,39	115,71	120,69	115,95	123,31	120,12

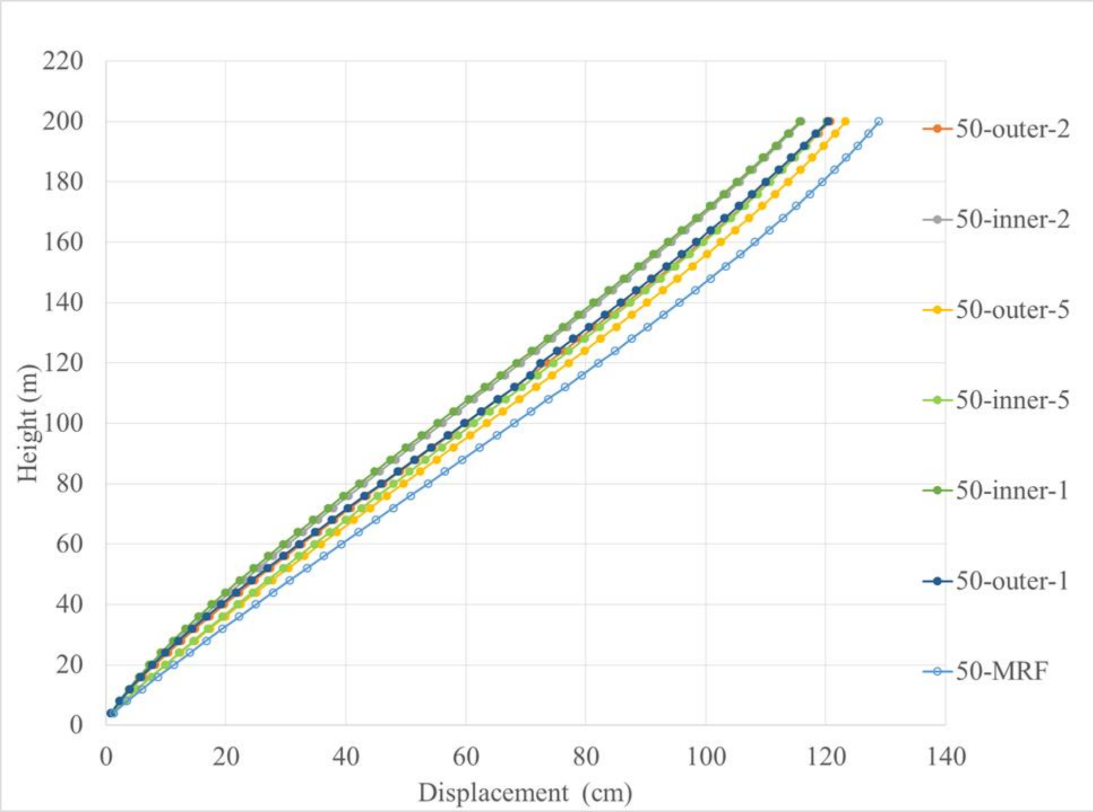


Figure 4.6 Distribution of displacement of the 50 storey MRF and retrofitted buildings with X-bracing in Y direction

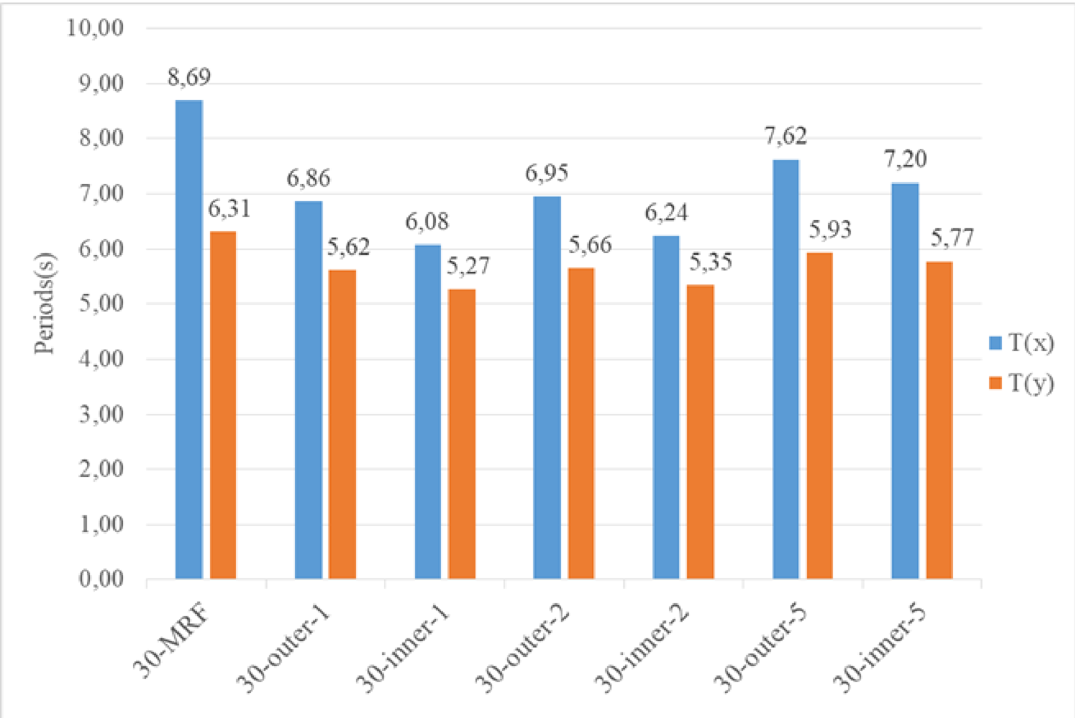


Figure 4.7 Distribution of periods of the 30 storey MRF and retrofitted buildings with X bracing

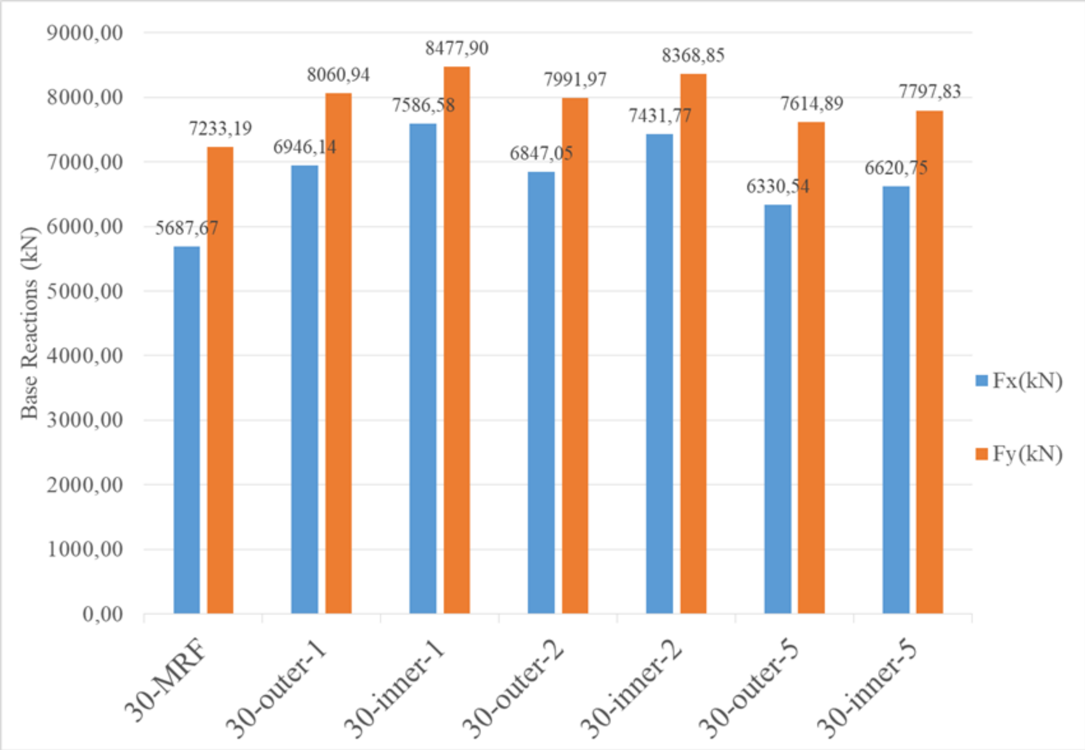


Figure 4.8 Distribution of base reactions of the 30 storey MRF and retrofitted buildings with X-bracing

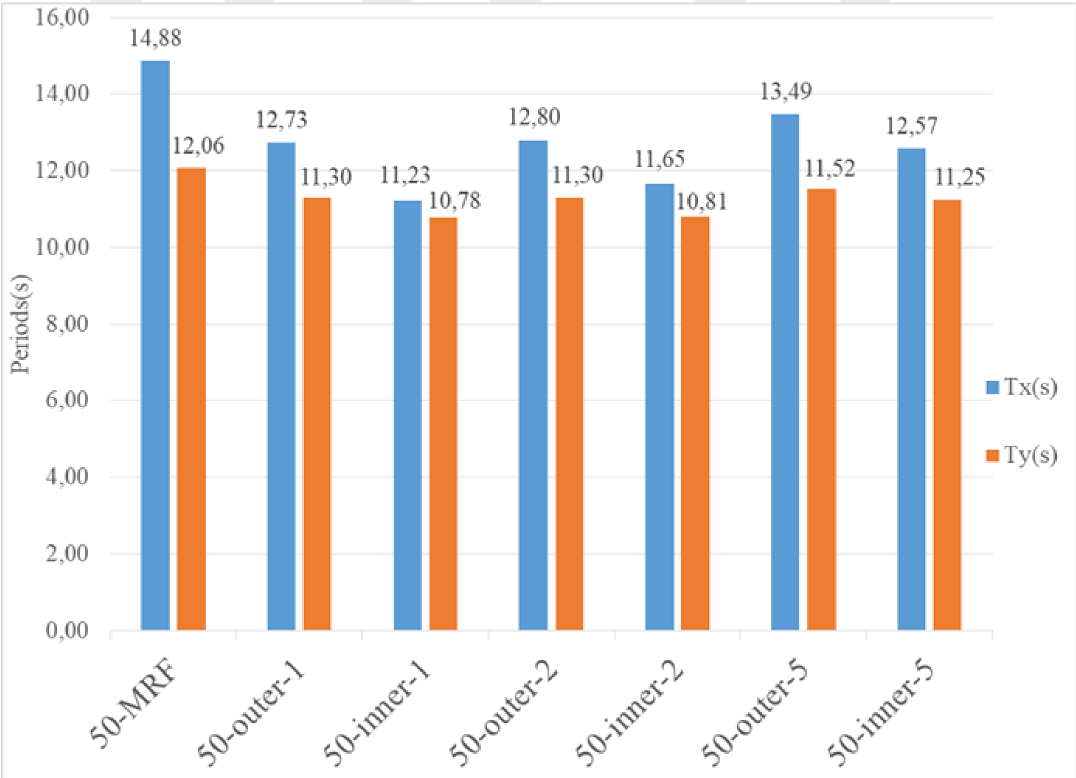


Figure 4.9 Distribution of periods of the 50 storey MRF and retrofitted buildings with X-bracing

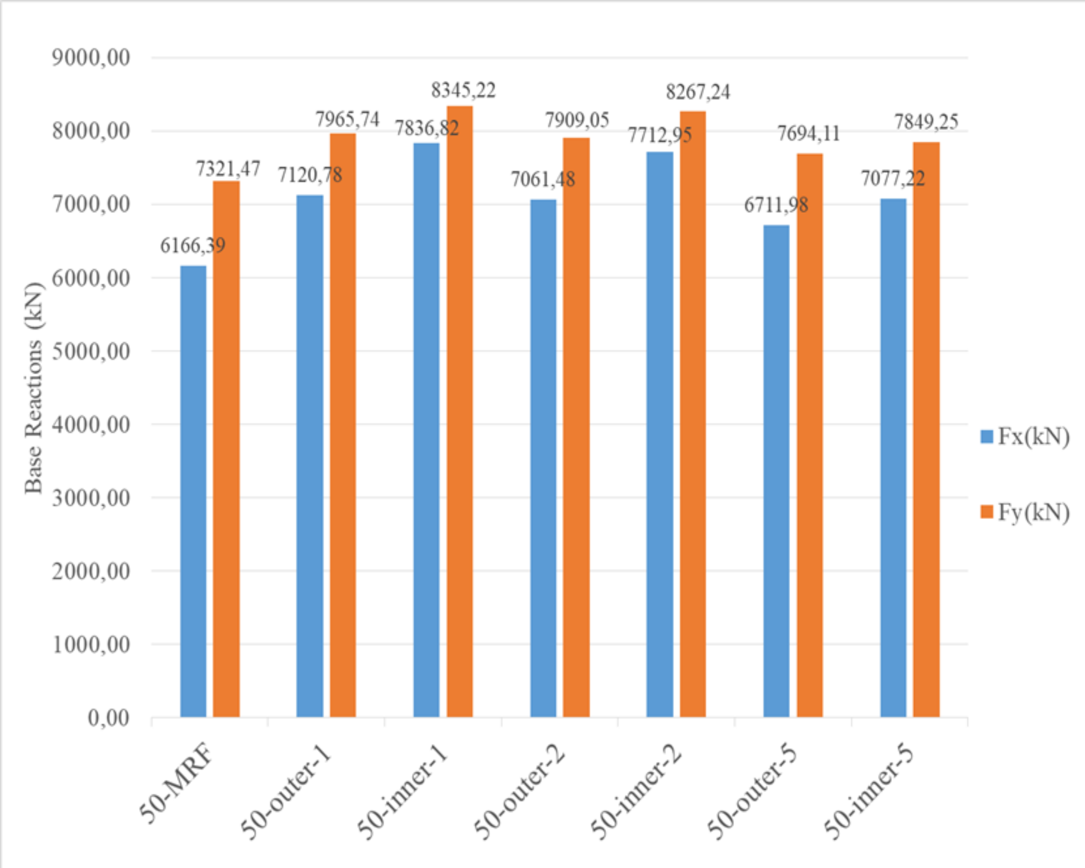


Figure 4.10 Distribution of base reactions of the 50 storey MRF and retrofitted buildings with X-bracing

4.2. Discussions

Compared to the results obtained after analysis from Sap2000 program using response spectrum method, it has been observed that steel braces significantly reduce periods and displacements of structures under earthquake loads. For 30 storey building as shown in Figure 4.3, the maximum displacement of the MRF in the X axis was achieved at 79.61 cm under earthquake loads, while the maximum displacement of the 30-storey MRF retrofitted with inner X brace every 1 storey (30-inner-1) equal to 55.40 cm. In the case of the 50 storey building as shown in Figure 4.5, the maximum displacement of the MRF in the X axis was achieved at 154.58 cm under earthquake loads, while the maximum displacement of the 50-storey MRF retrofitted with inner X brace every 1 storey (50-inner-1) equal to 115.02 cm. As a result, it was appeared that the use of the X brace every 1 storey was better than the other braced frames, in the case of using brace frame systems were decreased significantly the maximum displacement of the buildings.

The brace type, brace location and the number of stories were affected on the maximum storey displacement, the results for the 30-storey buildings are;

1. As shown in Figure 4.3, the maximum displacement of the 30-MRF in the X axis was 79.61 cm under earthquake loads, while the maximum displacement for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 64.45 cm, 55.40 cm, 65.01 cm, 56.66 cm, 70.33 cm and 64.72 cm, respectively.
2. As shown in Figure 4.4, the maximum displacement of the 30-MRF in the Y axis was 56.68 cm under earthquake loads, while the maximum displacement for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 51.12 cm, 47.53 cm, 51.36 cm, 48.10 cm, 53.13 cm and 51.41 cm, respectively.
3. As shown in Figure 4.7, the time period of the 30-MRF in the X axis was 8.69 s, while the time period for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 6.86 s, 6.08 s, 6.95 s, 6.24 s, 7.62 s and 7.20 s, respectively.
4. As shown in Figure 4.7, the time period of the 30-MRF in the Y axis was 6.31 s, while the time period for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 5.62 s, 5.27 s, 5.66 s, 5.35 s, 5.93 s and 5.77 s, respectively.
5. As shown in Figure 4.8, the base reaction of the 30-MRF in the X axis was 5687.67kN, while the base reaction for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 6946.14 kN, 7586.58 kN, 6847.05 kN, 7431.77 kN, 6330.54 kN and 6620.75 kN, respectively.
6. As shown in Figure 4.8, the base reaction of the 30-MRF in the Y axis was 7233.19 kN, while the base reaction for (30-outer-1), (30-inner-1), (30-outer-2), (30-inner-2), (30-outer-5), (30-inner-5) was 8060.94 kN, 8477.90 kN, 7991.97 kN, 8368.85 kN, 7614.89 kN and 7797.83 kN, respectively.

The results for the 50-storey buildings are;

1. As shown in Figure 4.5, the maximum displacement of the 50-MRF in the X axis was 154.58 cm under earthquake loads, while the maximum displacement for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 134.09 cm, 115.02 cm, 134.42 cm, 116.42 cm, 140.97 cm and 128.07 cm, respectively.

2. As shown in Figure 4.6, the maximum displacement of the 50-MRF in the Y axis was 128.89 cm under earthquake loads, while the maximum displacement for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 120.39 cm, 115.71 cm, 120.69 cm, 115.95 cm, 123.31 cm and 120.12 cm, respectively.
3. As shown in Figure 4.9, the time period of the 50-MRF in the X axis was 14.88 s, while the time period for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 12.73 s, 11.23 s, 12.80 s, 11.65 s, 13.49 s and 12.57 s, respectively.
4. As shown in Figure 4.9, the time period of the 50-MRF in the Y axis was 12.06 s, while the time period for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 11.30 s, 10.78 s, 11.30 s, 10.81 s, 11.52 s and 11.25 s, respectively.
5. As shown in Figure 4.10, the base reaction of the 50-MRF in the X axis was 6166.39kN, while the base reaction for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 7120.78 kN, 7836.82 kN, 7061.48 kN, 7712.95 kN, 6711.98 kN and 7077.22 kN, respectively.
6. As shown in Figure 4.10, the base reaction of the 50-MRF in the Y axis was 7321.47 kN, while the base reaction for (50-outer-1), (50-inner-1), (50-outer-2), (50-inner-2), (50-outer-5), (50-inner-5) was 7965.74 kN, 8345.22 kN, 7909.05 kN, 8267.24 kN, 7694.11 kN and 7849.25 kN, respectively.

In 30-storey buildings, maximum displacement value was formed in (30-MRF), while the lowest displacement value was observed in (30-inner-1). Also the same situation was observed in period values. 30.4% decrease in displacement value and 30.1% decrease in period value are indicative of the resistance and rigidity of brace members against horizontal loads. (30-inner-1) shows the best performance; it is thought that the brace members are located in the middle regions of the outer axes of the structures and are applied more frequently in floor intervals. The base reactions of (30-MRF) are also lower than the other models, indicating that the brace members give the structure rigidity.

In 50-storey buildings, maximum displacement value was formed in (50-MRF), while the lowest displacement value was observed in (50-inner-1). Also the same situation was observed in period values. 25.6% decrease in displacement value and

24.5% decrease in period value are indicative of the resistance and rigidity of brace members against horizontal loads. (50-inner-1) shows the best performance; it is thought that the brace members are located in the middle regions of the outer axes of the structures and are applied more frequently in floor intervals. The base reactions of (50-MRF) are also lower than the other models, indicating that the brace members give the structure rigidity.

It can be seen from the above results that the impact of the braces is clearly visible in reducing the maximum displacement and time periods of buildings. The maximum storey displacements, time periods and base shears were affected by the bracing location, number of stories and bracing type. It was appeared that the use of the X brace every 1 storey on middle location of building gives the best performance under seismic load.

CHAPTER 5

CONCLUSIONS

5.1. Conclusions of Study

In this study investigated the seismic analysis for moment resisting frame and structures retrofitted with bracing. The analysis is carried out using the response spectrum method. In this study, bracing system components were chosen to withstand seismic loads. SAP 2000 program was used to performing analysis and design. Data obtained from program were exported graphs and tables to compare for the study. The comparison is made by using the values obtained for displacement, base shear and time period of buildings. After examining the data obtained, the following conclusions are reached in this research:

1. As can be seen from the results of the analysis, it was observed that the use of braces in steel structures showed a significant decrease in the periods and displacements of the structures. This situation shows that the brace members can be used successfully to increase their strength and stiffness properties against horizontal loads.

2. In high rise steel structures strength and stiffness are more significant, so it is possible to decide a bracing system are adopted to enhance both these parameters. MRF buildings show higher storey displacement and time period indicating that MRF buildings are more ductile than other braced buildings. Therefore, excessive damage is likely to occur during an earthquake event.

3. Since the mass of the building will increase as the number of storeys increases, an increase is observed in the earthquake load affecting the building in parallel with the increase in the number of storeys. Base shear forces increase due to increasing seismic force.

4. The maximum displacement of the 30 storey frame retrofitted with inner X brace every 1 storey was smaller than other frames, and the decrease in the number of

storey's leads to a decrease in the displacements. Also, location of braces are affect the displacements. For example, maximum displacement of the frame retrofitted with braces in inner location smaller than frame retrofitted with braces in outer location.

5. The time period of the frame retrofitted with X brace every 1 storey was smaller than other frames and the time period of the moment resisting frame greater than other frames. This situation shows that the X brace every 1 storey exhibits the best performance.

6. The results show that using of braces increase strength and stiffness of frame and accordingly increase the base reactions of buildings. Due to the placement of the columns the maximum displacement in the long direction is greater than the displacement in the short direction of the building.

5.2. Future Studies

The results of this study will help the engineers in research the selection of the best steel bracing type and it’s positioning in the building geometry to keep the lateral displacements of the high-rise steel structures at low levels under dynamic loads using the response spectrum analysis. The proposed results calculated in this study need to be verified by further case studies, for example;

1. The same research can be done by changing the height of the buildings using other software.

2. Seismic responses with different configurations of various types of braces with different location of building can be studied.

3. Future research should look for the best brace frame structure with all types of advantages.

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