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IMPROVING LATERAL DEFORMATION BEHAVIOR OF HIGH RISE STEEL STRUCTURES UNDER DYNAMIC LOADINGS

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Improving Lateral Deformation Behavior of High Rise Steel Structures Under Dynamic Loadings

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Mustafa Mohammed RASHID

ABSTRACT

IMPROVING LATERAL DEFORMATION BEHAVIOR OF HIGH RISE STEEL STRUCTURES UNDER DYNAMIC LOADINGS

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M.Sc. in Civil Engineering

Supervisor: Assist. Prof. Dr. Talha EKMEKYAPAR

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The present study assesses the effect of dynamic load on steel building for different type of bracing. There are exist many researches on the effect of earthquake dynamic load on the rectangular buildings and a few or limited studies discuss the effects on circular buildings. Our study will treat high rise steel rectangular and circular buildings 10, 20 and 30 stories under dynamic load represented by the response spectrum model, and searching for the best bracing performance to resist the deformation in structural parts. For bracing in this study, we use the concentrically bracing and the eccentrically bracing types in different locations to find and search the best performance of it. The structural models will be subjected to a certain level of earthquake conditions depend on TEC 2007. The finite element models were built using SAP 2000 program in this study. Comparisons of the results showing that type and location of bracing lead to different performances for buildings under lateral loads. Understanding the underlying mechanics of this behavior will help engineers to consider more effective ways to design seismic resistant high rise steel buildings.

Keywords: High rise steel buildings, Earthquake analysis, Concentric bracing, Eccentric bracing, Response spectrum analysis.

ÖZET

YÜKSEK KATLI ÇELİK YAPILARIN DİNAMİK YÜKLER ALTINDA YANAL DEFORMASYON DAVRANIŞLARININ İYİLEŞTİRİLMESİ

RASHID, Mustafa Mohammed
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Bu çalışma farklı tiplerde çelik çapraz elemanlarla güçlendirilmiş çelik yapılarda dinamik yüklerin etkilerini değerlendirmektedir. Dörtgen kat planlarına sahip yapılar üzerinde dinamik yük etkilerini inceleyen birçok çalışma bulunmasına karşın dairesel kat planına sahip yapılar ile ilgili sınırlı sayıda çalışma bulunmaktadır. Bu çalışma 10, 20 ve 30 katlı dörtgen ve dairesel kat planlarına sahip yüksek çelik yapıların dinamik davranışını inceleyip yanal deplesmanları düşük seviyelerde tutmak için en iyi çelik çapraz çeşidini araştırmaktadır. Çelik çapraz takviyeleri için bu çalışmada merkezi ve dış merkezli çelik çapraz tipleri kat planında değişik yerlerde konumlandırılıp en iyi performans araştırılmıştır. Kurulan modeller TDY 2007 (Türk Deprem Yönetmeliği 2007) ilkelerine bağlı kalarak belirli deprem şartlarına maruz bırakılmıştır. Sonlu eleman modelleri SAP 2000 bilgisayar programı kullanılarak kurulmuştur. Karşılaştırmalı sonuçlar çelik çaprazların tipi ve konumunun yanal yükler altında değişik performanslara sahip olduğunu göstermektedir. Bu davranışların mekanik ilkelerinin anlaşılması mühendislerin yüksek katlı çelik yapılarda deprem yükleri için daha etkili yöntemleri göz önüne almalarına yardımcı olacaktır.

Anahtar kelimeler: Yüksek katlı çelik yapılar, Deprem analizi, Merkezi çelik çapraz, Dış merkezli çelik çapraz, Tepki spektrum analizi

To my beloved father, family, and my country

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LIST OF SYMBOLS

MRFs: Moment resisting frames

CBFs: Concentrically braced frames

CB: Conventional braces

BRB: Buckling restrained brace

SCBF: Special concentrically braced frame

OCBF: Ordinary concentrically braced frame

SMFs: Special moment frames

EBFs: Eccentrically braced frames

MRFs: Moment resisting frames

LFRS: Lateral force-resisting system

RS: Response spectrum

CQC: Complete Quadratic Combination

GMC: General Modal Combination

SRSS: Square Root of the Sum of their Squares

AISC: American Institute of Steel Construction

MDOF: Multi degree of freedom

SDOF: Single degree of freedom

TEC: Turkish Earthquake Code

CSI: Computers and Structures, Inc

M: the mass matrix.

C: the damping matrix.

f: the story force vector.

$S_{A\,max}$: Maximum spectral acceleration of the earthquake.

$S_{V\,max}$: Maximum speed spectral of the earthquake.

f_r : Solid frequency for input seismic movement data.

m_x , m_y , m_x : Loads in the global axis.

u''_{gx} , u''_{gy} , u''_{gz}: Acceleration of gravy at (t) in the global direction.

C: is a viscous damping matrix

K:.the static stiffness matrix for the system of structural elements.

u(t), **u**'(t) and **u**"(t): the time-dependent vectors.

A(T): The Spectral Acceleration Coefficient.

$S_{ae}(T)$: The elastic spectral acceleration.

A_0: The effective ground acceleration coefficient

(*I*): The building importance factor

S(T): the spectrum coefficient

T_A , T_B: Spectrum characteristic periods.

T: the building natural period

$R_a(T)$: Seismic Load Reduction Factor.

(R): Structural Behavior Factor.

CHAPTER ONE

INTRODUCTION

1.1 General Introduction

The important point in seismic design standards to keep the human life safety when strong earthquakes happen, will be through the development standards of design and implementation of buildings, and to reduce the risk of human life inside the buildings when these buildings are being under the earthquakes, and to maintain the important buildings on the performance of its functions during the earthquake. Earthquakes cause the building cracks, but the design values of earthquakes and the criteria that built upon it must keep the buildings from collapse.

Earthquakes pose a unique engineering problem, especially in the case of earthquakes that have a high strength when buildings be under maximum load represent the biggest risk which might happen to the structure. The probability that the structure is exposed in the default lifespan period to like this earthquake is very rare cases. This is the cause that leads to give specifications designed earthquakes to secure the lowest level of the design allows for the buildings to:

1. Resistance to weak earthquakes without damage
2. Resistance to moderate earthquake intensity without receiving significant structural damage, and allows obtaining some of the damage nonstructural
3. Resistance to high intensity earthquakes without collapse of the structural system, and allows obtaining strong structural and non-structural cracks in the building.

The objective of earthquake design is to give structure suitable stiffness and strength to resist lateral forces that happen by earthquake without any structural problems, and to keep structure in the elastic range without exceeding the drift requirements, also to prevent structures from collapse when it exposed to strong earthquakes. Consequently preventing the losses of human lives and reduced the financial losses.

The difficulties lie in identifying accurately the amount of force caused by the ground movement and its impact on structures, and prepare the structure to respond in an acceptable manner against any movement of the ground (Mazzolani, et al. 1995).

1.2 Steel as a Construction Material

The steel is used for more than 150 years as a construction material for structural buildings in the modern world, and at the end of the 19th century the world knowing the first type of standard sections of steel, this led to increase in the reputation of the steel as a structural material for high-rise buildings. Different shapes and sizes with appropriate thickness have been manufactured of steel sections, and steel sections can be readily fabricated, connected and combined in different ways, giving many options for designers to choose a suitable section for structures, and that give designers a spacious assortment of design options with unbounded potential of the designs. For owners and constructor there are some benefits associated with the use of steel construction, which are (AISC 1999):

- Steel allows for reduced structural construction time and the ability to construct in all seasons
- Steel builds large spans and bay sizes as needed, that give more flexibility for owners
- Steel is easier to modify and enhance if architectural changes are made in a facility over its life
- Steel is a durable with long-lasting and a recyclable
- Steel is a lightweight so can reduce foundation costs.

1.3 Properties of Steel as Structural Design Material

Steel is most widely construction material that used in modern high rise building around the world. This material has some properties may make the steel as favorite construction material, which are:

1. The ratio of strength / weight; when it has high amount of strength / weight ratio - that what is happening the dead load (dead weight) of steel structures is

comparatively less, this feature makes steel the first selection for long-span beams, high-rise structures and the construction in seismic areas

2. Ductility; steel may get big plastic deformation prior to failure, thus it allows for a large appropriate strength. Decently design of steel structures can have high ductility, which is a significant diacritic for resisting vibration loads like an explosion or seismic loads. The ductility of structure has energy-absorbing capability and won't appear sudden failure, it normally, appears a large obvious deflection before failure or breaking down
3. Predictability of material properties; the steel characteristics can be anticipated with a high degree of certainty. Steel in reality shows elastic behavior, adequate to a relatively high and generally well-defined stress level. Else, in contrast to reinforced concrete, the steel characteristics don't change so much with the time.
4. Speed of construct and repair; steel structures could construct quickly, this leads to faster returns in economic benefits. It also can be repaired swiftly and readily, and with the possibility of expansibility existing structures easily by adding new sections like wings or beams. Steel has a unique property of reusable that can use it again after dismantling of the structure
5. Quality and qualitative of steel structure; with high-quality and narrow tolerances can be built, and produce a prefabricated structural part that is suitable for prefabrication and huge production
6. Fatigue strength; steel structures have relatively good fatigue strength

Like other construction materials, steel has some points can be considered as drawbacks related to steel as a construction material in particular cases, which are:

1. Corrosion problem; it is the main problem for steel material because the direct exposure to natural circumstances, air and water making it liable to corrode. So, steel was needed to be treated with paint regularly to mitigate this problem.
2. Fire-resistance; when steel is exposed to high temperature that most often caused by building fires, that leads to decrease the steel strength, heat transfer between burning parts of the building is happening quickly. So, fireproofing procedure is one of the important conditions that must be met in steel buildings

3. Portability of buckling; because of high ratio of the strength/weight. Usually, the steel compression members are more portable to buckling than reinforced concrete compression members due to the slenderness of steel members.

1.4 Steel Frame Structure

Steel buildings today have become a common construction when you visit any big city in civilized areas. Commercial, industrial and residential buildings, built from steel or composite material like steel frame with reinforced concrete as shown in Figure 1.1. The buildings that built from wood or concrete can create with the same design and performance of steel with parallel specifications or better. Steel framed structures can be classified into several types (MacGinley, 1998):

1. Single-story, single- or multibay structure, there are many cases could regarded as this type like truss, stanchion frames, rigid frame of solid and lattice Members
2. Multi Story, single- or multibay structures, this type of steel structure could be braced or rigid frame construction system
3. Space structures like space decks, domes and towers are space decks and domes (except the Scheduler dome) are redundant structures
4. Tension structures and cable-supported roof structure
5. Stressed skin structures that cladding proving the structure

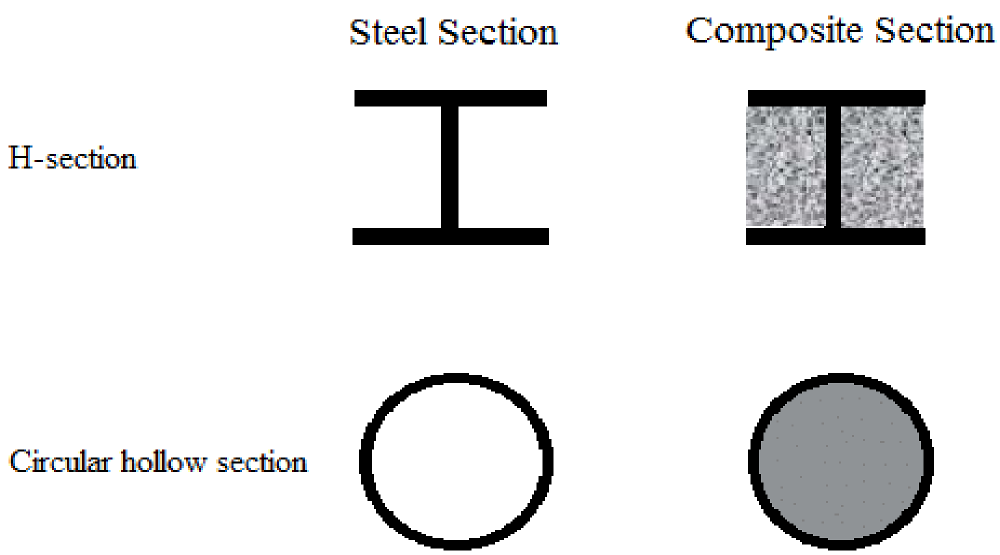


Figure 1.1 Steel frame section and composite section

As mentioned previously, that combinations structure which consists of the steel frame and concrete are very important therefore, used in numerous buildings. Configuration of some type of framed steel structures shown in Figure 1.2 like Braced, rigid frame, truss roof and space deck construction shown for comparison. For the framed structures the main elements are the beam, column, tie and trellis members. Beams and columns can be rolled or built-up from one or more sections I, H or box can use for that.

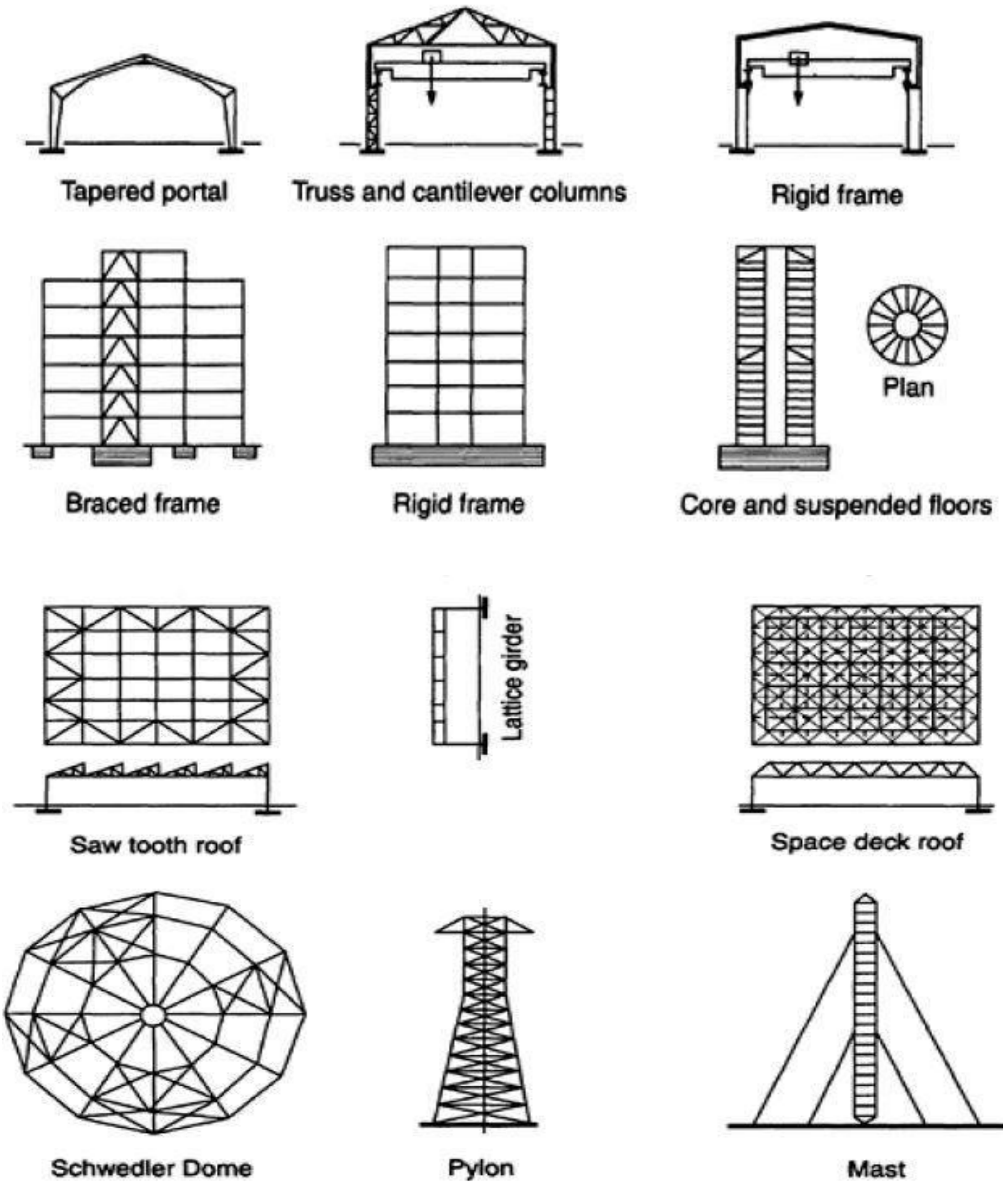


Figure 1.2 Some type of framed steel structure configurations (MacGinley, 1998).

1.4.1 Steel braced frame

Steel braced frames are structural systems used in multi-story buildings to improve behavior of buildings for resisting lateral loads, the main cause of lateral loads are wind loads and seismic loads. Steel bracing is easy to construct and economical compared to other ways to strengthen buildings, meet the design requirements strength and stiffness by virtue of flexibility that owned, this system very often are placed perpendicularly or diagonally, also it doesn't take a large space.

Steel braced frames permit to get a significant increase of stiffness with a slight increase in weight, so it's considered as a wholesome solution for existing buildings that suffer from weakness in the lateral load. Usually, the bracing provide an increase in stiffness and stabilization of structures under the influence lateral loads, as well as to reduce the effect of lateral displacement dramatically.

Steel braced frame divided into two main types, are concentric and eccentrically as shown in Figure1.4 and Figure1.4. The concentric bracing provide increased in lateral stiffness of the frames, also decrease the lateral drift. But, the increase of stiffness it may contribute to increase the inertia force during the earthquake effect. Moreover, bracing could decrease the shear forces and bending moment in columns, in the same time lead to increment axial compression on columns that connected with it.

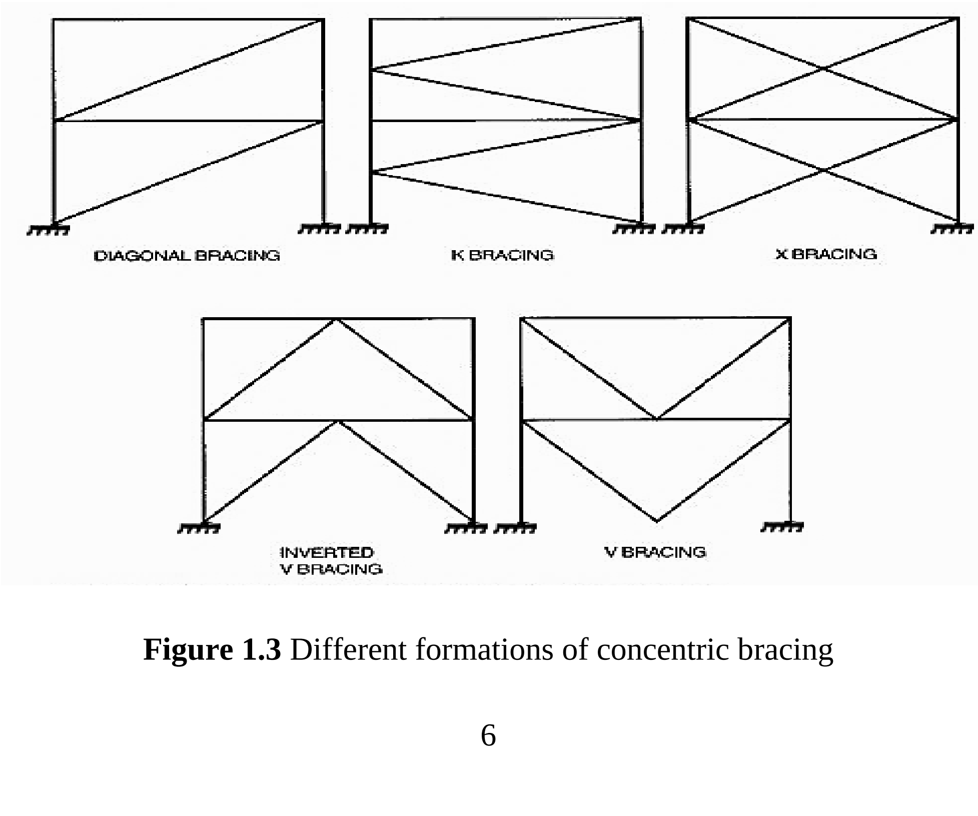


Figure 1.3 Different formations of concentric bracing

Eccentrically braced frames; it describes a way of connection between the braces to beam, and it is improving the ability of dissipating energy and decreases the lateral stiffness of the system. Lateral stiffness of the system rely on the flexural stiffness of the beams, and the earthquake force dissipates through the vertical component of the bracing, causing of a lateral massive load on the beams at the point of connection (Tafheem & Khusru, 2013).

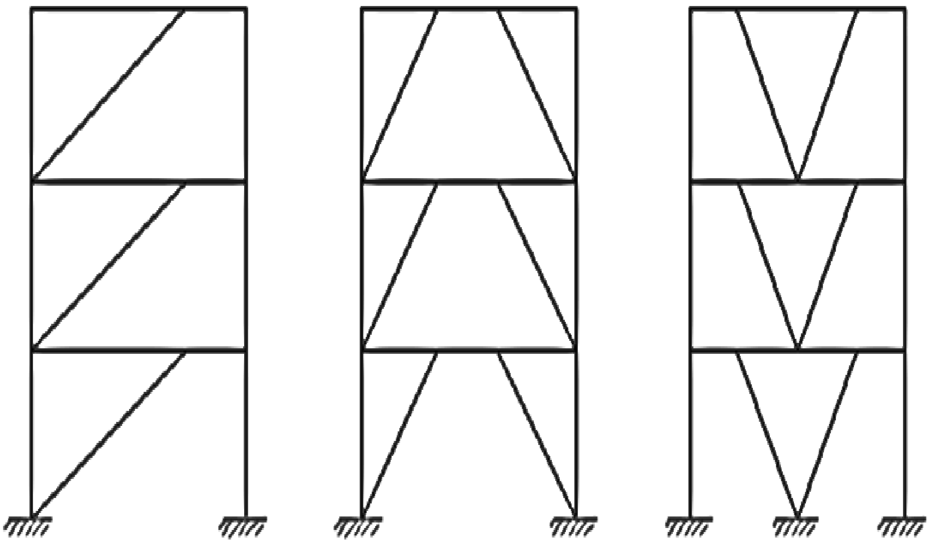


Figure 1.4 Different formations of eccentrically bracing

1.5 Objectives and Research Tasks

The principal purpose of this study is to compare the seismic performance of the existing steel buildings with different type of bracing in different locations. The case study examines 10, 20 and 30 story steel buildings. The buildings have 2 types of plan. The structures were modeled by using a finite element method and evaluated by responses spectrum. The seismic response of the original frames and those with braces were analyzed using earthquake ground motion, the proposal the use of seismic equation in the analysis is to find the period and acceleration that the most important conditions in seismic analysis, depend on the Turkish Code of Specification for Buildings to be Built in Seismic Zones /2007.Consequently, every detail of lateral movements will be explored, and the methods of controlling of this behavior will be refined from point of civil engineering view**.**

1.6 Organization of the Thesis

This dissertation is comprised of five chapters:

Chapter 1-Introduction: The aim and objective of the thesis are summarized.

Chapter 2-Literature Review: This chapter is briefly given the background on the previous studies about steel bracing systems.

Chapter 3-Methodology: In the methodology, the analytical model of the existing structures (10, 20 and 30 stories) is explained. The description of the analytical methods and the characteristics of the ground motion records are also provided.

Chapter 4-Results and Discussion: This chapter presents and compares the results obtained from Sap2000 program of each structural system.

Chapter 5-Conclusions: The conclusions are given in the light of the findings of the results of the analysis.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

The design of buildings to resist earthquakes requires designing for static loads and dynamic loads. Magnitude of earthquake dynamic characteristics cannot determine for a structure. This indeterminacy is due to the compounding of uncertainties in the earthquakes themselves with those of the soil and geology and the building characteristics. Certainly, there is, and has been, considerable studies undertaken to decrease the level of uncertainty in all of the above factors and this work is valid and continues to improve our knowledge.

Structural steel is the most outstanding material available to meet the requirements of seismic design. The most common used in earthquake resisting building systems for major structures are:

- Moment resisting frames
- Steel plate shears walls
- Steel concentrically braced frames
- Steel eccentrically braced frames
- Steel plate and concrete composite shear walls

The traditional seismic force resisting systems previously listed, while still appropriate and in use for many structures, frequently have liabilities in responding to one or more of the above factors. For example, for large open bay structures, moment frames are frequently uneconomical due to the large member's required to limit lateral drift. Concrete shear walls, while excellent for limiting drift in low and mid-rise structures, have severe architectural liabilities for many structures and, due to high forming costs, are frequently uneconomical. Steel braced frames were excellent for limiting drift, have questionable post-elastic performance, particularly when used in large heavy structures.

The key to this performance by apparently "under-designed" structures is primarily the post-elastic behavior of the materials and systems. Given materials able to provide large inelastic strains without failure, and systems which preclude instability and brittle connection fracture, large amounts of earthquake input energy can be dissipated by local yielding of the structure, and still stable without failure (Saunders, 1985).

The brittle fracture for the beam to column and braces to column connections that have resulted in decreasing the performance and energy dissipation capacity under earthquake excitation is shown in Figure 2.1. As a consequence beam-to-column connections and braces may be negligent in ductile moment resisting frames (MRFs) or concentrically braced frames (CBFs) if they are not sufficiently capacity designed (Bruneau et al., 1998; Tremblay, 2002; Broderick et al., 2005).

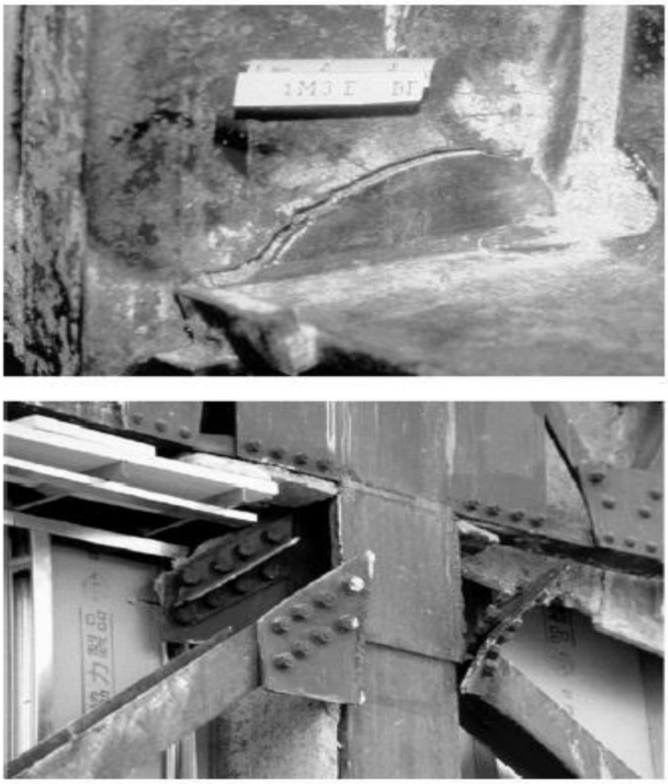


Figure 2.1 Fracture in beam-to-column connections in the Northridge earthquake (top) and web tear-out in bolted brace-to-column connections during the 1995 Kobe earthquake (bottom) (Di Sarno & Elnashai, 2009).

In retort to various practical and economic matters, engineers are revolving to the use of braces steel frames. Whenever hysteretic dampers are utilized, it is anticipated that the braces can intensify the energy absorption of structures and/or reduce the demand levied by earthquake loads. Structures are expected to resist safely the lateral load induced by an earthquake and avoid the risk of brittle failure if their energy absorption capacity is augmented. Design demands of structural and nonstructural component are conceived to be smaller than their capacity when global modification is applied as shown in Figure 2.2 (Di Sarno & Elnashai, 2009).

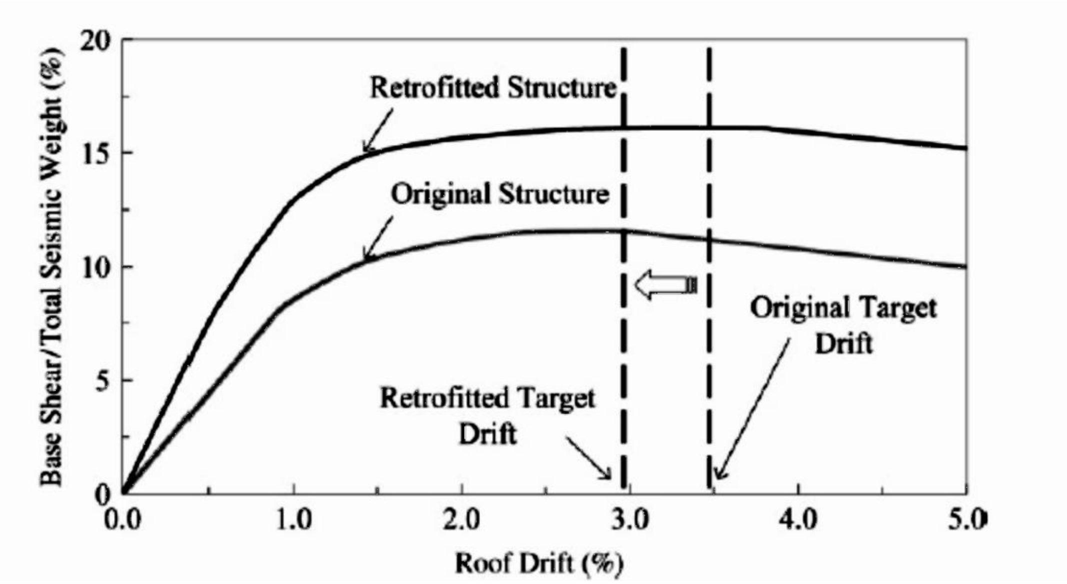


Figure 2.2 Characteristics of global intervention attitudes in seismic retrofitting of structures (Di Sarno & Elnashai, 2009).

In conventional braced frames, bracing is considered a force effect on the structures that dissipate seismic energy through yielding in both tension and compression. However, due to potential problems and difficulties aroused from buckling deformation of the conventional braces (CB), the thought of buckling restrained brace (BRB) behaved out to improve compressive ability and achieved more favourable behavior. BRB is exhibit stable and balanced hysteresis behavior by cooperative ductile compression yielding before the onset of buckling (Asgarian & Amirhesari, 2008; Mahmoudi & Zaree, 2010).

Three-story frame was designed and examine by shake-table testing as shown in Figure 2.3. One of the main purposes of the structural fuse concept being to concentrate seismically induced damage on disposable elements, this experimental project assessed the replaceability of BRB designed as sacrifice table and easy-to-repair members. BRB replaceability was examined in a test-assessment-replacement-test sequence. BRB were also connected to the frame using removable and eccentric gusset plates as shown in Figure 2.3 (Bruneau, 2007).

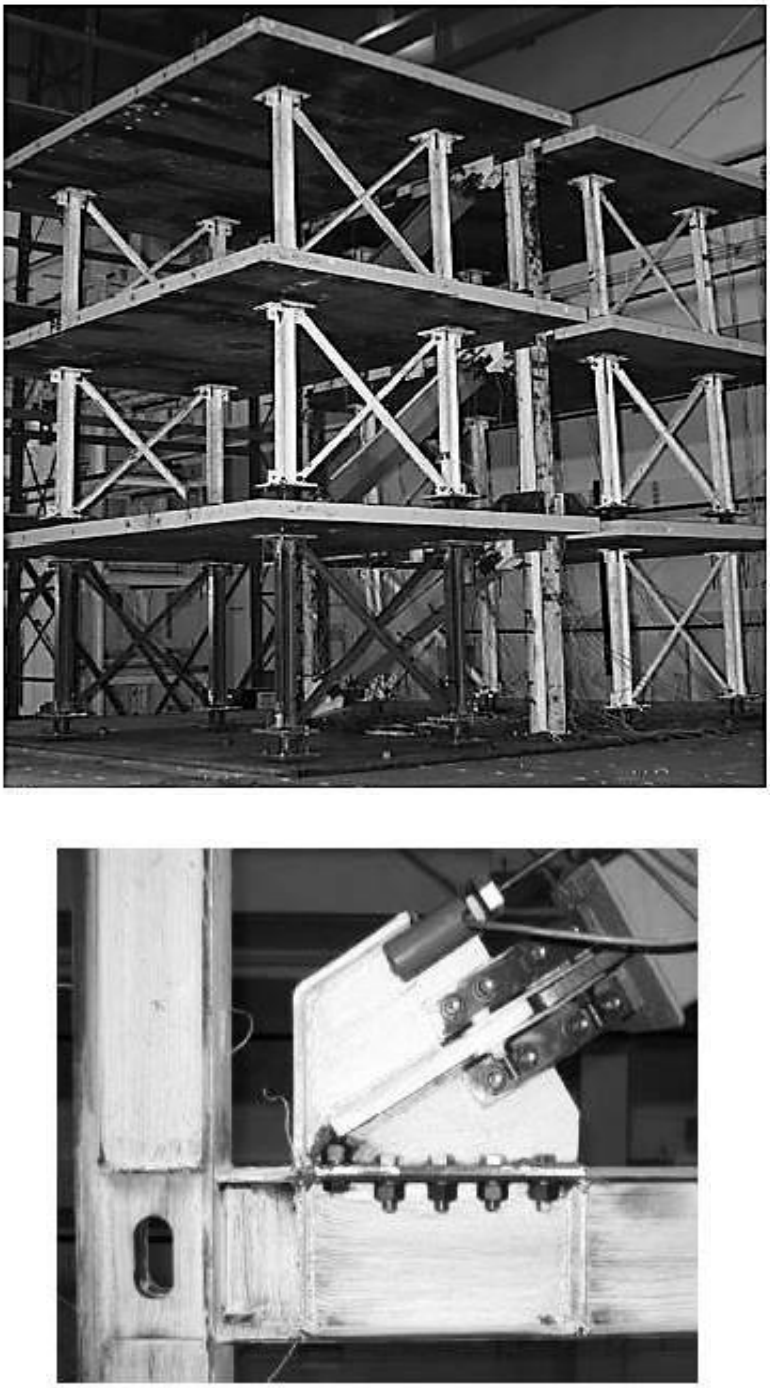


Figure 2.3 Three-story shake-table test specimen (top), removable and eccentric gusset plates (bottom) (Bruneau, 2007).

The researcher examined three concentrically-braced sub-frames using the shake table tests as shown in Figure 2.4. And these frames were compared with a series of correlation inelastic analyses. The brace cross-section was changed between tests to investigate the influence of brace slenderness on the stiffness, resistance and ductility displayed in the frame under strong earthquake loading. Time-history and nonlinear static analyses were considered on three concentrically-braced sub-frames. Response simulations using a two-dimensional analytical model of the test frame were compared with the experimental results (Broderick et al., 2008).

In their study, for three single-storys braced frames it was conducted experimental study using shake table and analytical study. Also, it was calculated the base shear–frame drift relationship by nonlinear static analysis and the acceleration response of the frame using time-history analysis. The results showed that the axial force in a brace for experimental and analytical study had similar level. The pushover analysis result provided an accurate envelope of the observed hysteresis curve, with slightly better agreement being achieved when both the compression and tension brace were included in the structural model.

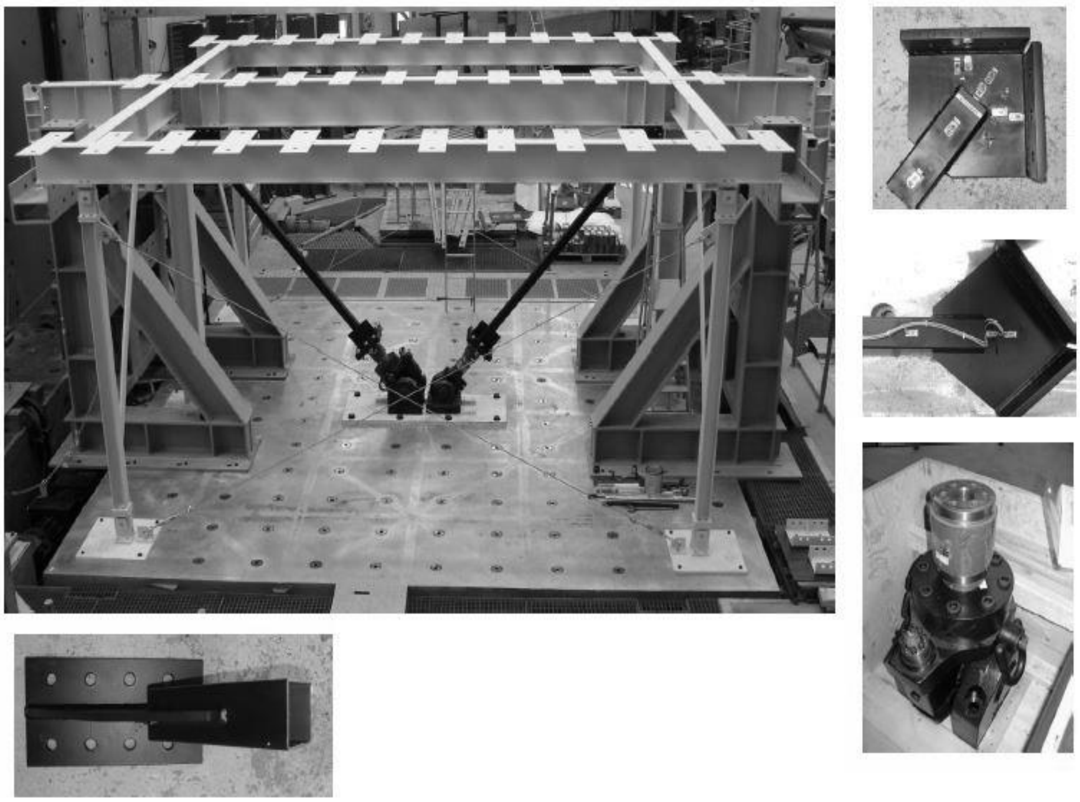


Figure 2.4 View of test frame (Broderick et al., 2008).

2.2 Concentric Braced Frames

One of the earthquake resistant systems for building is CBF, it is found to be economical system of lateral bracing for low to moderate height buildings of relatively light weight. Use of these frames for larger and heavier structures has become more prevalent largely due to their inherent lateral stiffness to meet the new drift limitations, economic and convenience as a seismic resisting system in rehabilitation of existing concrete and masonry structures. CBFs frame is one type of the frame sections. The buckling of first story braces in compression controls the behavior of this system, resulting in failure and loss of lateral resistance. Unexpected failure of steel structures during past strong seismic load excitation led to full fill adequate for modern structures in seismic areas.

Up to the 1994 UBC, The CBF was dealing with the codes as basically elastic truss systems. Post-elastic behavior, it was just deliberated within recommended a decrease in computing brace strengths, which leads to increasing the elastic force capability of these systems. (Astaneh et al. 1985; Hassan & Goel. 1991; Goel, 1992). This further ductile braced frame systems can accomplish tri-linear hysteretic behavior, with the three varieties of behavior exists the elastic, post buckling and tensile yielding ranges (Bruneau et al., 1998; AISC 2005).

CBF maintains a lateral load in the elastic range for the truss systems basically during axial forces in members. It meets at a point or with small eccentricities which does not a cause of inelastic deformation. Braced frames in the inelastic range, may concern the flexure of frame members, nevertheless the inelastic drift is predictable to serve as a basis a consequence of brace axial deformation, and with the exception of in certain formations that are not suggested (SEAOC, 2001).

The slenderness ratio of bracing members is an important parameter regulatory factor in braced frame structures (Rahgozar & Humar, 1998). The estimating of over strength, ductility and response modification factors of the chevron type concentric braced frames with varied stories and span lengths by using pushover analyses (Kim & Choi, 2005).

In a survey of past experimental studies on the inelastic response of diagonal steel bracing members, exposed to cyclic inelastic loading was done to gather data for the seismic design of concentrically braced steel frames for which a ductile response is essential under earthquakes. The observed parameters are the buckling strength of the bracing members, the brace post-buckling compressive resistance at various ductility levels, the maximum bracing tensile strength comprising strain hardening effects, and the lateral deformations of the brace upon buckling. Equations are planned for each of these parameters. So, the maximum ductility that can be reached by rectangular hollow bracing members was inspected (Tremblay, 2002). The subject of systems with special ductility has been offered since 1997 with most codes and provisions like SEAOC-1999, BSSC-1997, Fema 450, Fema 2003 and AISC 1997.

The current approach, which is under study for adoption in the codes is expected to include a reduced capacity for braces, based on study of cyclic load test data for compression members. The result is anticipated to be reduction of normally used capacities of long, slender braces (length divided by the radius of gyration in the range of 120) of 1/2 or more. Shorter braces will suffer lesser reductions. Also included in the revised codes will be required improvements in brace connection details which will greatly decrease the probability of brittle connection failure. These improvements will increase the safety of CBFs systems and possibly foster even wider use of them.

The researchers divided the CBFs into two main groups, special concentrically braced systems (SCBF) and ordinary concentrically braced systems (OCBF) (Tehranizadeh, Taghikhani, Kioumarsi, & Hajnajafi, 2011).The CBFs classify as either ordinary or special. The OCBF do not have extensive qualifications regarding elements or connections, they are generally utilized in low seismic risk areas (Sabelli et al., 2013). Limited ductility and low energy dissipation capacity because of braces buckling, defeat of connections and unsymmetrical behavior of the braces in tension and compression. Observant the defects for OCBF, seismic design needs for braced frames were altered and the idea of special concentric braced frames was established (Sabelli et al., 2003). Special frames (SCBF) are generally employed in areas of high earthquake risk, the goal of the CBFs design is ensured satisfactory ductility (i.e., to stretch without breaking suddenly) (Sabelli et al., 2013).

The ratio of width-thickness (λps) for OCBF should be bigger than the ratio of SCBF braces to permit of used as SCBF brace sections (AISC 2005). The braces could reach the plastic deformation and plastic hinges if get a reduction in braces width-thickness ratio for SCBF systems have been made in braces before they get under buckling. End connections and connection plates should satisfy the strength requirements according to use code. The inelastic behavior of braces likes plastic-buckling and tensile yielding are guaranteed by all these requirements. SCBF systems could sustain large inelastic deformations without significant strength reduction (Tehranizadeh, Taghikhani, Kioumarsi, & Hajnajafi, 2011):

1. Tensile-brace: that the ductile portion is the whole length of bracing member.
2. Compressive-brace: that the inelastic bucking causes plastic hinges in two ends and the middle of the bracing member.

It has been recognized for some time that these systems have inherent liabilities in the post-elastic range, since the majority of yielding and therefore energy absorbing capability is concentrated on the brace elements which alternate between tension and compression. The tension yield of the brace results in decreased compression capacity and stiffness of the brace with each successive cycle. It leads continually increasing deflections and possible eventual failure.

The above noted-weakness of this system previously accounted for in the Uniform Building Code in a rather arbitrary fashion by requiring that members and connections of brace frames be designed for forces 25% larger than those obtained from the code of seismic analysis. While perhaps qualitatively correct, quantitatively this increase had no rational basis. Because of the increased use of braced frames in larger, heavier and more important structures, it was felt that improved design requirements were needed (Saunders, 1985).

Studies have specified that the confidence level of accomplishing collapse prevention performance for typical SCBF can be tremendously and unsatisfactorily low when compared with special moment frames (SMFs) (Shih-Ho et al, 2008). The present results of a study in which a newly developed performance-based plastic design methodology was useful to CBF with buckling type braces. Initially the method was established and effectively applied to moment frames and more lately applied to other steel framing systems. The design indication uses pre-selected target drifts and yield mechanisms as performance limit shapes. The design lateral forces are resulted with an energy balance calculation where the energy required pushing the structure up to the target, drift is deliberate as a fraction of elastic input energy which is attained from the nominated elastic design velocity spectra, then plastic design is done to detail the frame members and connections to attain the planned yield mechanism and behavior.

The nonlinear dynamic analysis is applied in a three-story CBF, and it is verified that a drift concentration is predictable, especially in the first story, whenever the CBF is exposed to a set of large ground motions with a chance of an increase of two percent in fifty years. A basic speculative formulation is obtained to describe the outcome of gravity columns. Two cases are measured: gravity columns ideally fixed, and gravity columns pinned at the base. It is well known that major palliation of drift concentration can be attained, mainly when gravity columns are fixed at the base. Nonlinear time-history analysis is carried out to authenticate the theoretical remarks and count the stiffness/strength requests of gravity columns to evade drift concentration (Xiaodong et al, 2009).

The researcher tried to calculate the overstrength of the concentrically steel braced frames (CBFs), bearing in mind the reserved strength and due to members post-buckling. Therefore, a static nonlinear (pushover) analysis has been achieved on the model buildings with single and double bracing bays, diverse stories and brace arrangements (chevron V, inverted V and X-bracing). It has been comprehended that the number of bracing bays and the height of buildings have a low effect on reserve strength as a result of brace post-buckling. On the other hand, these parameters have a deep effect on the overstrength factor. These outcomes show that the overstrength values for CBFs, suggested in seismic design codes, need to be improved (Mahmoudi et al, 2011).

Single or multiply braced bays can be divided to a reasonable pattern of braces and can be designed, provided that shear is resisted at every story. Innovative design concepts could use it with an excellent opportunity for choosing bracing system type depend on the efficient and economical bracing system for construction that presents the structural engineer. However, for bracing trusses a suitable depth can be often an essential consideration. Truss stiffness vary as the square of the depth and as a preliminary guide, a height-to-width ratio of 8 to 10 is considered proper for a reasonably efficient bracing system. The possibility for disrupting architectural planning always can be because a space of multi-story depth and optimum height-to-width truss bracing, so compelled the structural engineer to used bracing systems with less efficient in construction (Jason A. Cook 2007).

The CBFs exhibit relatively poor energy absorption and dissipation through inelastic response. Progressive slackening of braces, degradation of compressive strength and premature fracture renders, this is system inefficient and unreliable for seismic applications.

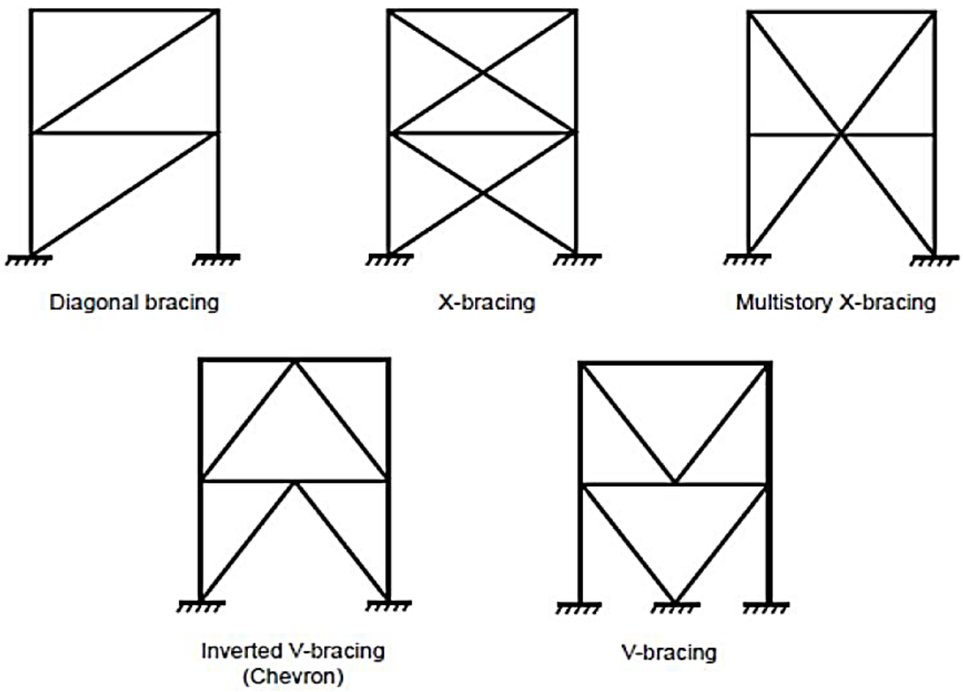


Figure 2.5 Different configurations of CBFs (Sabelli et al., 2013)

Widespread CBF configurations are presented in Figure 2.5. Bracing across single bays considered one of the prevalent type from interior bracing on one-story additions. Typically, can find this in X or K bracing system planner has single diagonals or double diagonals, the K bracing system allows for distributions over the middle of the bay which led to the augmentation popularity, whereas the X bracing system or single diagonals substantially eliminate distributions possibility. Diagonal braced bays, whether single or double, are sometimes paired with discontinuous vierendeel bays allowing for unrestricted distributions. The braced bays can then be hidden within the walls (Jason A. Cook 2007).

2.3 Eccentrically Braced Frames

Eccentrically braced frames are one of the earthquake resistance system that has been spreader in building construction because of the appropriate ductility behavior of eccentrically braced frames (EBF), so that give the researchers chose to make comparison between (SCBF) and this system. The link-beams of (EBFs) are prospective to resist the significant inelastic deformations that effect on the building under the dynamic load of the earthquake motion. The diagonal braces, columns, and beam segments outside of the links should be designed to remain essentially elastic under the maximum forces that can be generated by the fully-yielded and strain-hardened links (AISC 2005).

The EBFs is unquestionably the most popular and intriguing of the new systems, because it combines the stiffness advantages of the braced frame with post elastic performance, comparable in its ability to dissipate energy, to the ductile steel moment frame. An eccentrically braced frame has been defined as a braced frame in which at least one end of each brace frames only into a beam and in such a way that at least one stable ductile link is formed in each beam. For framed lateral force resisting systems as a continuum between the extremes of the moment frame, which depends primarily on bending and shear resistance of the frame elements, to the normal braced frame which depends primarily on the axial strength of diagonal members, the eccentrically braced frame would represent the entire array between the extremes.

Ductility and stiffness, is important characteristics of eccentrically braced frame systems (Engelhardt et al. 1989). Generally these types of braced bents to resist the lateral forces by bending action of beams and columns as shown in Figure 2.6, These provide less lateral stiffness hence less efficient as compared to diagonal bracing systems (Siddiqi,Hameed,&Akmal,2014).

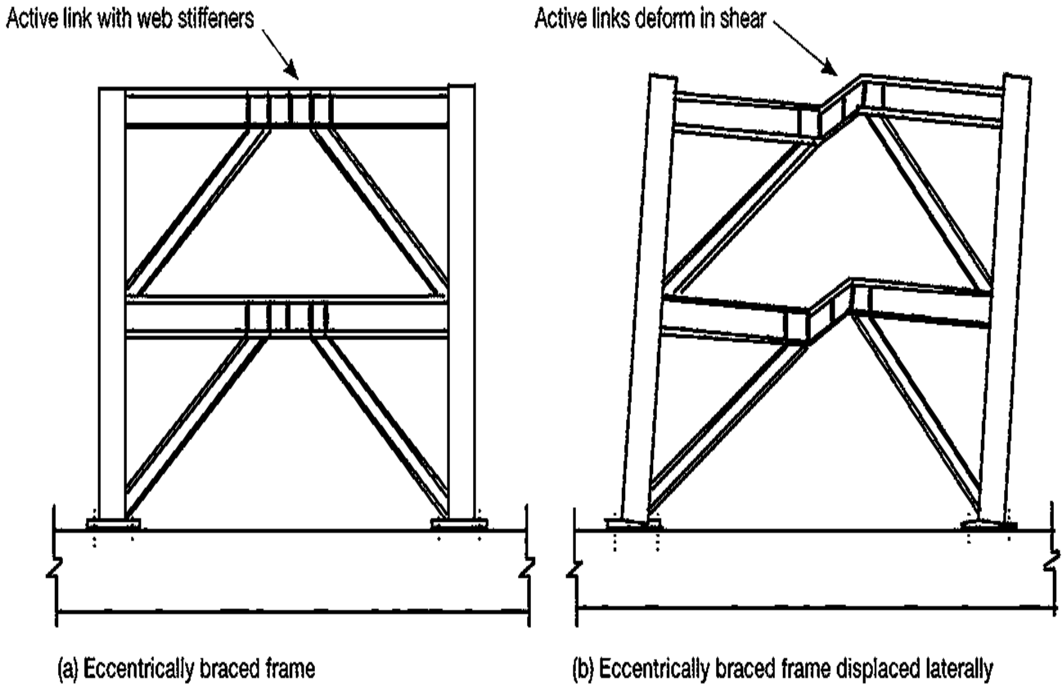


Figure 2.6 EBFs under the effect of lateral force (Akmal, 2014)

For CBFs to be utilized in high seismic regions, special detailing is required to ensure that the frames behave in the prescribed manner. In the 1970s, a new set of frame configurations, shown in (Figure 2.7), was proposed for seismic design that would combine the advantages of MRFs and CBFs while decreasing the disadvantages; the seismic-resisting EBF is the product of decades of research. (Figure 2.7a) depicts a modified chevron configuration in which there is one mid-beam link per level; the braces of the above level could be inverted to form a modified two-story x configuration, which would reduce the axial load transferred to the beams. The frame configuration in (Figure 2.7b) depicts a column-link configuration in which the link is adjacent to one of the frame columns. (Figure 2.7c) depicts a second modified chevron configuration in which two links are created due to brace-column eccentricity; in this case, one link is considered active and one passive. The passive link can introduce uncertainty in the inelastic behavior of the frame as the two links do not necessarily equally share the inelastic deformation, as the nomenclature suggests (Popov and Engelhardt, 1988).

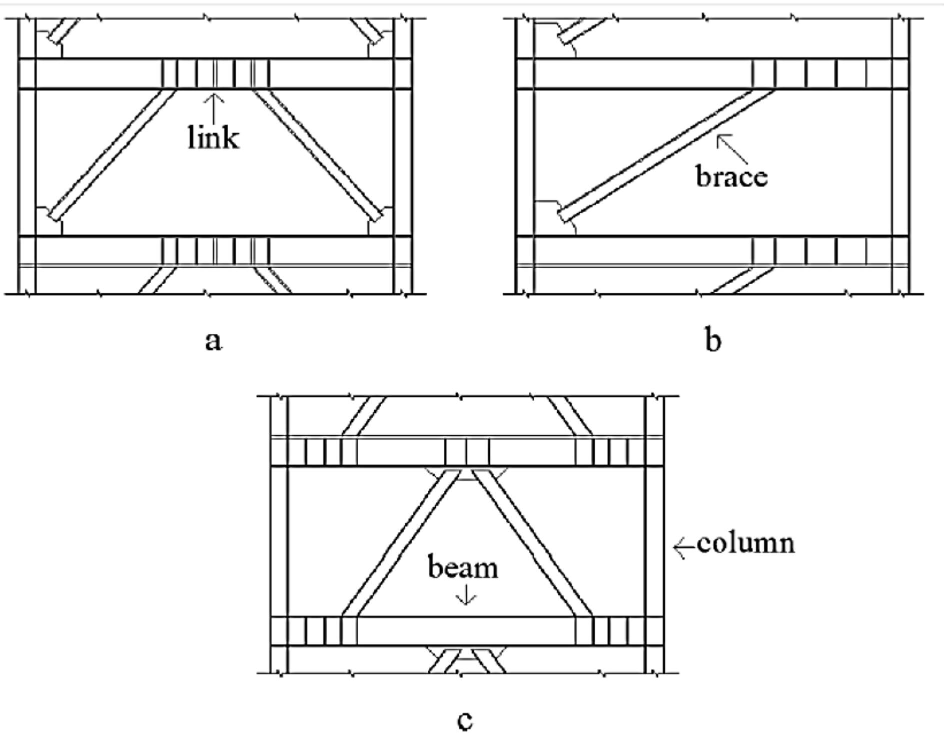


Figure 2.7 Eccentric brace frame configurations (Popov & Engelhardt, 1988).

EBFs successfully combine the high level of ductility of MRFs and the high level of stiffness of CBFs by introducing eccentricity, between a frame cross bracing and column (Popov & Engelhardt, 1988). The cross brace of an EBF provides the elastic stiffness of CBF and the eccentricity of the cross brace creates a link that is responsible for the ductility and, therefore, energy dissipation capacity of MRF. The following sections describe the behavior of the link of an EBF; all other frame components are intended to remain elastic, and as such, adhere to conventional elastic behaviors.

EBFs has been a good configuration for the research from the late 1970s (Roeder & Popov 1977, Roeder & Popov 1978) to present date. Resisting horizontal actions was emphasized by all the previous studies on the EBFs. Consequently the utilizations in civil engineering growing day after day and be more modern and meet the requirement of construction. EBFs in buildings typically include the use of shear (links, which are sections of beams that yield and plastically deform in shear, to provide a stiff and ductile lateral load resisting system).

EBFs have a lower stiffness than the CBF and they show more ductile behavior. In fact, EBFs are a suitable combination of MRF and EBFs. In early 1970, EBFs were used for the first time in Japan due to both properties of moment frames (high ductility) and CBFs (high stiffness)(Chimeh & Homami, 2012).

EBFs can be used in different forms depending on the location of the link beam as shown in Figure 2.8. Each EBFs that composed of four main elements are link beam, non-link beam, bracing and column (Kober & Dima, 2003).

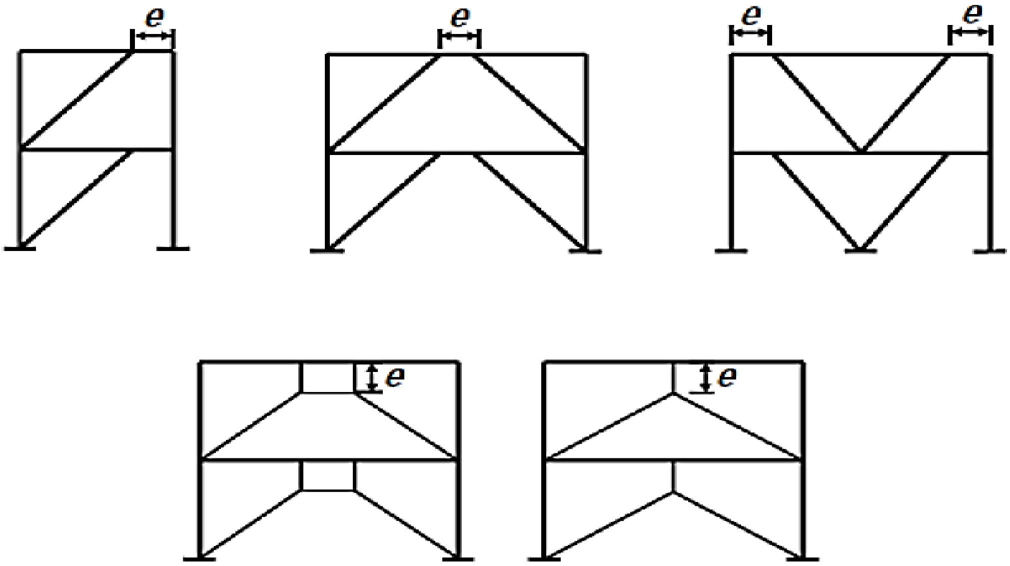


Figure 2.8 Types of eccentrically braced frames depending on the location of the link beam (Kober & Dima, 2003).

The eccentric braced frame is a system to resist the hybrid lateral force. Actually, it can be considered as a superposition of two framing systems that are: the CBFs and the MRFs. This system can collect the essential advantages of all traditional framing systems and reduced particular disadvantages, as well. Generally eccentrically braced frame system has good characteristics are: possession high elastic stiffness, excellent ductility and energy dissipation capacity, stable inelastic response under cyclic lateral loading (Mario D. Aniello 2007)

It can be extracted that the ductility of whole chain can be controlled by the ductility of one of its sector, so the nominal tensile strength of this sector is assumed to be controlled by its ductility. The rest sectors of the chain should be designed to have a higher strength than the maximum strength limit of the lean sectors because this part could be brittle. The eccentrically braced frame system parts divided to lean and brittle. The link beam has to be regarded as a lean sector of the chain, other parts of the system like columns and beams out of the link have to be regarded as brittle parts of the chain (Bruneau et al., 1998).

The main role of absorption and depreciation of inductive energy resulting from an earthquake plays by the link beam. On the other hand, link beams act like fuses and show ductility. By examining the tensile strength of string of chain we can have a clearer understanding of the design essentials of EBFs as shown in Figure 2.9 (Tehranizadeh et al., 2011).

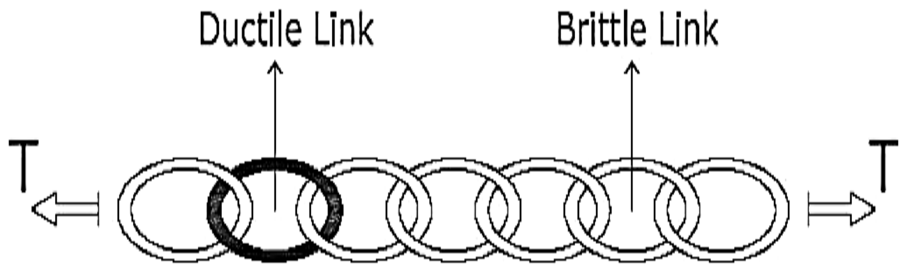


Figure 2.9 Strings of chain represent EBF systems (Tehranizadeh et al., 2011)

Besides K-bracing of CBFs types, there is another type in which door and window openings can be allowed known as eccentric bracing as shown in Figure 2.10. Such type of bracing arrangement causes the bending of the horizontal members of the web of braced bent (Siddiqi et al., 2014). The removable link, that was used in several Christchurch rebuild projects, features a bolt extended end plate to facilitate ease of removal post a major earthquake as shown in Figure 2.11 (Fussell & Cowie, 2014).

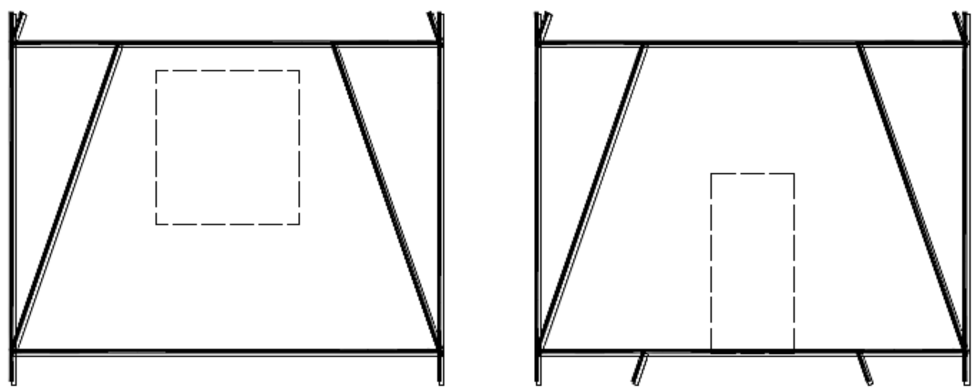


Figure 2.10 Eccentric bracing and openings of door and windows (Siddiqi et al., 2014)

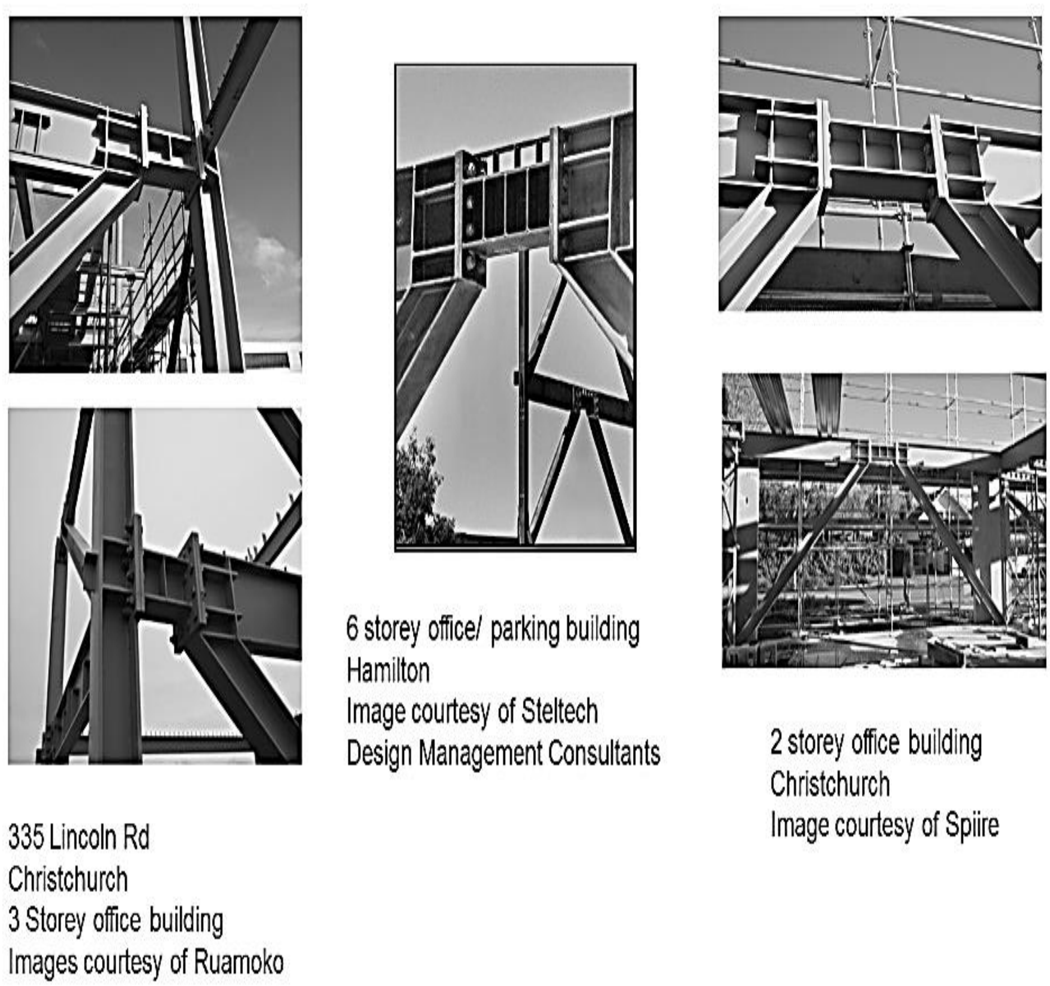


Figure 2.11 EBFs with removable links (Fussell & Cowie, 2014)

The EBFs performs as a hybrid between frame action and braced frame action. The bracing provides excellent stiffness useful in limiting building lateral deformations, while the link beam element is designed as a "fuse" to limit the force in the braces and thus prevent non-ductile type failures such as tension failure of the brace connection or buckling of the brace. The action of the link, particularly when it is designed to yield in shear before it yields in bending, is particularly effective energy dissipation. A feeling for this energy dissipation can be obtained by examining Figure 2.12 which is representative of the type of open, stable hysteresis loops observed in the testing of properly designed shear links (Saunders, 1985).

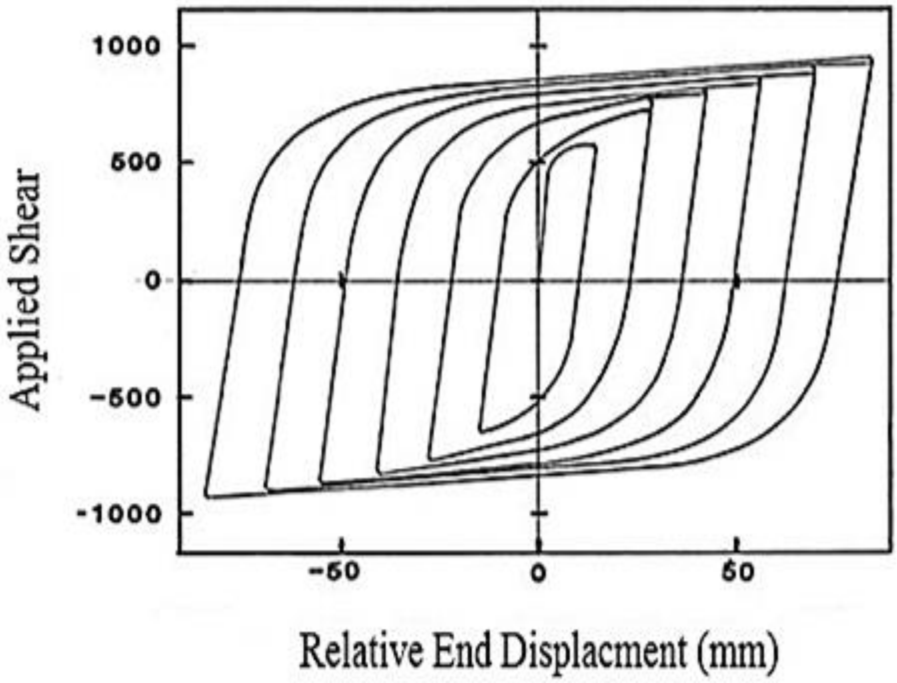


Figure 2.12 Relationship between the applied shear and displacement of link beam (Saunders, 1985).

The design of an eccentrically braced frame system is predicated on the following:

1. The link beam should be capable of large inelastic vertical deformations (on the order of 10% of its length) through a number of cycles without buckling or tearing failure.
2. The strength of the brace and its Connection should exceed by a comfortable margin the yield capacity of the link beam.
3. Columns and other elements of the system should be capable of resisting elastically the forces occurring at the yield of the link beam.

Once the link beam yields, it acts as a fuse to protect the balance of the system from further increases of loading (except for secondary effects such as strain hardening). This system is highly advantageous when there is a need for high ductility and energy absorption coupled with high lateral stiffness.

The full scale sub-assembly EBF frame test set up as shown in Figure 2.13, it was designed to represent the first story of a 5 story EBF frame designed, accordance with the appropriate Canadian codes for a high seismic area (Fussell & Cowie, 2014).

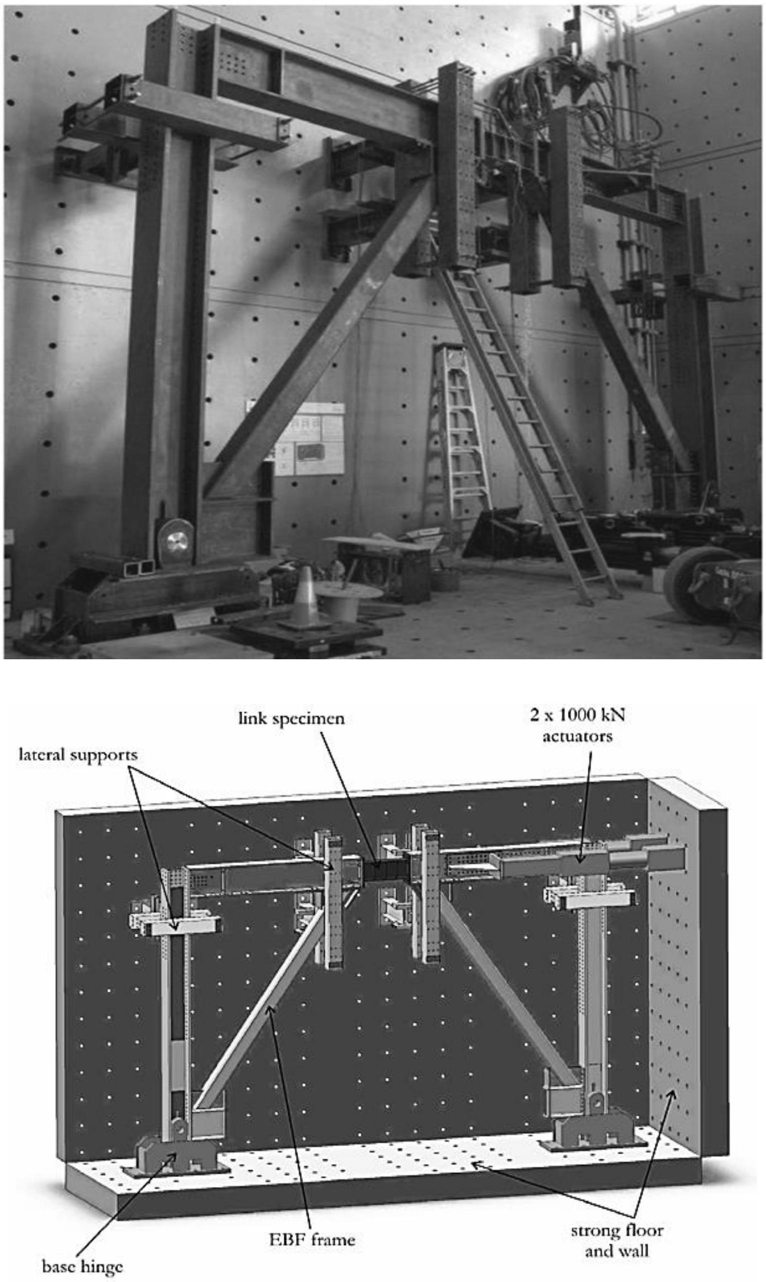


Figure 2.13 EBF frame and test setup (Fussell & Cowie, 2014).

Within an EBF, the link element is designed to undergo severe inelastic deformation. During an extreme seismic event, the link may experience strain on a magnitude that induce strain hardening. Figure 2.14 illustrates an idealized stress-strain curve for structural steel; for strain hardening to occur, the structure must pass through two stages of behavior. During low loading, a structure should remain in the region “a,” the elastic range; in the elastic range an increase in stress results in a linear increase in strain related to the modulus of elasticity, E of the structural material. During moderate loading, a structure may enter region “b,” with the transition between “a” and “b” characterized by inelastic (non-linear) behavior. In region “b,” the strain increases at a constant stress level, or behaves plastically. During plastic deformation, permanent residual deformations occur though the deformations may not be detrimental to the structural capacity upon unloading. After the structure's plastic capacity is reached, additional inelastic behavior occurs as strain hardening; during strain hardening, the structure can undergo further deformation with a non-linear increase in stress. After the maximum tensile load is reached, necking occurs in members as strain continues to increase. During necking, the cross-sectional area of the seismic fuse in the LFRS decreases, reducing the stress; as strain continues to increase the member ruptures, indicated by point “d” as shown in (Figure2.14) (Arce et al., 2001).

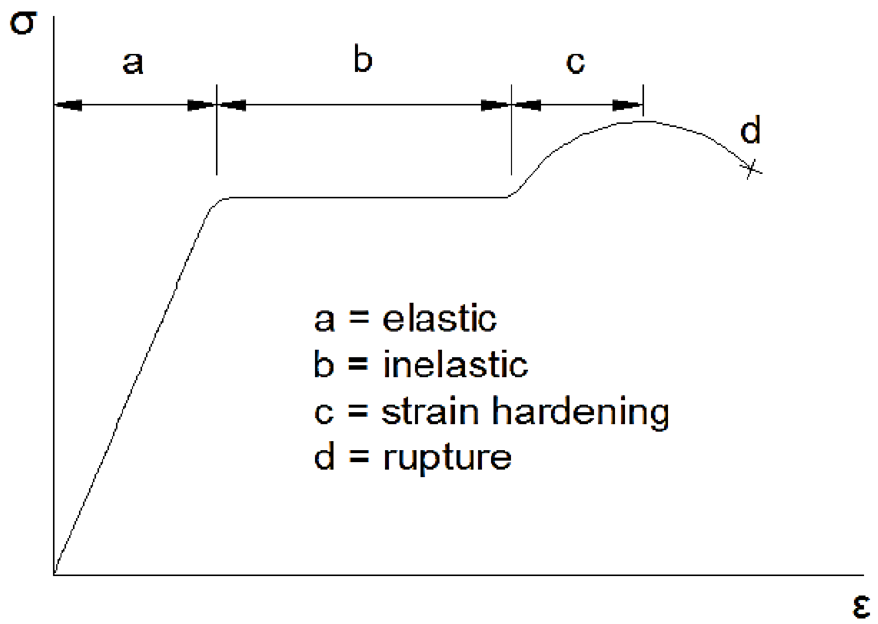


Figure 2.14 Idealized structural steel stress-strain curve (Arce et al., 2001).

Studying strain hardening in the link element requires the reduction of the shear link length ratio limit from 2. Furthermore, as a link element experiences large rotation angles, large end moments and steep, strain gradients develop causing large flange strain. Large flange strain leads to instability in the form of web buckling after yielding; for unstiffened webs, web buckling occurs very shortly after shear yielding. Web buckling of shear links causes a severe reduction in load-carrying capacity, reducing energy dissipation and ductility. And equally spaced web stiffeners preclude web buckling, which allows end moments to increase beyond resulting in larger flange strain. To prevent flange weld failure, the maximum permissible moment for desirable shear link behavior is, the corresponding shear for the bounded moment is approximately. And from statics the maximum for shear links (Kasai and Popov, 1986).

CHAPTER THREE

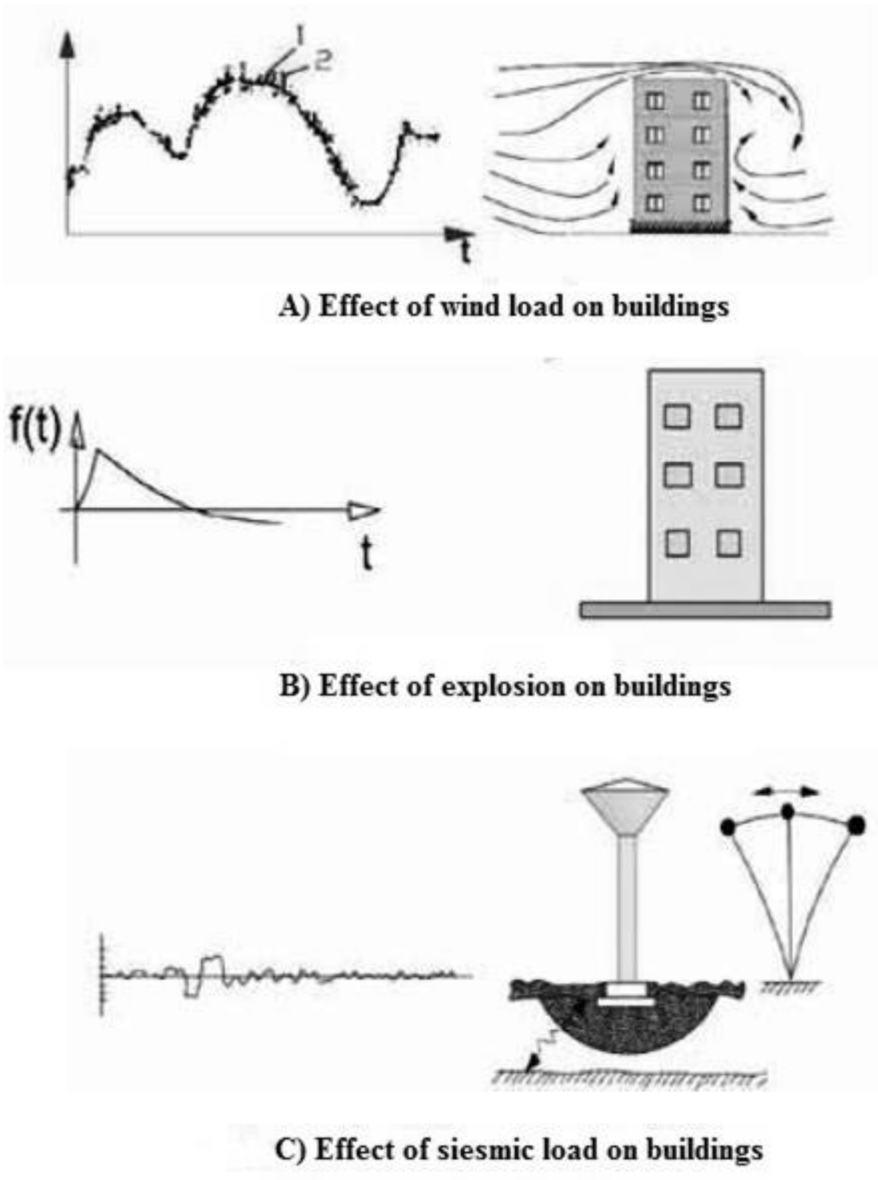
METHODOLOGY

3.1 Introduction

The main function for all types of structural systems which are used in the erection of buildings is to transfer loads in a safe and effective way, there are different types of loads resulting from the effect of gravity and considered from the most common loads like dead load, live load and any other vertical loads. Moreover, lateral loads are also an effect on buildings, lateral loads may evolution high stresses due to vibrations caused by sway movements or vibrations that caused by winds, explosions or earthquakes as shown in Figure 3.1. Therefore, the structures can resist lateral forces by owning a sufficient strength against vertical loads with adequate stiffness. In the life span of structural system like building should have sufficient capacity of carrying all gravity loads and transfer to foundations.

For resisting the lateral forces, it can be dealt with these forces in the same method that use for gravity loads. Traditionally, the columns designed to be able to carry the weight of building and bear small sideways movement from the seismic or wind effect. But when a major earthquake happens because unexpected earth motion that caused significant damage to buildings, this damage to structures in turn causes casualties do not compensate in life with a big number of injuries. Therefore, structures are designed to resist moderate earthquake intensity which might occur during the life of the structure, where it should have sufficient stiffness and strength to resist the impact of earthquakes, also control deflection and to prevent any possible damage could happen.

The design of structures is normally based on guideline methods of building codes. In the static case usually the loads that effect on structures are low and keep it in the elastic range. So, under a powerful seismic load maybe the structure under the effect of forces beyond the elastic range that may cause large damages in the structural elements. Thus, the dynamic load one of an important characteristic for a structural design.



A) Effect of wind load on buildings

B) Effect of explosion on buildings

C) Effect of siesmic load on buildings

Figure 3.1 The effect of dynamic loads on buildings.

The internal forces are grouped in elements, offsets, story force, story shear force and reactions from each method with statistical information, the 3-D type must be taken into account mutual influences of the phases when installing maximum values of these phases. The values of components can reduce flexible response when the design provided that under no circumstances is not less than the value of the design base shear force {resulting from this reduction}, on the base shear strength resulting from the response flexible shear divided by the modulus (*R*).

3.2 Analysis Using Computer Programs

The computer program can do the analysis, but the result still depends on the input data used and formed by engineer in addition to the model created. Also analysis procedure and type with output arrangement is important to get the correct results. Because of complexity of analysis calculation, especially for large buildings or 3D (three dimensions) structures that its calculation need time and sometimes it may be impossible doing it without any errors. If hand calculation method is used, it needs more iteration to get nearly exact or reasonable value. All software uses nearly same methods that are used for hand calculations, but concentrate on an iteration method or geometric stiffness that is nearly exact. The differences between hand calculations and software performing analysis are:-

1. Notes on the hand computation methods:-
   - Applicable for small problem or small structures
   - Difficult for even medium sized problem.
   - Three dimensional analysis almost impossible.
   - The probability of errors rises with the size of the structure.
   - Time needing is high for performing analysis.

2. Notes on using the analysis by computer
   - Matrix methods of structural analysis.
   - Try to develop numerical techniques.
   - Using the finite element method.
   - Programming languages developed.
   - Arranging input and output is easy with drawings.

3.3 Dynamic Analysis of SAP 2000

Dynamic analysis relies on the dynamic effects of the loads applied to the structure which change with the times in terms of value or the application periods, This is done through calculating the free vibrations, frequencies and analysis the spectra of response, In a manner specific elements, from analysis types which carried out by:

1. Modal analysis. This way takes into account the number of vibration models, distinguishes the model which have longest natural vibration period (smaller

frequencies), are calculated the internal force and stress for each element of the structure and each type of vibration model.

2. Response spectrum (RS) analysis. Through get a spectral response curve toward a certain direction and deemed it as a graphic relationship between the structure and the acceleration response.

3. Time history analysis (analysis of variable loads with time) linear and nonlinear.

3.3.1 Modal analysis

This type of analysis treats with a number of vibration model and choose from which owns longest natural vibration period (smaller frequencies). In this method can analysis single and multi-story structure, where the program calculates both vibration, vibration model, accelerations and mass ratios for each model, then find the forces and moments for each element of the structure.

3.3.2 Response spectrum analysis

This approach permits the multiple model of the response of a building to be taken into account when it be in the frequency domain. This is considered from the requirements of building codes for structures except in some cases, such as very simple or very complex structures. The response of a structure may be defined as a combination of a lot special shapes (models) which vibrating string correspond to the "harmonics", obtain these models for structures by using computer analysis. For each model, the response is read from design of spectrum depending on the modal frequency and the modal mass and then combines them to provide an estimate the total response of the structure.

It is worth noting that the results of the RS analysis that using in the response spectrum, obtained from a ground motion are usually different from those which will be calculated directly from a linear dynamic analysis by using that ground motion directly, especially after lost phase information in the process of generating the response spectrum. In some cases where the RS approach is no longer appropriate, such as when the structures be either too irregular, too tall or of significance to a community in disaster response, it be needed to more complex analysis, like nonlinear static or dynamic analysis (Al-hamdany, 2010).

In this method to find spectrums response curves as a function of time related to a specific direction, and can be represented by a graphic relationship between the natural vibration period of the structure and the responses then analyze these curves. There are some attempts to develop the RS method by using the following steps:

1. Obtain the model shapes, frequencies and modal participation factors by using a modal analysis of the structures
2. It is possible to provide maximum response as same which was obtained in each mode of vibration, by deriving an equivalent static load of the acceleration response spectrum
3. Find the total maximum response of the structure by combining maximum modal responses

There is a unique concept for the earthquake that is the hypothesis equivalent of lateral forces, because it is converts the dynamic analysis partly to dynamic and static analysis to obtain the maximum stresses, this maximum stress is considered interesting and useful. The set of lateral force that will produce the same peak response as which found in the dynamic analysis of structure, these include a single mode of vibration for equivalence.

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

$$\mathbf{M}\mathbf{u}''(t) + \mathbf{C}\,\mathbf{u}'(t) + \mathbf{K}\,\mathbf{u}(t) = m_x\,\dot{\mathbf{u}}''_{gx}(t) + m_y\,\mathbf{u}''_{gy}(t) + m_z\,\mathbf{u}''_{gz}(t) \quad (3.1)$$

Where is:

- (m_x, m_y, m_x), 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.
- **u**(t), **u**'(t) and **u**"(t), are the time-dependent vectors for the absolute node displacements, velocities and accelerations, respectively.

After finding the maximum responses and acceleration, in this way the analysis gives the internal forces, moments and stress results. Therefore, the program can be accomplished any number of situations of spectral analysis in one analysis process and can give a special name for each analysis case, and the analysis of each case is different from the other in the spectrum acceleration that used and also in the way of composition results.

RS value for specific region are derived from time records of potential ground accelerations that can happen in this region, provided that include this records periods old and new not just the result of record one period that shown in Figure 3.2 below. RS design depends on the geological characteristics, tectonic, seismic, and soil characteristics for region of study.

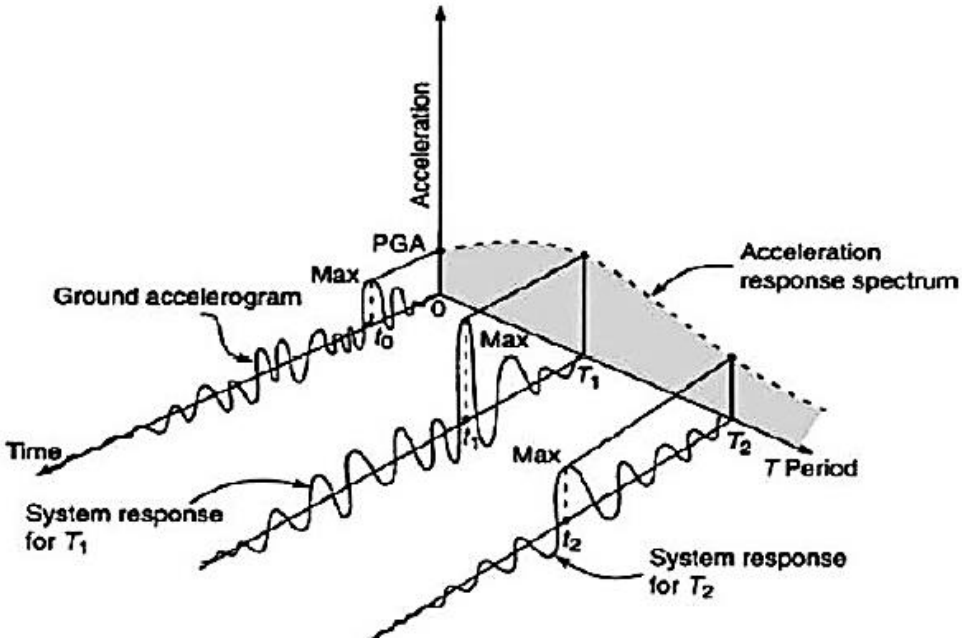


Figure 3.2 Description schematic of response spectrum

This method is favoured for the earthquake engineering community because of:

1. It is progress a method of procedure the equivalent static lateral load analysis
2. It permits to get a clear and easy understanding of the roles of different modes of vibration
3. It gives a simple method to find the design forces that effecting on the structural members under seismic load.
4. It beneficial roughly for evaluation of seismic reliability of structures.

3.3.2.1 **Response spectrum curve**

This curve represents graphically function connect the vibration period of structures and pseudo spectrum response acceleration, has been mentioned in this paragraph if it is not selected any constant, the program will depend single constant acceleration represents value for all periods. As can use a scale factor to convert acceleration values, resultant by earth gravity, the modules are compatible with the matter as in the Figure 3.3, which shows how the vibration damping of the structure.

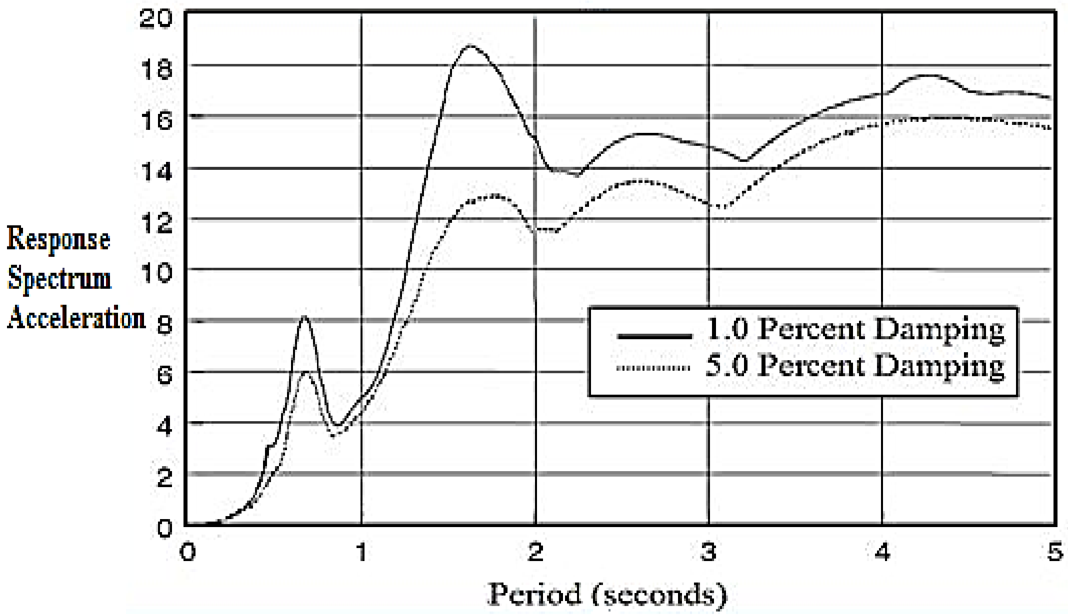


Figure 3.3 Illustrates how vibration damping of the structure (Wilson, 2002).

If the domain for RS is undefined to cover all modes of vibration, the program will extend the painted curved from the both sides by using constant acceleration has a value of acceleration at the nearest known point on the edge of the domain.

3.3.2.2 **Modal combination**

Can be summarized the analysis steps that used in the program in some points are as follows:

1. Determine vibration phases and neutral vibration period for studying structure.
2. Determine the maximum accelerations for a neutral vibration period of the design spectrum for every natural pattern.

3. Identify the mass which are effective or contribute for each natural pattern, and through the forces or maximum inertia loads.
4. Determine the maximum values of the responses (transfer, speeds and accelerations) for calculating inertia forces of every natural vibration pattern.

3.3.2.3 **Complete quadratic combination method**

Complete quadratic combination method (CQC) gives compositions maximum values for the RS (S) according to the following relationship.

$$S_1 = \sqrt{\sum_{J=1}^{N} S_{1.J}^2 + 2\sum_{j=1}^{n} \left| S_{1,i} \quad , \quad S_{1.j} \right|} \qquad i \neq j \tag{3.2}$$

It should be noted here that in the case where the seismic force is parallel to a global axis (like x axis, for example) should note when forming the equations of forces or inertia loads ($P_{i,\,j}$) that the spectral coordinates ($S_{a,i}$), take their values as follows:

1. Vertical coordinates for the spectrum design ($S_{ax,\,j}$) and the accompanying natural vibration period ($T = T_1$) for all pattern compounds (J), it has an alternative values on the axis (X).
2. All other transitions for modal (J) takes a value equal to zero ($S_{ay,\,j} = 0$), as well a deformations torsion or compounds for torsion ($S_{aw,\,j} = 0$) on the axis (Y).

Considered the CQC as a default method for analysis in the (sap2000) program, which takes into account the interdependence modals resulting from damping spectrum that be near each other. When the damping is non-existent devolve result of the calculation in this way to the other method of modal analysis which is (SRSS).

3.3.2.4 **General modal combination (GMC) method**

This statistical method differs from the first method (CQC) in that it takes into account the correlation between models using a solid response. The damping is determined in one of two methods mentioned in the case of (CQC), provided identify two frequencies (f_1, f_2) relating to seismic loads and not for the structure, their mission is defining the content of solid response to the earthquake. With the knowledge that:

$$f_1 = \frac{S_{A\,max}}{2\pi\, S_{V\,max}} \tag{3.3}$$

$$f_2 = \frac{f_1+2f_r}{3} \tag{3.4}$$

Where is:

- ($S_{A\,max}$) Maximum spectral acceleration of the earthquake.
- ($S_{V\,max}$) Maximum speed spectral of the earthquake.
- (f_r) Solid frequency for input seismic movement data. Is the frequency which be spectral acceleration fixed confront infinite frequency, and is considered by some that (f_2 =f_r).

The default values in the program (f_1=1) and (f_2=0), and the latest value means that the frequency is infinite, presumably on the way (GMC) the lack of a solid response to the least frequencies (f_1), and bigger than the (f_2). Whereas (f_2 >f_1>0).

3.3.2.5 **Square root of the sum of their squares (SRSS) method**

This option works to collect the results of loading cases selected in a manner the square root of the sum of squares, and is used in the case of dynamic analysis as a case collecting response spectra.

$$S_1 = \sqrt{S_{1,1}^2 + S_{1,2}^2 + S_{1,3}^2 + \cdots + S_{1,j}^2} \tag{3.5}$$

Thus, in order for bending moments become the previous relationship as follows:

$$M_s = \sqrt{M_{s.1}^2 + M_{s.2}^2 + M_{s.3}^2 + \cdots + M_{s.j}^2} \tag{3.6}$$

This means that the idea of composition based on that not all the model arrives to the maximum values at the same time. Therefore, it can consider the typical resonances, including transitions and torsion, independent from each other. Thus, the results produce a maximum value most probable from the square root of the sum of the squares (SRSS) and based on probability theory.

For example, the concept of the independence of the models is by the European code EC8 (CEN, 94) true in the case of incidence the following relationship:

$$T_i = 0.9\, T_j \qquad (j < i) \qquad (3.7)$$

Where (T_i , T_j), is the natural vibration period for any consecutive two models of vibration of the presumed model in seismic design. If it did not materialize the previous equation should use more precise other ways.

3.3.2.6 **Combine the absolute value method (Absolute)**

This method relies on the aggregate of the results of the model absolute values of responses. This method considered is unpalatable to a large extent.

3.3.2.7 **Limitations of the response spectrum method**

1- Story drifts calculation. All displacement values obtained from the RS method are positive. Thus, a scheme of a dynamic displaced shape has relative poor lack of information because each displacement depends on estimating the maximum value. Internal story displacements are utilized to find out the damage to non-structural elements. But, it cannot be calculated directly from the potential peak values of displacement.

2- Computation of spectra stresses in beams. Can be calculated the stresses inside the cross section of a beam by using the following equation main:

$$\sigma = \frac{P}{A} + \frac{M_y\, x}{I_y} + \frac{M_x\, y}{I_x} \qquad (3.8)$$

Use the previous equation to calculate a specific x and y point in the cross section and for finding maximum spectral axial force and moments for all positive values. It is clear that the resultant stresses may be useless because all forces will perhaps do not give their peak values at the same time.

3- Check procedure for design steel and concrete beams. Regrettably, the design strength ratios cannot be obtained in each mode, because most design check equations for steel structure is a nonlinear function of the axial force in the members.

4- Check of shear force in bolts. Considered interesting problem for obtaining the maximum shear force in a bolt, is not correct to calculate the maximum shear force from a vector collecting because the x and y shears do not found their peak values at the same time (Wilson, 2002).

3.3.3 Time history analysis

Perform this analysis for application of the tremors typical on the structure similar to the real earthquake, where they are activating the forces of inertia or load generated by multiplying the blocks with accelerations according to Newton's second law. This provides that the acceleration of an object arising from a net force directly proportional to the amount of that force in the same direction, and inversely commensurate with the mass of the object. However, the second law gives us an exact relationship between force (F), mass (M), and acceleration (A). It can be expressed as a mathematical equation:

$$F = M \times A \tag{3.9}$$

Being seismic movement representation according to this method in terms of the time history of ground accelerations quantities related (transition and speed), and remains the standard acceptance relates mainly, the extent of the proposed spectrum agrees with the spectrum flexible response. The time history analysis can be divided to three patterns are:

1. Linear transient analysis. The analyses of initial conditions are equal to the zero or the terms of the end of the previous cycle analysis. Considered is the behavior of all of the items linear through the new analysis cycle.
2. Periodic analysis model. Analysis of this type begins from initial conditions, are adjusted in accordance with the end of the previous analysis cycle, the behavior of all the elements also linear through the new analysis cycle.
3. Nonlinear transient analysis model. It is similar to the first analysis with the assumption that the behavior of all nllink elements nonlinear and the behavior of the other elements are linear. Used nllink element for modeling linear behavior of a structure like gaps, dampers and the Insulators.

In nonlinear static and nonlinear time-history analysis, the post-yield behavior by assigning concentrated plastic hinges to the frame and tendon objects was simulated. Elastic behavior occurred over member length and then deformation beyond the elastic limit occurred entirely within hinges, which were modeled in discrete locations. Inelastic behavior was obtained through the integration of the plastic strain and plastic curvature which occurred within a specified hinge length, typically on the order of member depth (FEMA 356, 2000).

This type of analysis treat with linear and nonlinear systems with dynamic analysis is significantly effective, as be this analysis incorporates dealing with real-time for earthquakes as a result of ground motion and shows the true description for the potential mechanism deformation and collapse in a structure. However, it is dull and complex analysis for the presence of a lot of complex mathematical equations. In spite of that, non-linear dynamic analysis is considered more accurate than the ready analysis methods, there are considerable contrasts in demands determined by utilizing current techniques for scaling ground motions and its design was exhausting. Surrogate analytical techniques include simplification of the hypotheses (like the neglect of higher modes, hypotheses of linearity and the hypotheses of independence of each potentially inelastic "modal" response) which lead to design quantities that can bear slightly related to the demands prospect in the design event. Notes have been considered from the advantages of the time history method:

1. Analysis of a structure, applying data over increment time steps as a function of acceleration, force, moment and displacement.
2. The closer the spacing of time steps, the more accurate the solution will be.
3. Eigenvalues generated for the structure based on response to time history.
4. Considered to be more realistic compared to RS analysis.
5. Most useful for very long or very tall structures (flexible structures).

Time history analysis has some point can consider as a drawback like long time consuming, generates large quantities of data and may not always reduce seismic forces in a structure. Depend on soil properties, structure type and available data.

3.4 Pushover Analysis Method

Nonlinear static pushover analysis procedure has been introduced to the Civil Engineering society and used simultaneously with the advent of designing on the basis of performance. A simple explanation of the Pushover analysis is: performing a static, nonlinear process in which the amount of the structural loading is boosted incrementally in accordance with a specific predefined pattern. Feeble links and failure modes of the structure can be figured out, while the amount of loading increases. The loading is continuous with and compatible with the outcome of the cyclic performance and behavior and also load reversals that are being figured out with the help of implementing altered continuous force-deformation provisions and with damping approximations (Nourbakhsh, 2011)**.**

Pushover analysis is a static and nonlinear procedure that Increase from magnitude of the structural loading gradually, and this increment is in accordance with a pre-specified pattern. Static pushover analysis is an endeavour to assess the real strength of the structure conducted by the structural engineering, and aspire it be a helpful and efficient tool (Ahmed, 2012).

Pushover analysis operates in an easy and effective way and is considered as an alternative to dynamic time-history analysis. In spite of minimalism, it is able to provide important information on the structural responses. Actually, pushover could be used to identify critical areas, which is expected to be the high inelastic deformations in those areas, and strength Irregularities which may cause significant changes in the inelastic dynamic response characteristics. Furthermore, it could also progress realistic assessments on the force required in elements which are likely to be brittle, such as shear-dominated members. Eventually, the concatenation of yielding and/or failure of structural components and the progress of the overall capacity curve of the structure pushover analysis has ability to predictability it, and hence verification from sufficiency of the seismic load path, which represents both of structural and non-structural elements of the system (Antoniou & Pinho, 2004).

The static pushover analysis method has no strict theoretical base. It is mainly based on the assumption that the response of the structure is controlled by the first mode of vibration and mode shape, or by the first few models of vibration, this shape remains constant throughout the elastic and inelastic response of the structure. This provides the basis for transforming a dynamic problem to a static problem which is theoretically flawed. Furthermore, 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 illustrated in Figure 3.4.

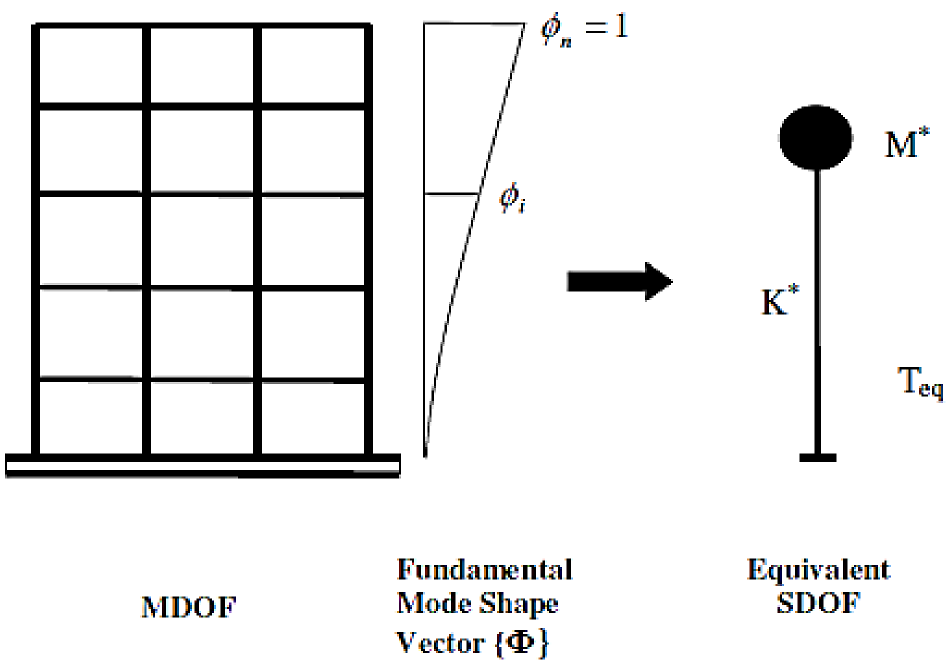


Figure 3.4 Conceptual diagrams for transformation of MDOF to SDOF system (Themellis, 2008)

The earthquake induced motion of an elastic or inelastic MDOF system can be derived from its governing differential equation:

$$[\mathbf{M}][\ddot{U}] + [\mathbf{C}][\dot{U}] + [F] = -[\mathbf{M}][\mathbf{1}]\ddot{u}_g \qquad (3.10)$$

Where is:

- **M** is the mass matrix.
- **C** is the damping matrix.
- **f** is the story force vector.
- The (1) is an influence vector characterizing the displacements of the masses when a unit ground displacement is statically applied, and (*g*) is the ground acceleration history (Themellis, 2008).

The foresee from this analysis to assess the parameters for critical response that effecting on structural system and elements of structural system as much as possible for those in prospect by nonlinear dynamic analysis. Pushover analysis provides information on a lot of response properties which cannot be available from an elastic static or elastic dynamic analysis. These characteristics are (OĞUZ, 2005):

1. Estimates of inter story drifts and its distribution along the height
2. Determine the force demands of brittle members, like an axial force demands on columns and moment demands of beam-column connections
3. Obtain the deformation values for ductile members
4. Determine the location of weak points in the structure
5. The consequences of the strength impairment of individual members on the behavior of structural system
6. Determination of strength discontinuities in plan or elevation which will lead to changes for dynamic properties in the inelastic range
7. Check the completion and sufficiency of load path pushover analysis also display weaknesses in the design which can remain hidden

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Computational Methods

Three different structural systems of steel building modelling, analysis and design are done using SAP2000 v14 software. Therefore, three dimensional analysis was conducted. The column foundations were considered as fixed in all cases. These frames were designed on each direction, the structures were considered symmetric for a circular building plan so the effect will be same from all directions, non-symmetric for a rectangular plan so the effect will be different for each direction.

The structures at 10, 20 and 30 story building, the dimensions of the internal beam and external beam are (IPE360). They were varied Internal Column and External Column is (IPE400). All the supports of structures are fixed. In the retrofitting of the steel buildings, the structural steel braces in the configuration of concentrically (x) bracing type of circular and rectangular plan, the configuration of eccentrically bracing with different link length for rectangular building and one link length for circular building. For the brace elements, pipe section was utilized. At all story buildings, the dimension of the bracing pipe was (TUBO-D193.7X4.5) depend on EURO CODE one of default code in the program. The design live load of the building was taken as 2.00 KN/m^2 because the frames represent typical office buildings that were constructed in accordance to minimum design loads for buildings and other Structures.

An eccentrically braced frame is a structural framing system in which the axial forces in the bracing members are transferred either to other braces or two columns through sea and bending in short beam segments called active links. The active link acts as a fuse, dissipating large amounts of input energy upon lateral overloading of the structure and its behavior dominates the behavior of the structure. While experiments have demonstrated the good global behavior of the framing system, the link length that used in my study equal to (L/5) from the original beam length that equal to (L) for circular building, link length equal to (L/3), (L/5) and (L/10) form the beam length (L) were used for rectangular building.

For seismic load that submit in this study all the variables, design equation and parameter depend on the Turkish Code For Building That Built In Seismic Zone (TEC 2007), Static and dynamic nonlinear analyses were carried out using the finite element program in SAP 2000 non-linear version 14 (CSI, 2011) to determine the seismic performance of the original steel frames and those with concentric and eccentrically braced frames. Response spectra are very useful tools of earthquake engineering for analysing the performance of structures and equipment in earthquakes, since many behave principally as simple oscillators. Thus, if you can find out the natural frequency of the structure, then the peak response of the building can be estimated by reading the value of the ground response spectrum for the appropriate frequency. In most building codes in seismic regions, this value forms the basis for calculating the forces that a structure must be designed to resist (seismic analysis).

4.1.1 Dynamic methods

Dynamic analysis probably can be conducted by time-history, response analysis or either the mode superposition or step by-step methods of analysis. However, standard time history ground motions, for the purpose of design, were not defined. Therefore, most engineers use the response spectrum method of analysis as the basic approach. The procedure in a response spectrum analysis starts with the calculation of the three dimensional mode shapes and frequency.

This method permits the multiple modes of response of a building to be taken into account (in the frequency range). This be required in a lot of building codes for all structures except for very simple or very complex structures. A combination of many special shapes (modes) that in a vibrating string correspond to the "harmonics" considered as the definition of response of a structure. Computer analysis can be used to determine these modes for a structure, the response is read from the design spectrum for each mode, based on the modal frequency and the modal mass.

The result of a response spectrum analysis can obtained from a ground motion is typically different, from that should be calculated directly from a linear dynamic analysis using that ground motion directly, since phase information is lost in the process of generating the response spectrum. The structures must be treated as three dimensional systems model should be developed in the very early stages of the design since it can be used for static wind, vertical loads as well as dynamic seismic loads. Only structural elements with significant stiffness and ductility should be modelled, non-structural brittle components can be neglected. However, shearing and axial deformations can be considered in all members without a significant increase in computational effort by using finite element models, through using the computer software (Sap 2000) (Al-hamdany, 2010)

The considerations of earthquake design in the TEC 2007 correspond to high intensity earthquake. For buildings the probability of exceeding the design of earthquake within a period of 50 years is 10 %. Earthquakes with different probabilities of exceedance are considered in assessment and retrofit of existing buildings. Seismic zones cited in this TEC2007 divided into four zones as shown in Figure 4.1 which are the first, second, third and fourth, depending on the Seismic Zoning Map of Turkey (TEC, 2007).

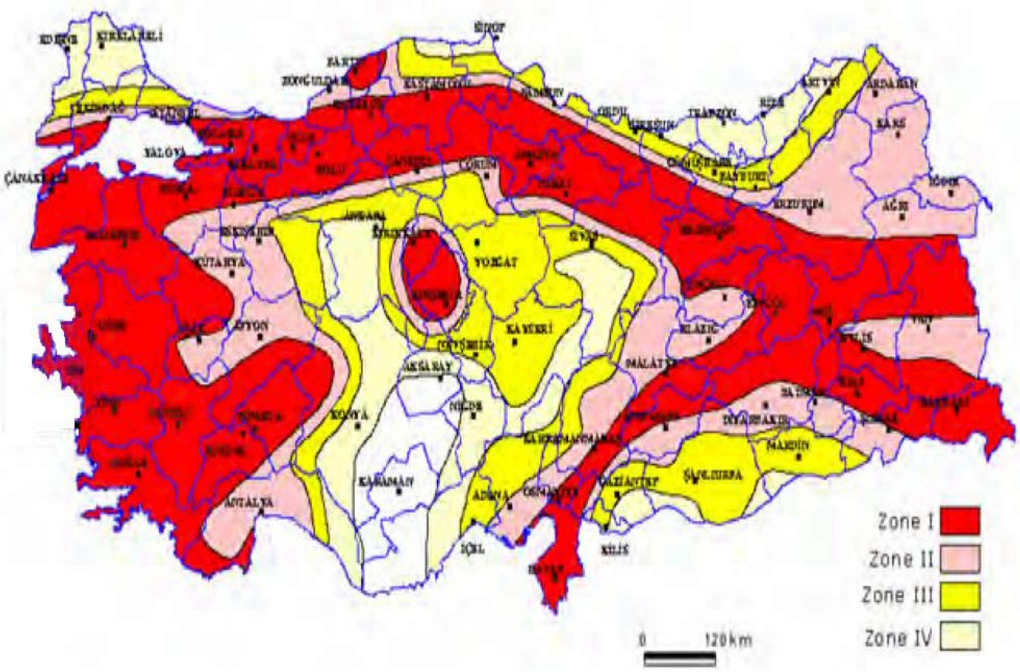


Figure 4.1 Seismic zone maps of Turkey

Seismic analysis requirements that be applied to earthquake resistant design of steel structures that will be built in seismic zones. The structural system should resist the seismic loads as a whole as well as each structural element of the system shall be provided with sufficient stiffness, stability and strength to a safe transfer of lateral seismic loads between the elements of the structural system down to the foundation soil. There is some consideration in TEC 2007, should be taken into account which are:

1. The seismic loads that act on buildings must be based according to spectral acceleration coefficient and seismic load reduction factors
2. Seismic loads shall be assumed to act asynchronously along the two perpendicular axes of the building in the horizontal plane
3. Load factors that be used to determine design internal forces under the combined effects of seismic loads and other loads according to the ultimate strength theory must be taken from the relevant structural specifications
4. Assumed that the wind loads and seismic loads do not act simultaneously, and the most favourable response quantity due to wind or earthquake must be considered for the design of each structural element.

For design response spectrum should be presented each seismic characteristics that occur in a site. Spectrum response design depends on the geological characteristics, tectonic, seismic and soil on the study site. Internal forces and deformations occurring in the elements are calculated by examining the obtained model of the structure with the earthquake loads affecting the building and under the common effects of the other load. Since linear analyses, which form the basis for the assessment of force based methodologies are generally done being in compliance with the provisions of the earthquake regulation, maximum effects occurring in the cross sections must be calculated by using the load combinations specified in the regulation. For above mentions to determine the earthquake effects on the structure, spectral acceleration coefficient is used for that purpose.

The Spectral Acceleration Coefficient $A(T)$ use for determining seismic loads is given by Eq.(4.1) and (4.2). The elastic spectral acceleration, $S_{ae}(T)$, which is defined as the ordinate of 5% damped, elastic design acceleration spectrum; is equal to spectral acceleration coefficient times the acceleration of gravity.

$$A(T) = A_0 * I * S(T) \tag{4.1}$$

$$S_{ae}(T) = A(T) * g \tag{4.2}$$

The effective ground acceleration coefficient (A_o), introduced in Eq. (4.1) depends on the seismic zone (as described previously) is specified in Table 4.1.

Table 4.1 Effective ground acceleration coefficient (A_o) (TEC, 2007)

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

The seismic zone number 2 was chosen for our study to find the effective ground acceleration coefficient. The building importance factor (*I*) given in Eq. (4.1) is specified in Table 4.2.

Table 4.2 Building importance factor (TEC, 2007)

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
1. Buildings to be utilised after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilised 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

Our model is office buildings. Therefore, it represented in the fourth type of the building importance factor table. So, the important factor in our case study will equal to 1. For the spectrum coefficient *S(T)*, given in Eq. (4.1) will determined by Eq. (4.3), depending on local site conditions and the building natural period (T) (Figure 4.2).

$$S(T) = 1 + 1.5\,\frac{T}{T_A} \qquad (0 \le T \le T_A)$$

$$S(T) = 2.5 \qquad (T_A \le T \le T_B) \tag{4.3}$$

$$S(T) = 2.5\left[\frac{T_B}{T}\right]^{\mathbf{0.8}} \qquad (T_B \le T)$$

Spectrum characteristic periods, T_A and T_B, shown in Eq. (2.2) are specified in the table 4.3, depending on Local Site Classes**.** Our local site class is Z3 which $T_A = 0.15$ and $T_B = 0.6$ second.

Table 4.3 Spectrum characteristic periods (T_A, T_B) (TEC, 2007)

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

Soil groups and local site classes to be considered as the bases of determination of local soil conditions, values of soil parameters showed in Table 4.4 in TEC 2007, to be considered as standard, values given for guidance only in determining the soil groups. The investigations of soil based on appropriate site and laboratory tests are mandatory to be conducted for below given buildings with related reports prepared and attached to design documents. Soil groups and local site classes defined in Table 4.4 and Table 4.5 (TEC, 2007).

Elastic acceleration spectrum can be found through special investigations by considering local seismic and site conditions. Nevertheless, spectral acceleration coefficients correspond to so obtain acceleration spectrum ordinates shall in no case be less than those determined by Eq. (4.1) based on relevant characteristic periods specified in Table 4.3.

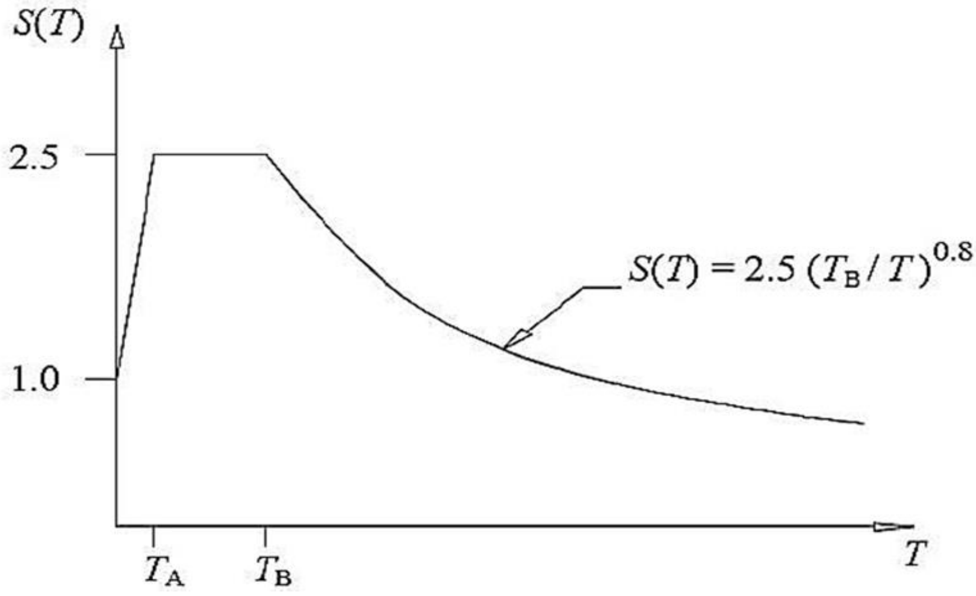


Figure 4.2 Relationship between the building natural period (T) and the spectrum coefficient S(T) (TEC, 2007)

Table 4.4 Soil groups (TEC, 2007)

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

Table 4.5 local site classes (TEC, 2007)

Local Site Class	*Soil Group according to Table 12.1 and Topmost Layer Thickness* ($\mathbf{h_1}$)
Z1	Group **(A)** soils Group **(B)** soils with $\mathbf{h_1 \leq 15\ m}$
Z2	Group **(B)** soils with $\mathbf{h_1 > 15\ m}$ Group **(C)** soils with $\mathbf{h_1 \leq 15\ m}$
Z3	Group **(C)** soils with $\mathbf{15\ m < h_1 \leq 50\ m}$ Group **(D)** soils with $\mathbf{h_1 \leq 10\ m}$
Z4	Group **(C)** soils with $\mathbf{h_1 > 50\ m}$ Group **(D)** soils with $\mathbf{h_1 > 10\ m}$

Elastic seismic loads that found from spectral acceleration coefficient as has been mentioned previously, it is split to below-defined seismic load reduction factor to account for the specific nonlinear behavior of the structural system during an earthquake. Seismic Load Reduction Factor $R_a(T)$, identified by Eq (4.4) in terms of Structural Behavior Factor *(R)* that will describe in Table 4.6.

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad (0 \leq T \leq T_A)$$
$$R_a(T) = R \qquad (T_A < T) \tag{4.4}$$

Table 4.6 Structural behavior factors (R) (TEC, 2007)

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

4.1.2 Rectangular buildings

A 10, 20 and 30 story steel buildings as a frame model was used in order to compare the seismic response of the original structures and retrofitted structures. The dimensions of the internal beam and external beam are (IPE360), they were varied internal column and external column is (IPE400). Also, for slap 4000 Psi concrete and thickness of 0.12m membrane was used. The buildings were modeled considering a series of planar frames connected at each floor. Thus, only three dimensional analysis was conducted. The column foundations were considered as fixed. These frames were designed as six bays on the X-axis and four bays on the Y-axis with 4m story height and 5m width in both directions, the structures were considered as regular in shape in order to carry out the analysis on two dimensional models which easy to interpretation of the results of the analysis.

In the retrofitting of the steel buildings, the structural steel braces in the configuration of concentrically (x) bracing type, the configuration of eccentrically bracing with different link length, link length equal to (L/3), (L/5) and (L/10) form the beam length (L) were used for rectangular building. For the brace elements, pipe section was utilized. At all story buildings, the dimension of the bracing pipe was (TUBO-D193.7X4.5) depend on EURO CODE. Typical floor plan and elevation of the case study steel buildings are given in Figure 4.3 and Figure 4.4 as an example.

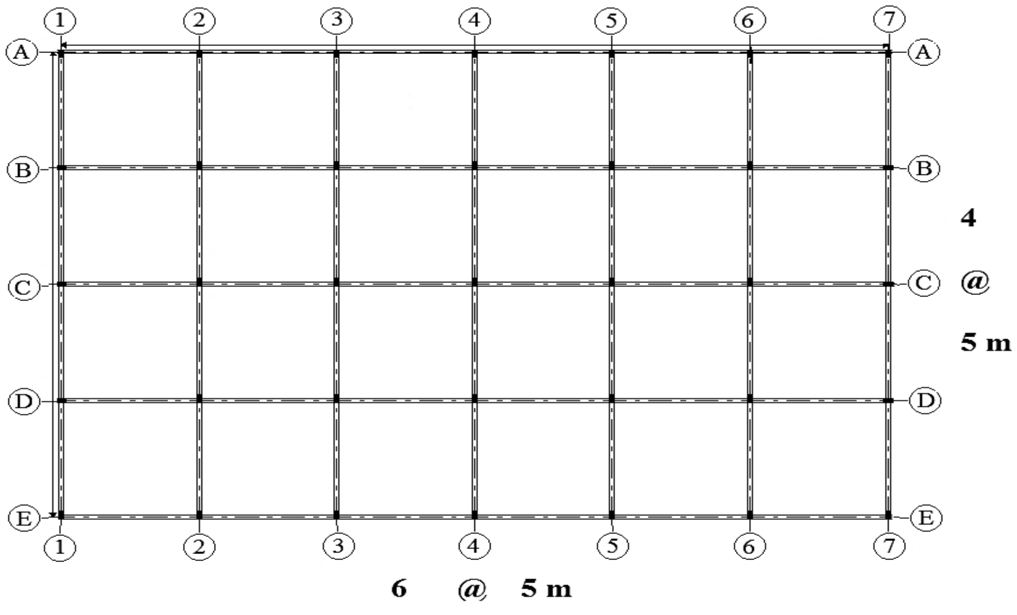
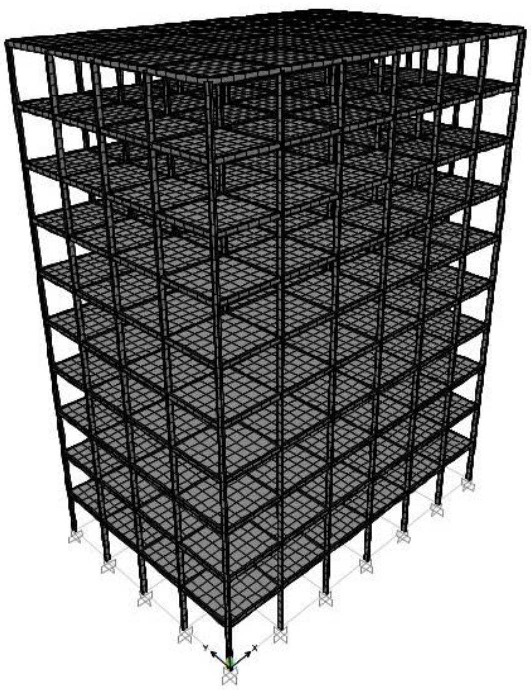
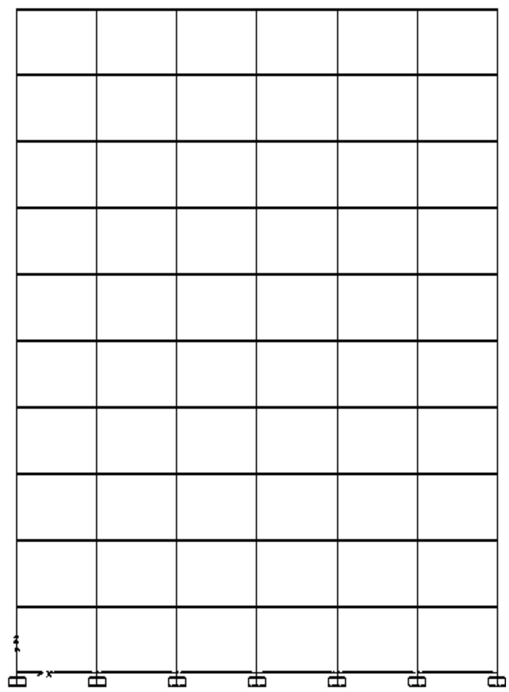


Figure 4.3 Rectangular building plan



a)



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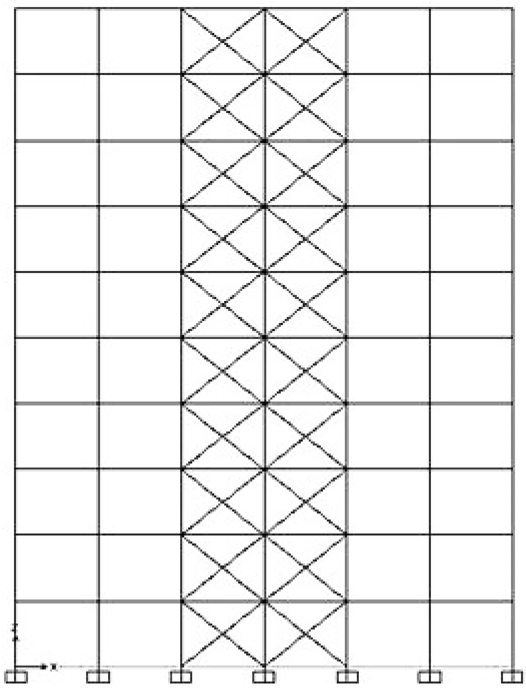

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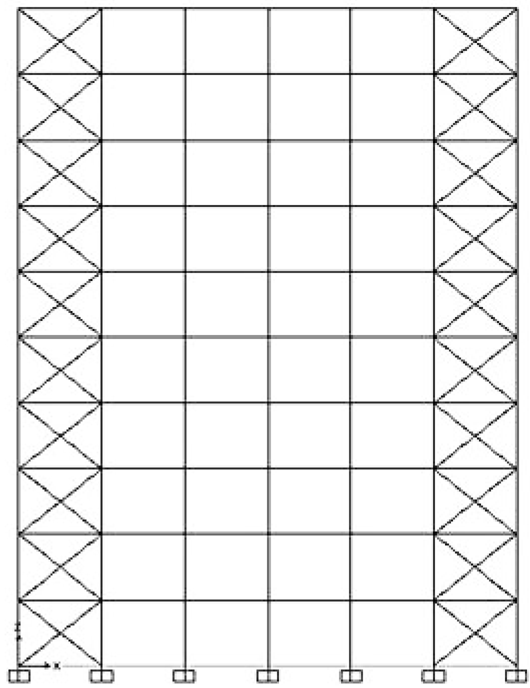


d)

Figure 4.4 Steel building -10- story (a) 3-dimensional view, (b) elevation of the frame, (c) the frame retrofitted with inner x brace and (d) the frame retrofitted with outer x brace.

4.1.3 Circular buildings

A 10, 20 and 30 story steel circular buildings as a frame model was used in order to compare the seismic response of the original structures and retrofitted structures. The dimensions of the internal beam and external beam are (IPE360), they were varied internal column and external column is (IPE400), for slap 4000 Psi concrete and thickness of 0.12m membrane was used. The buildings were modeled considering a series of planar frames connected at each floor. Thus, three dimensional analysis was conducted. The column foundations were considered as fixed. The building outer circumference equal to 78.5 m and core circumference equal to 15.7, the structures were considered as regular in shape in order to carry out the analysis on one dimensional models which easy to interpretation of the results of the analysis.

In the retrofitting of the steel buildings, the structural steel braces in the configuration of concentrically (x) bracing type, the configuration of eccentrically bracing with link length equal (L/5) form the beam length (L) were used in different locations as shown in Figure 4.5. The bracing submit in the outer span, middle span and the span between the core and the middle span. For the brace elements, pipe section was utilized. At all story buildings, the dimension of the bracing pipe was (TUBO-D193.7X4.5) depend on EURO CODE. Typical floor side view and plan of the case study steel buildings are given in Figure 4.6 and Figure 4.7 as an example.

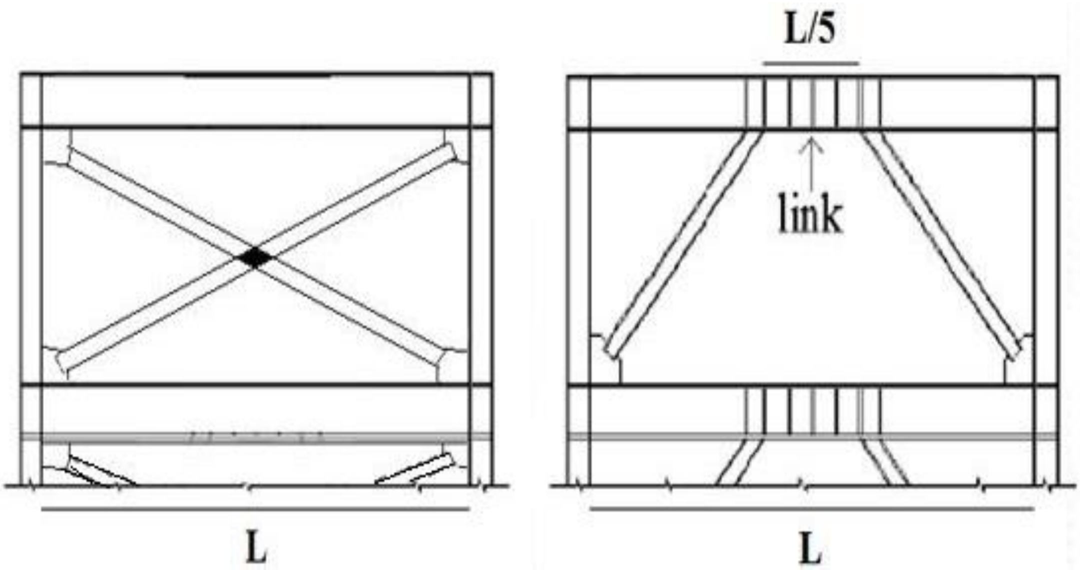
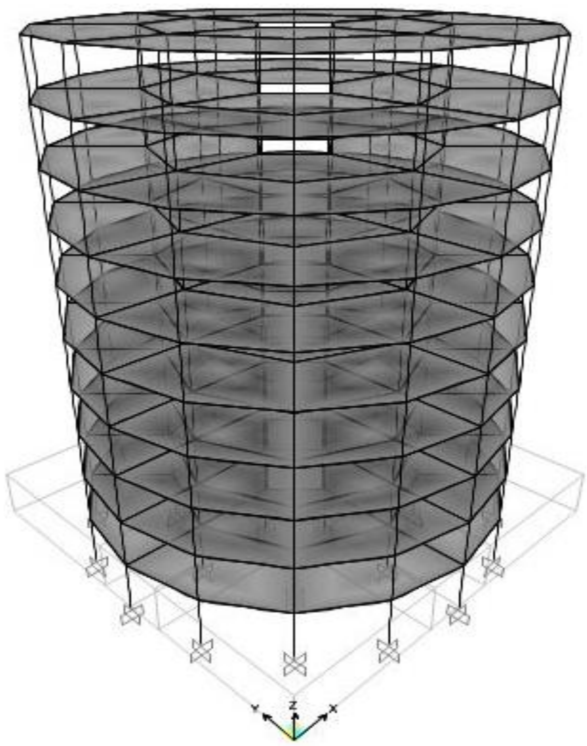
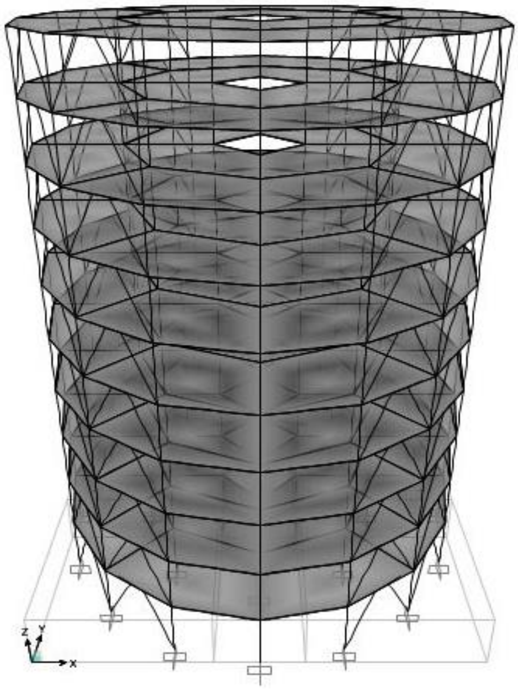


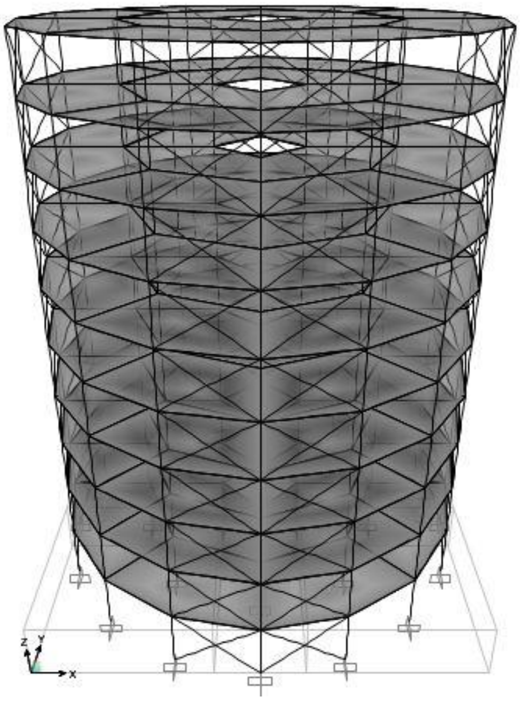
Figure 4.5 Examples on bracing that used in this study



a)



b)



c)

Figure 4.6 Steel building -10- story (a) 3-dimensional view, (b) the frame retrofitted with outer eccentrically braced and (c) the frame retrofitted with outer x bracing.

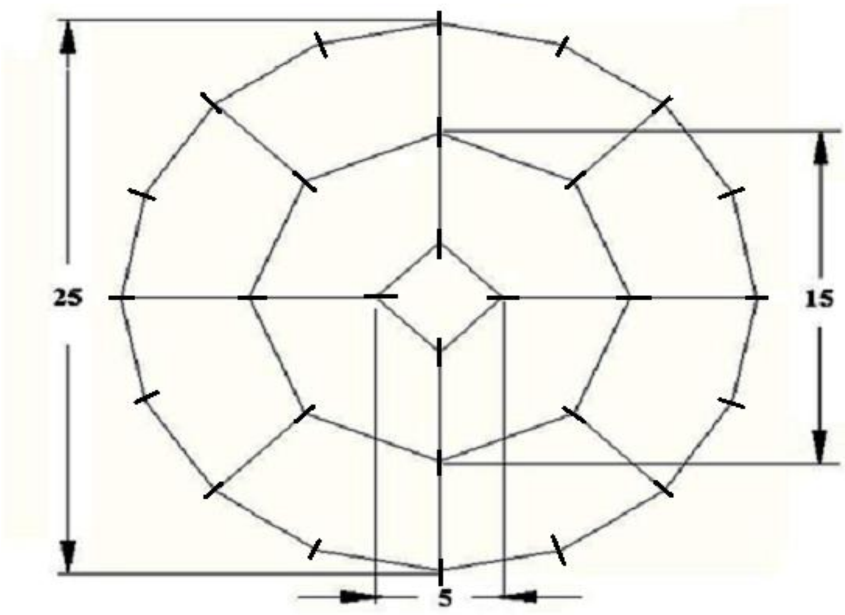


Figure 4.7 Circular building plan.

4.2 Computational Results

In this study, the results of the existing frames, frames retrofitted with concentrically and eccentrically braces obtained response spectrum analysis were given and discussed comparatively. In this study, different cases were considered and structural performance of the original frame, concentrically braced frame and eccentrically braced frame having different number of stories under the effect of earthquake loadings were evaluated. And In comparison, of the analysis results of the three different structural systems of rectangular steel building, three different structural systems of the circular steel building, with the same dimensions are considered for this study on SAP2000 v14 software, and performance characteristics in terms of variation of story displacement were given.

A diagram different value of displacement in a structure was created by this analysis that would detect any premature failure. The analysis results for the different models in the study was chosen the maximum value in all load cases, including load combination, and by using SAP2000 program has calculated for each story & each buildings in this study. When the frames exposed to real ground motion record, the response of it either in forces or in deformations were calculated.

4.2.1 Rectangular buildings

The figures and tables below show the variation of story displacements for the 10, 20 and 30 story original and retrofitted frames, respectively. As seen from the figures, the structures were subjected to ground motion, that it has been applied by the program. It was observed that all structures under investigation were considerably affected by the given earthquake. Moreover, the use of concentrically and eccentrically braced for the purpose of retrofit decreased remarkably the value of the maximum story displacements as compared to original frames, especially in the case of retrofitted with bracing systems.

The maximum story displacement was affected by the number of stories, bracing type and bracing location. For example, in the case of 10 story frame of rectangular plan with x bracing in inner location, the maximum story displacement was smaller than other frames, by increasing numbers of stories the maximum story displacements were also increased.

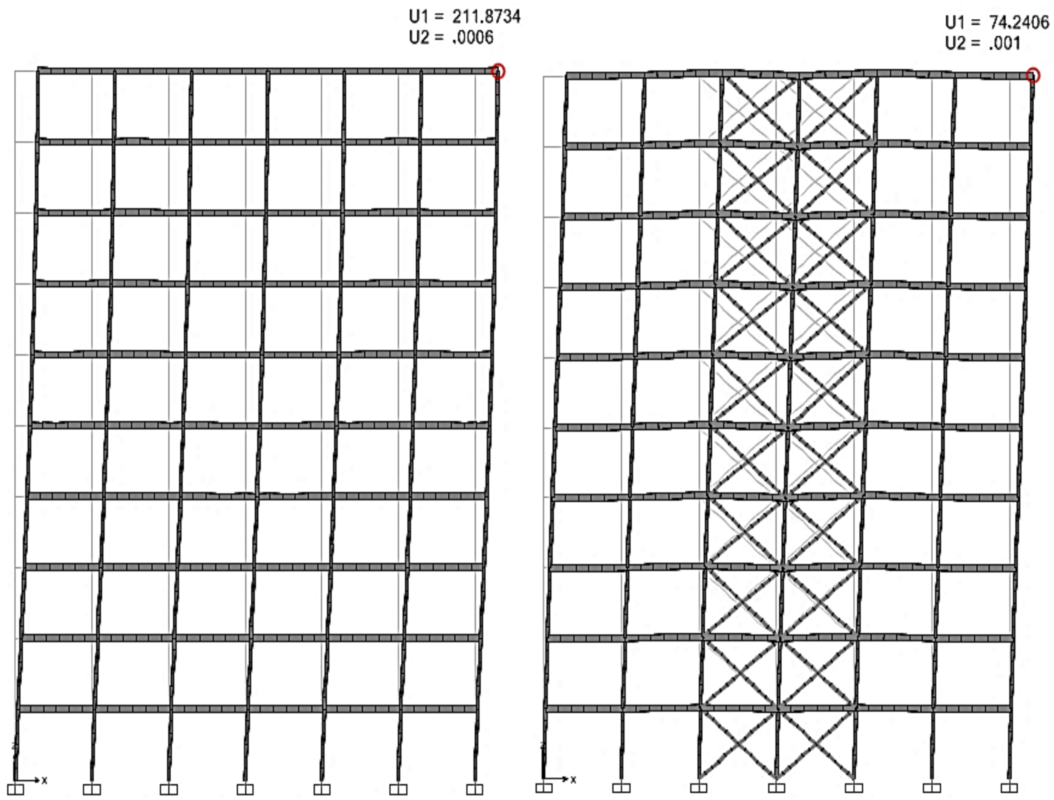


Figure 4.8 The bracing performance for 10 story building under seismic load, showing the reduced in maximum displacement.

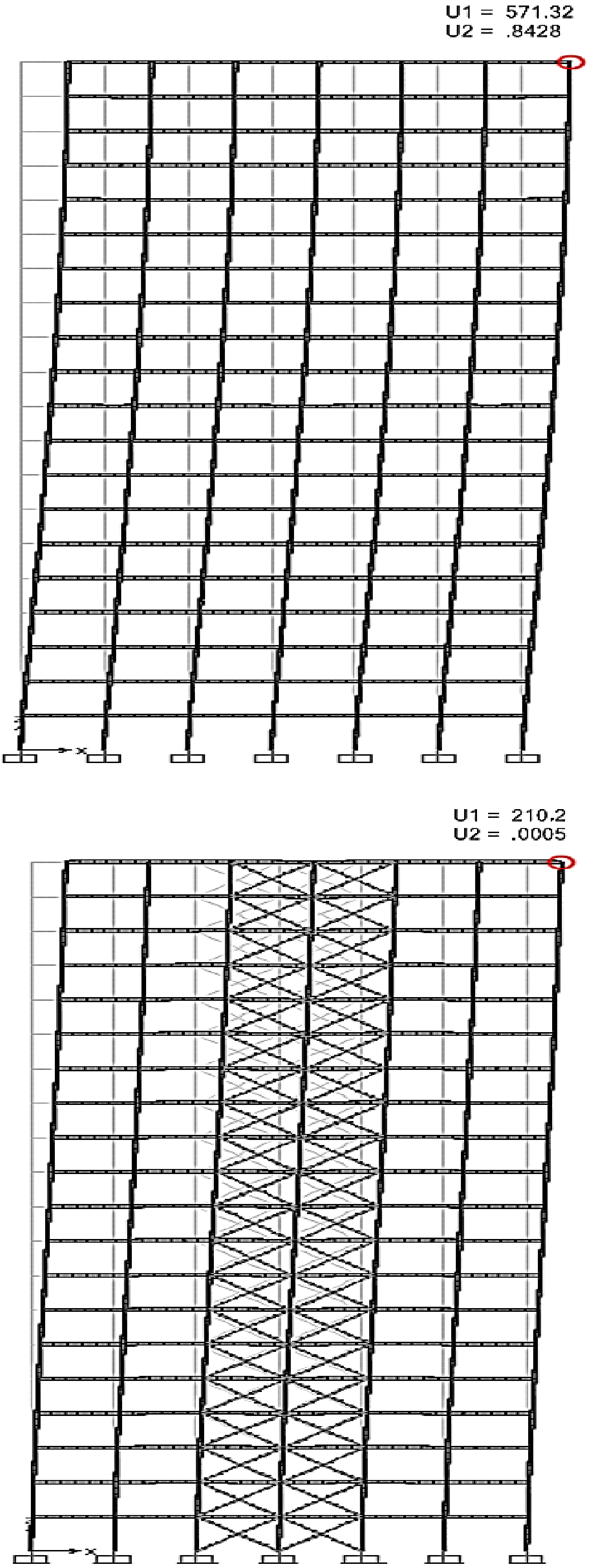
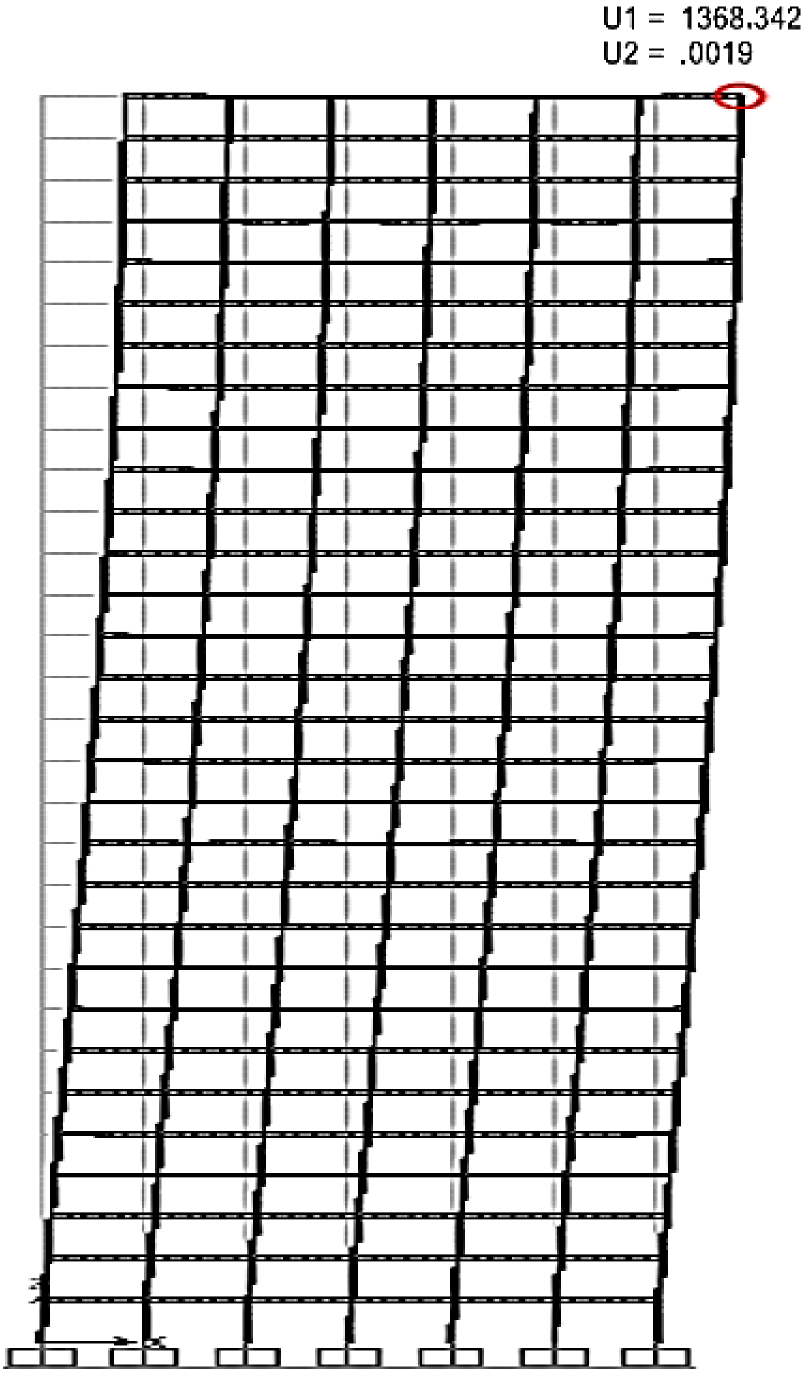
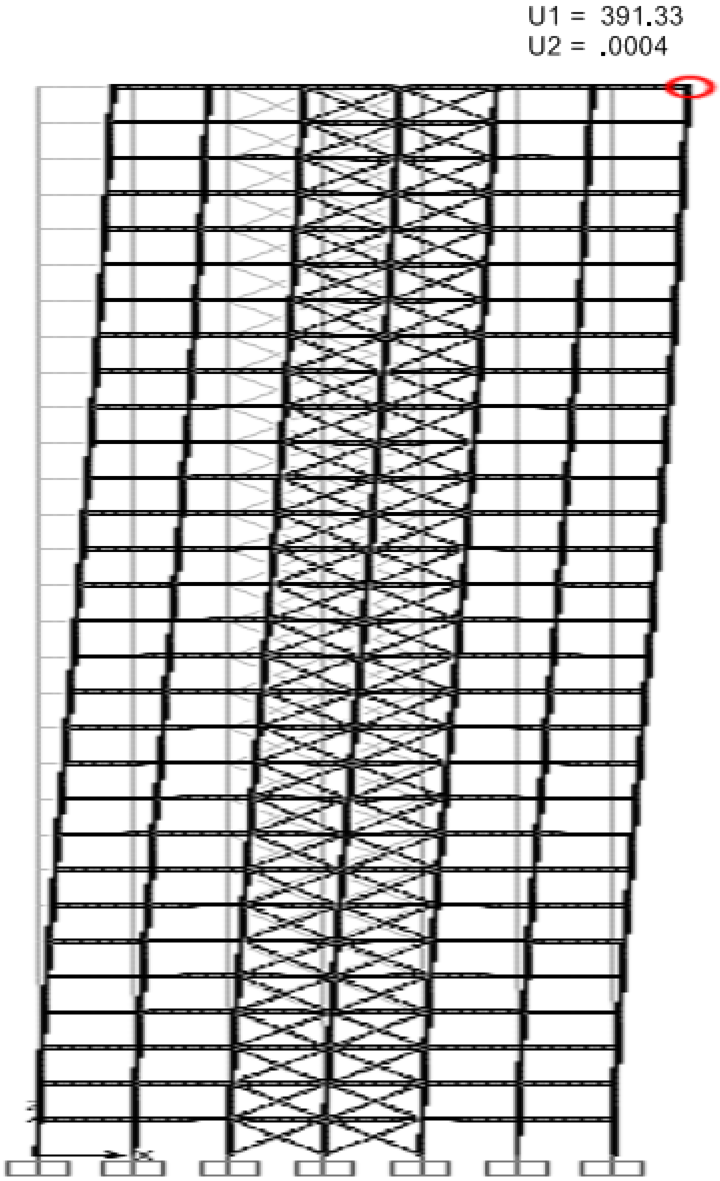


Figure 4.9 The bracing performance for 20 story building under seismic load, showing the reduced in maximum displacement.



a)



b)

Figure 4.10 The bracing performance for 30 story building under seismic load, showing the reduced in maximum displacement (a) Without bracing, (b) With internal bracing

Table 4.7 Result of internal bracing on 10 story building in X direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	23.26	10.79	8.89	7.37	5.41
8	53.58	23.23	19.28	16.85	12.37
12	82.80	35.46	29.79	26.95	20.14
16	110.12	47.26	40.17	37.38	28.35
20	135.12	58.38	50.15	47.86	36.73
24	157.42	68.57	59.50	58.16	45.05
28	176.64	77.60	68.02	68.01	53.13
32	192.37	85.24	75.46	77.18	60.80
36	204.18	91.24	81.60	85.38	67.90
40	211.87	95.35	86.16	92.26	74.24

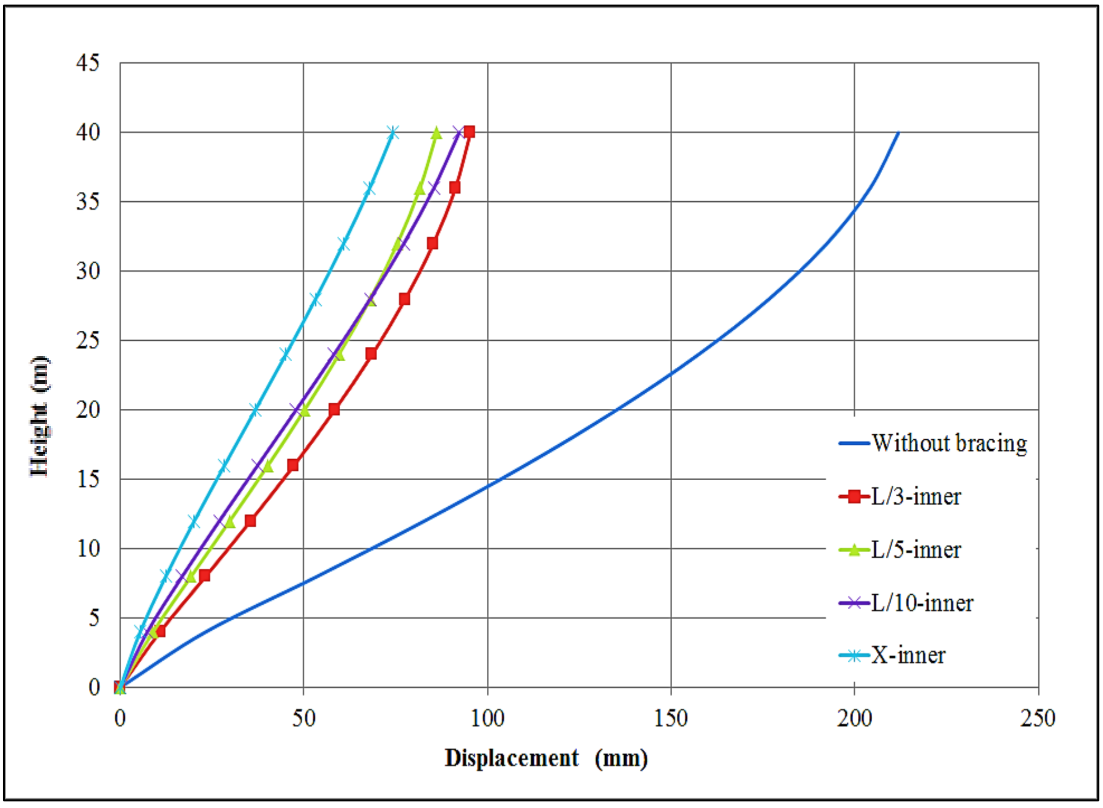


Figure 4.11 The effect of internal bracing on 10 story building in X direction.

Table 4.8 Result of internal bracing on 10 story building in Y direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	10.45	7.61	6.76	5.88	4.70
8	24.30	16.94	15.03	14.35	10.76
12	37.74	26.01	23.22	23.27	17.30
16	50.28	34.61	31.11	32.22	24.02
20	61.78	42.60	38.55	40.97	30.71
24	72.08	49.84	45.41	49.36	37.18
28	80.99	56.23	51.56	57.19	43.29
32	88.32	61.61	56.85	64.28	48.90
36	93.88	65.79	61.11	70.41	53.90
40	97.48	68.58	64.10	75.20	58.07

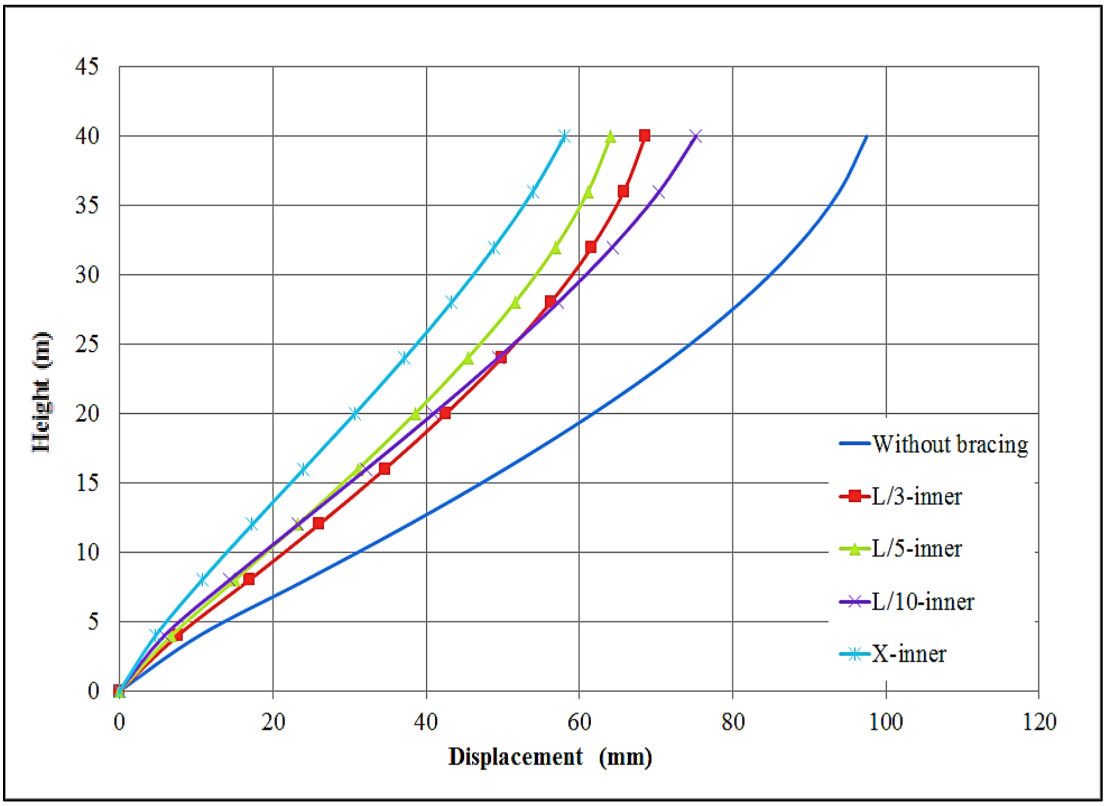


Figure 4.12 The effect of internal bracing on 10 story building in Y direction

Table 4.9 Result of outer bracing on 10 story building in X direction

Height (m)	Without	L/3-outer	L/5-outer	L/10-outer	X-outer
0	0	0	0	0	0
4	23.26	9.86	8.10	6.89	5.37
8	53.58	22.06	18.45	16.38	13.12
12	82.80	34.71	29.67	27.19	22.31
16	110.12	47.48	41.31	38.86	32.38
20	135.12	60.03	53.01	50.99	42.92
24	157.42	72.04	64.45	63.23	53.62
28	176.64	83.19	75.32	75.26	64.26
32	192.37	93.20	85.34	86.77	74.62
36	204.18	101.73	94.20	97.46	84.52
40	211.87	108.45	101.59	106.97	93.77

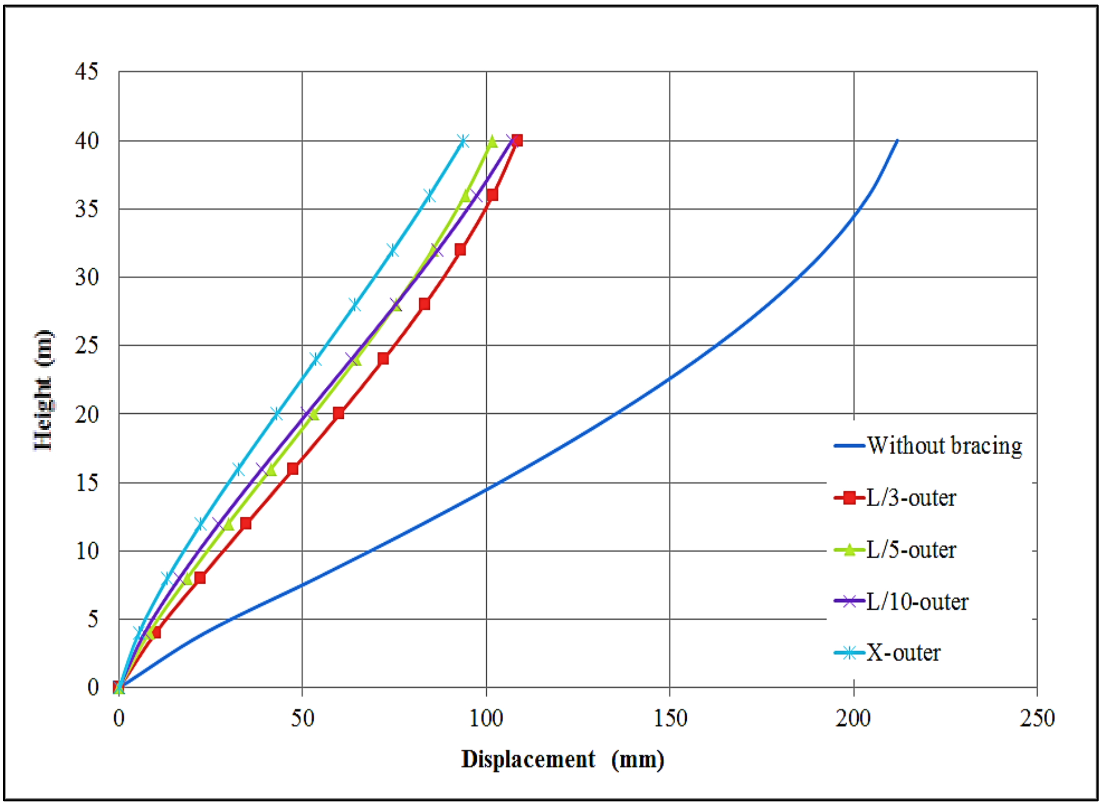


Figure 4.13 The effect of outer bracing on 10 story building in X direction

Table 4.10 Result of outer bracing on 10 story building in Y direction

Height (m)	Without	L/3-outer	L/5-outer	L/10-outer	X-outer
0	0	0	0	0	0
4	10.45	7.31	6.45	5.84	4.68
8	24.30	16.79	14.94	14.20	11.31
12	37.74	26.37	23.77	23.35	18.82
16	50.28	35.71	32.58	32.83	26.73
20	61.78	44.59	41.10	42.34	34.69
24	72.08	52.83	49.15	51.60	42.48
28	80.99	60.28	56.55	60.40	49.92
32	88.32	66.73	63.11	68.54	56.85
36	93.88	71.97	68.62	75.76	63.17
40	97.48	75.80	72.88	81.80	68.72

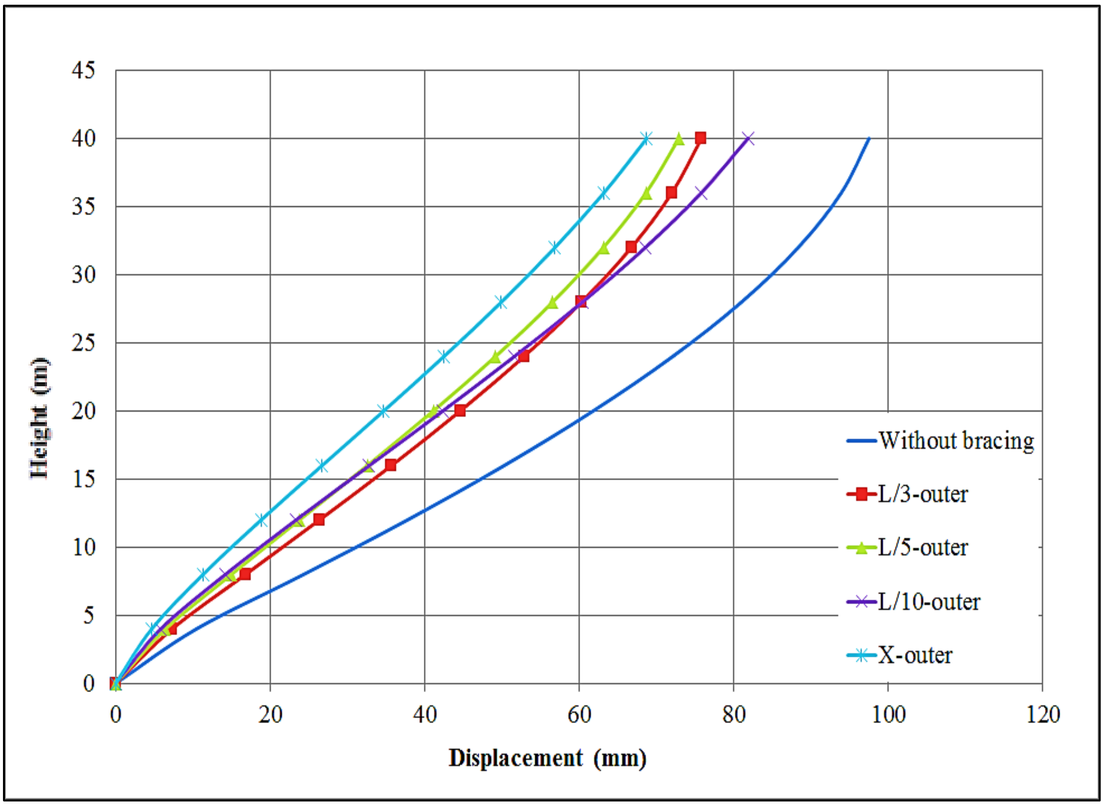


Figure 4.14 The effect of outer bracing on 10 story building in Y direction

Table 4.11 Result of internal bracing on 20 story building in X direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	28.79	11.44	9.33	7.59	5.69
8	67.88	25.35	20.91	17.85	13.58
12	107.53	39.90	33.44	29.64	23.01
16	146.92	54.88	46.66	42.63	33.58
20	185.77	70.10	60.34	56.53	44.96
24	223.86	85.41	74.29	71.11	56.93
28	260.95	100.64	88.35	86.17	69.26
32	296.82	115.66	102.34	101.51	81.79
36	331.29	130.33	116.13	116.95	94.37
40	364.18	144.55	129.61	132.34	106.86
44	395.30	158.23	142.67	147.54	119.15
48	424.51	171.25	155.22	162.45	131.15
52	451.65	183.55	167.18	176.96	142.78
56	476.55	195.05	178.46	190.97	153.98
60	499.06	205.65	188.99	204.40	164.69
64	519.04	215.29	198.71	217.17	174.88
68	536.35	223.89	207.56	229.23	184.53
72	550.83	231.39	215.47	240.51	193.63
76	562.46	237.71	222.41	250.98	202.21
80	571.32	242.77	228.26	260.49	210.21

Table 4.12 Result of internal bracing on 20 story building in Y direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	11.34	7.97	7.03	6.40	4.93
8	26.92	18.22	16.11	15.35	11.70
12	42.94	28.85	25.74	25.24	19.49
16	58.95	39.67	35.73	35.80	28.03
20	74.80	50.59	45.95	46.84	37.11
24	90.38	61.55	56.31	58.21	46.56
28	105.65	72.48	66.74	69.78	56.27
32	120.55	83.34	77.18	81.45	66.12
36	135.06	94.07	87.56	93.12	76.03
40	149.10	104.59	97.80	104.69	85.92
44	162.62	114.87	107.86	116.09	95.73
48	175.55	124.85	117.68	127.24	105.39
52	187.81	134.48	127.20	138.09	114.84
56	199.34	143.69	136.37	148.56	124.04
60	210.07	152.43	145.12	158.60	132.94
64	219.94	160.63	153.39	168.13	141.50
68	228.85	168.21	161.12	177.11	149.68
72	236.70	175.11	168.25	185.46	157.45
76	243.48	181.25	174.71	193.15	164.80
80	248.94	186.53	180.38	199.97	171.63

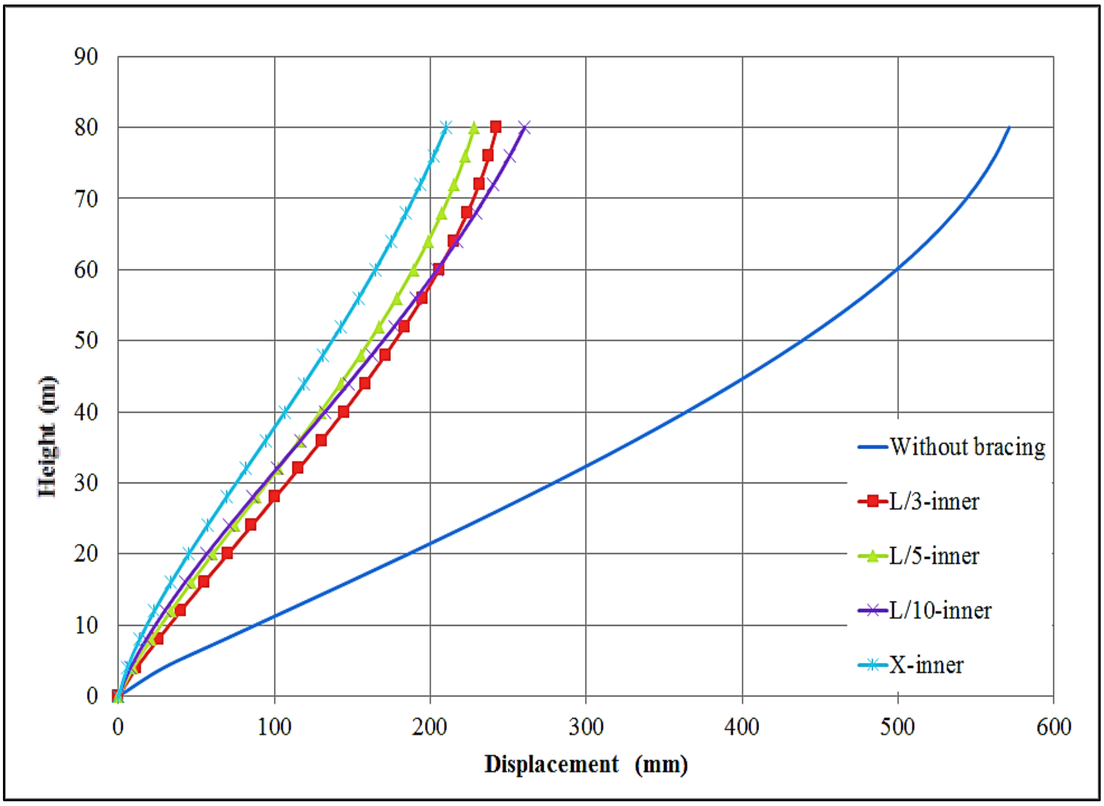


Figure 4.15 The effect of internal bracing on 20 story building in X direction

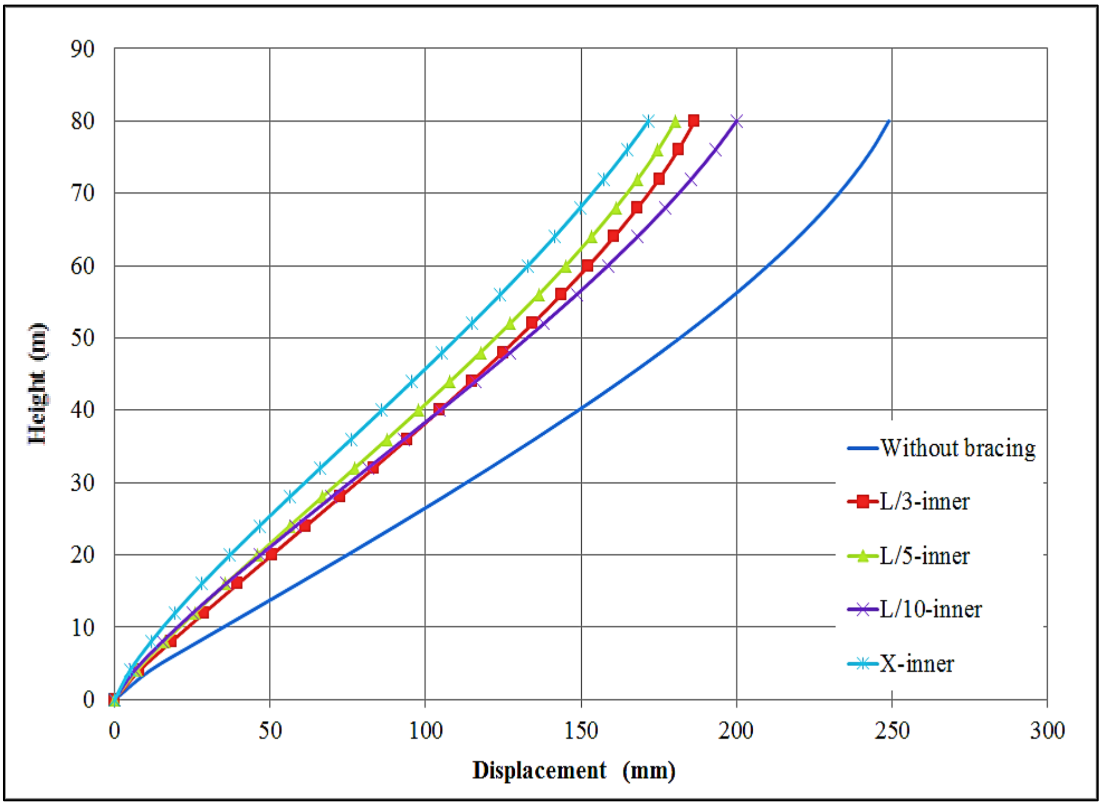


Figure 4.16 The effect of internal bracing on 20 story building in Y direction

Table 4.13 Result of outer bracing on 20 story building in X direction

Height (m)	Without	L/3-outer	L/5-outer	L/10-outer	X-outer
0	0	0	0	0	0
4	28.79	9.91	8.03	6.74	5.60
8	67.88	23.20	19.32	17.12	14.47
12	107.53	38.09	32.55	30.02	25.73
16	146.92	54.20	47.27	44.95	38.80
20	185.77	71.24	63.12	61.50	53.23
24	223.86	88.94	79.82	79.31	68.69
28	260.95	107.07	97.09	98.08	84.90
32	296.82	125.39	114.70	117.52	101.60
36	331.29	143.74	132.44	137.39	118.61
40	364.18	161.95	150.15	157.47	135.75
44	395.30	179.87	167.68	177.61	152.87
48	424.51	197.39	184.90	197.63	169.83
52	451.65	214.37	201.69	217.40	186.55
56	476.55	230.70	217.94	236.79	202.93
60	499.06	246.30	233.55	255.70	218.89
64	519.04	261.04	248.45	274.04	234.38
68	536.35	274.84	262.54	291.73	249.35
72	550.83	287.61	275.78	308.70	263.81
76	562.46	299.27	288.10	324.92	277.76
80	571.32	309.78	299.48	340.38	291.23

Table 4.14 Result of outer bracing on 20 story building in Y direction

Height (m)	Without	L/3-outer	L/5-outer	L/10-outer	X-outer
0	0	0	0	0	0
4	11.34	7.55	6.42	5.91	4.97
8	26.92	17.96	15.49	15.11	12.52
12	42.94	29.21	25.62	25.98	21.63
16	58.95	41.00	36.47	38.06	31.83
20	74.80	53.12	47.78	51.00	42.77
24	90.38	65.43	59.39	64.54	54.20
28	105.65	77.83	71.18	78.48	65.92
32	120.55	90.20	83.01	92.62	77.77
36	135.06	102.45	94.78	106.80	89.63
40	149.10	114.48	106.38	120.88	101.39
44	162.62	126.22	117.74	134.74	112.94
48	175.55	137.60	128.78	148.29	124.22
52	187.81	148.56	139.44	161.44	135.15
56	199.34	159.03	149.66	174.10	145.68
60	210.07	168.93	159.37	186.21	155.75
64	219.94	178.21	168.50	197.70	165.33
68	228.85	186.79	177.02	208.53	174.39
72	236.70	194.62	184.86	218.66	182.93
76	243.48	201.62	191.98	228.05	190.95
80	248.94	207.74	198.33	236.65	198.47

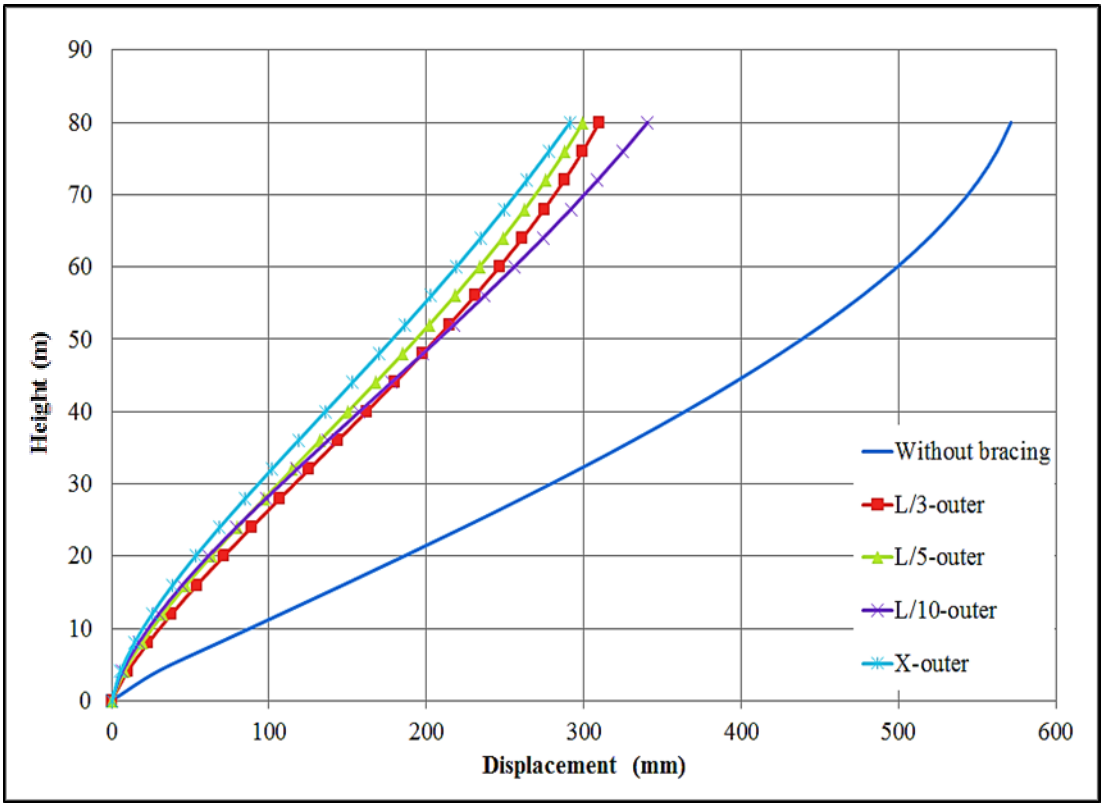


Figure 4.17 The effect of outer bracing on 20 story building in X direction

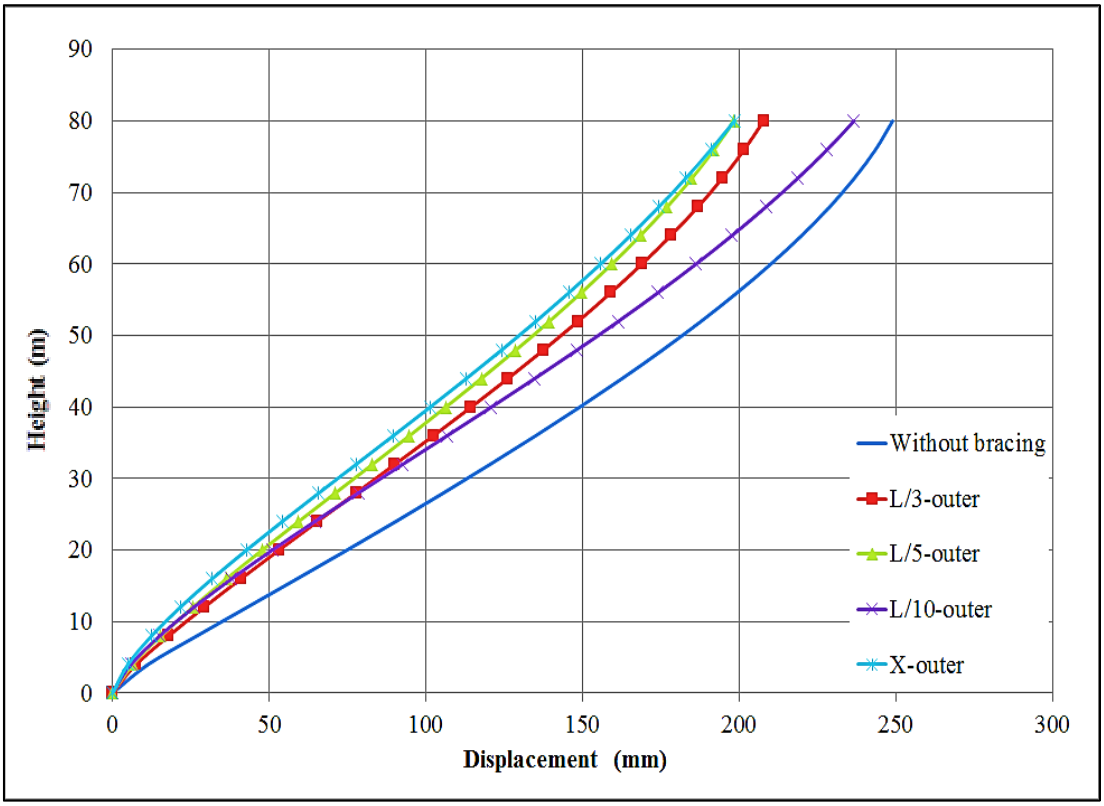


Figure 4.18 The effect of outer bracing on 20 story building in Y direction

Table 4.15 Result of internal bracing on 30 story building in X direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	41.77	11.63	9.39	7.65	5.76
8	99.34	26.15	21.41	18.49	14.08
12	159.00	41.75	34.82	31.39	24.31
16	219.61	58.20	49.33	46.04	36.08
20	280.74	75.26	64.67	62.12	49.03
24	342.07	92.76	80.64	79.37	62.90
28	403.34	110.57	97.09	97.57	77.48
32	464.30	128.58	113.88	116.52	92.60
36	524.72	146.69	130.88	136.05	108.09
40	584.38	164.79	147.99	155.99	123.85
44	643.07	182.79	165.10	176.19	139.76
48	700.57	200.62	182.13	196.51	155.72
52	756.72	218.22	199.00	216.85	171.65
56	811.34	235.51	215.65	237.09	187.46
60	864.27	252.46	232.02	257.16	203.10
64	915.36	269.00	248.06	276.96	218.51
68	964.45	285.09	263.72	296.42	233.64
72	1011.39	300.68	278.94	315.47	248.44
76	1056.07	315.72	293.68	334.05	262.88
80	1098.35	330.18	307.92	352.10	276.91
84	1138.11	344.01	321.60	369.59	290.52
88	1175.26	357.18	334.69	386.47	303.66
92	1209.68	369.65	347.16	402.71	316.33
96	1241.28	381.37	358.97	418.27	328.50
100	1269.97	392.32	370.09	433.14	340.16
104	1295.69	402.45	380.51	447.31	351.33
108	1318.38	411.74	390.20	460.76	362.00
112	1337.97	420.16	399.14	473.51	372.20
116	1354.47	427.69	407.33	485.58	381.98
120	1368.34	434.26	414.71	496.88	391.33

Table 4.16 Result of internal bracing on 30 story building in Y direction

Height (m)	Without	L/3-inner	L/5-inner	L/10-inner	X-inner
0	0	0	0	0	0
4	11.67	7.87	6.92	6.22	4.91
8	27.96	18.26	16.13	15.27	11.89
12	45.05	29.35	26.20	25.64	20.19
16	62.57	41.00	36.98	37.09	29.57
20	80.36	53.07	48.33	49.42	39.79
24	98.33	65.50	60.12	62.45	50.70
28	116.40	78.23	72.31	76.05	62.17
32	134.52	91.22	84.82	90.12	74.10
36	152.65	104.43	97.61	104.57	86.42
40	170.76	117.83	110.65	119.33	99.05
44	188.81	131.39	123.88	134.33	111.95
48	206.79	145.06	137.26	149.49	125.05
52	224.66	158.82	150.77	164.76	138.32
56	242.38	172.62	164.35	180.09	151.70
60	259.94	186.44	177.98	195.42	165.17
64	277.27	200.25	191.62	210.72	178.68
68	294.34	214.01	205.25	225.93	192.21
72	311.11	227.70	218.83	241.03	205.72
76	327.53	241.27	232.33	255.97	219.17
80	343.58	254.70	245.71	270.72	232.55
84	359.19	267.94	258.94	285.23	245.81
88	374.34	280.95	271.97	299.46	258.91
92	388.97	293.69	284.77	313.38	271.84
96	403.04	306.12	297.29	326.95	284.55
100	416.50	318.20	309.50	340.14	297.02
104	429.28	329.88	321.35	352.91	309.23
108	441.32	341.11	332.81	365.25	321.15
112	452.56	351.85	343.84	377.11	332.77
116	462.96	362.04	354.39	388.49	344.09
120	472.51	371.64	364.39	399.24	355.06

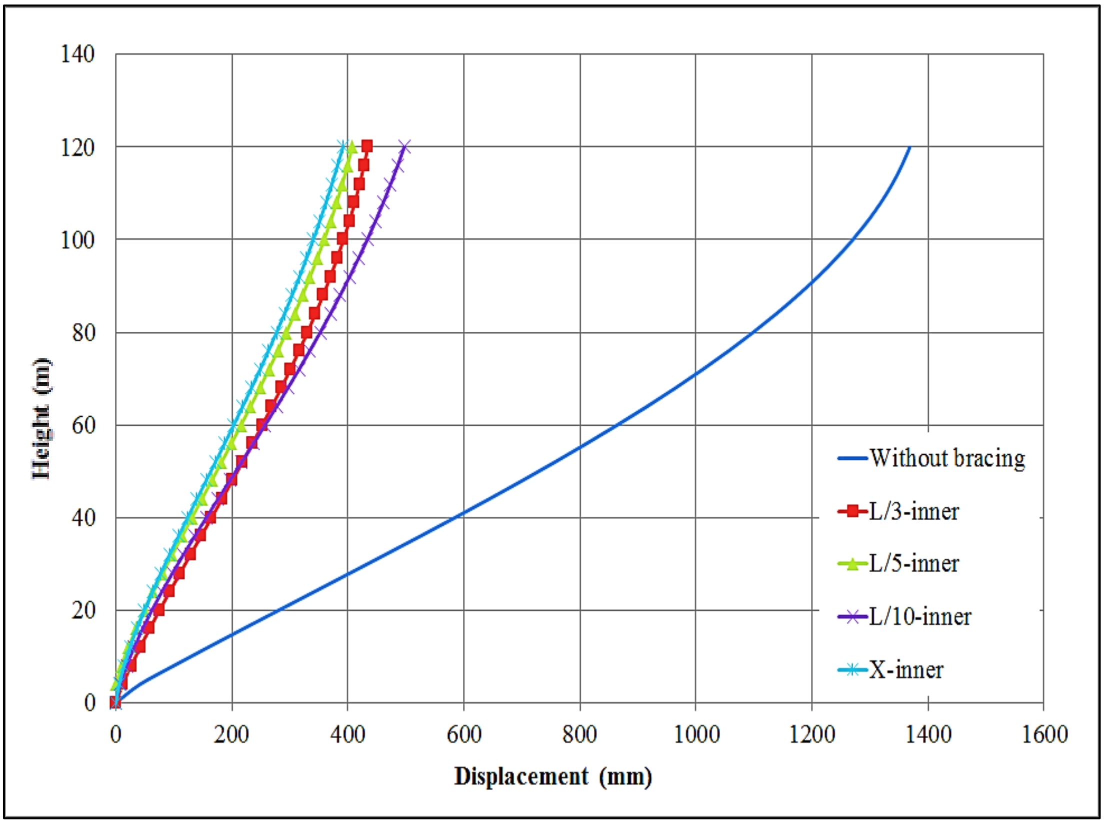


Figure 4.19 The effect of internal bracing on 30 story building in X direction

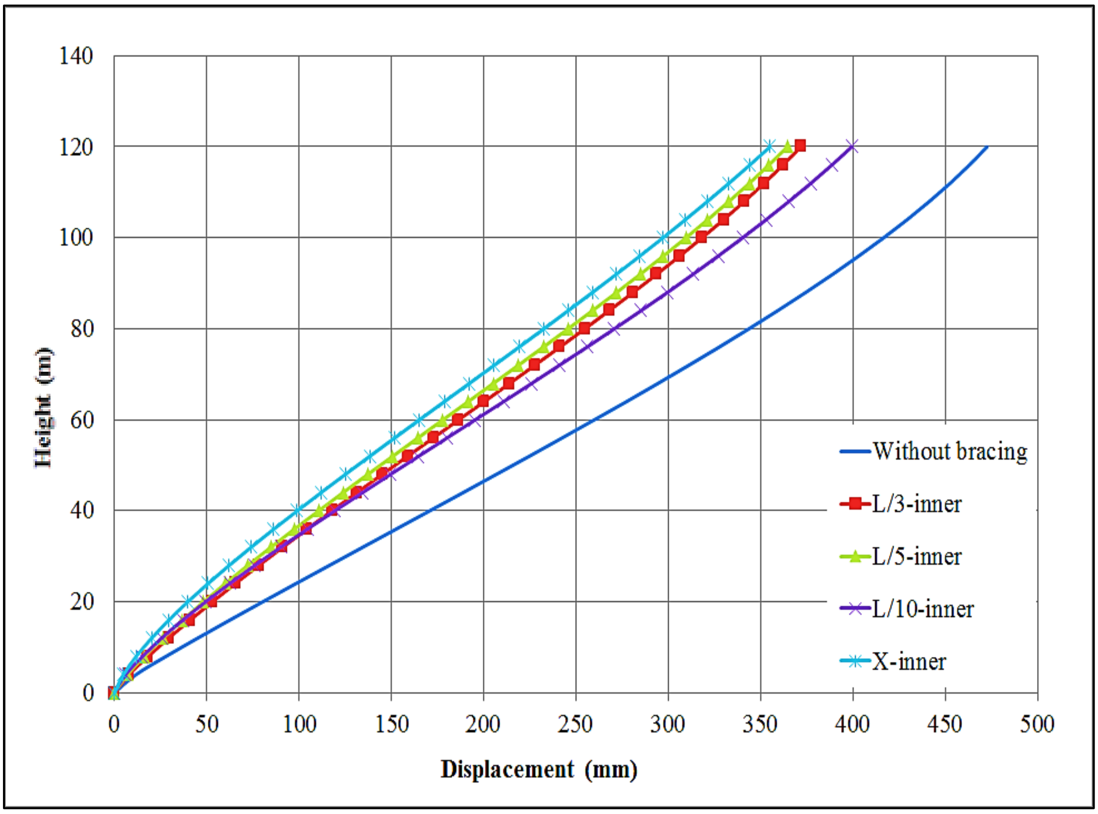


Figure 4.20 The effect of internal bracing on 30 story building in Y direction

Table 4.17 Result of outer bracing on 30 story building in X direction

Height (m)	Without	L/3-outer	L/5-inner	L/10-outer	X-outer
0	0	0	0	0	0
4	41.77	10.78	8.55	7.40	6.06
8	99.34	26.06	21.38	20.00	16.22
12	159.00	43.97	37.13	36.63	29.57
16	219.61	64.04	55.26	56.63	45.50
20	280.74	85.85	75.30	79.47	63.50
24	342.07	109.06	96.90	104.67	83.18
28	403.34	133.42	119.74	131.86	104.21
32	464.30	158.67	143.58	160.68	126.32
36	524.72	184.62	168.20	190.84	149.28
40	584.38	211.05	193.37	222.04	172.88
44	643.07	237.79	218.93	254.05	196.95
48	700.57	264.68	244.71	286.63	221.33
52	756.72	291.58	270.57	319.58	245.88
56	811.34	318.38	296.38	352.73	270.47
60	864.27	344.94	322.03	385.91	294.98
64	915.36	371.18	347.41	418.97	319.31
68	964.45	396.98	372.44	451.78	343.38
72	1011.39	422.26	397.01	484.20	367.10
76	1056.07	446.93	421.05	516.14	390.41
80	1098.35	470.93	444.49	547.50	413.25
84	1138.11	494.18	467.28	578.19	435.55
88	1175.26	516.63	489.34	608.13	457.28
92	1209.68	538.21	510.63	637.28	478.40
96	1241.28	558.88	531.12	665.57	498.89
100	1269.97	578.60	550.76	692.98	518.74
104	1295.69	597.33	569.54	719.50	537.95
108	1318.38	615.06	587.46	745.13	556.55
112	1337.97	631.78	604.51	769.89	574.57
116	1354.47	647.48	620.72	793.85	592.09
120	1368.34	662.23	636.15	817.12	609.20

Table 4.18 Result of outer bracing on 30 story building in Y direction

Height (m)	Without	L/3-outer	L/5-outer	L/10-outer	X-outer
0	0	0	0	0	0
4	11.67	7.46	6.37	5.78	4.98
8	27.96	18.06	15.71	15.20	12.84
12	45.05	29.91	26.50	26.77	22.61
16	62.57	42.69	38.41	40.03	33.83
20	80.36	56.19	51.18	54.60	46.16
24	98.33	70.25	64.61	70.20	59.33
28	116.40	84.73	78.55	86.60	73.15
32	134.52	99.56	92.90	103.61	87.46
36	152.65	114.65	107.56	121.10	102.15
40	170.76	129.95	122.47	138.94	117.13
44	188.81	145.39	137.54	157.02	132.30
48	206.79	160.91	152.73	175.24	147.61
52	224.66	176.45	167.98	193.51	162.97
56	242.38	191.98	183.23	211.75	178.35
60	259.94	207.45	198.43	229.89	193.68
64	277.27	222.82	213.55	247.88	208.92
68	294.34	238.05	228.55	265.67	224.05
72	311.11	253.11	243.40	283.20	239.01
76	327.53	267.96	258.05	300.42	253.78
80	343.58	282.56	272.47	317.30	268.32
84	359.19	296.87	286.61	333.79	282.58
88	374.34	310.85	300.45	349.84	296.55
92	388.97	324.46	313.94	365.43	310.18
96	403.04	337.65	327.05	380.52	323.45
100	416.50	350.40	339.73	395.08	336.34
104	429.28	362.65	351.97	409.10	348.82
108	441.32	374.38	363.72	422.57	360.91
112	452.56	385.54	374.97	435.47	372.58
116	462.96	396.10	385.69	447.81	383.88
120	472.51	406.05	395.87	459.57	394.82

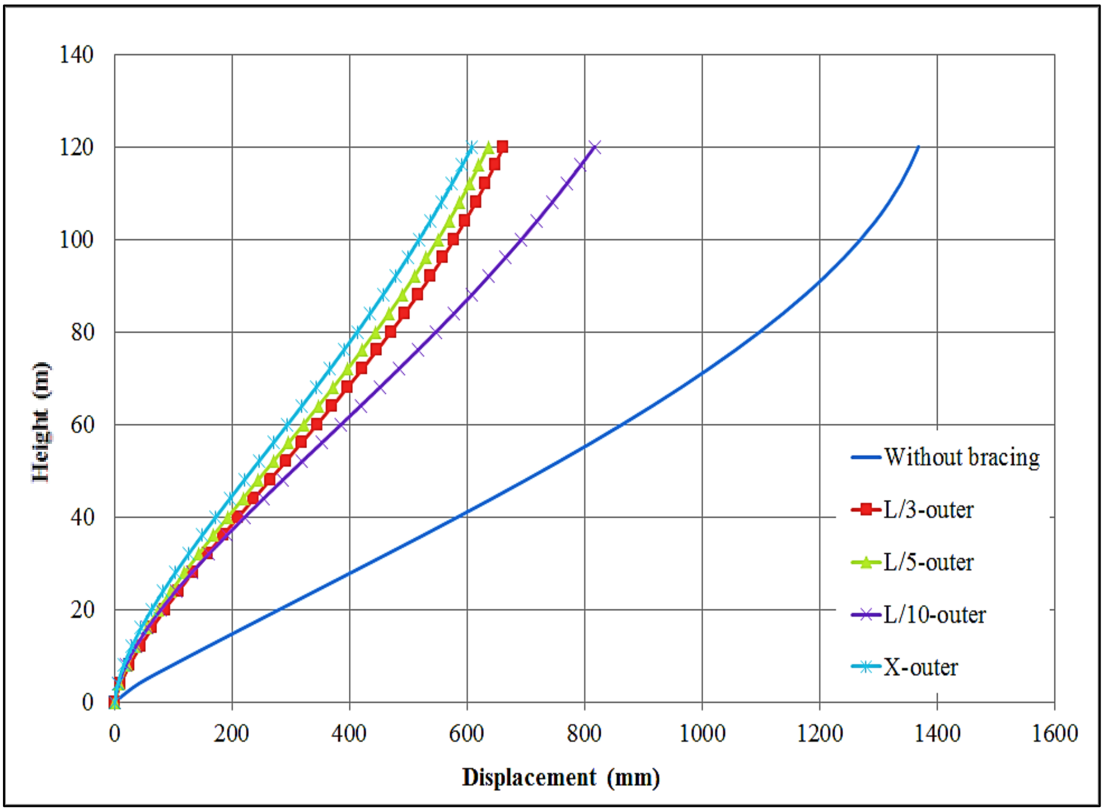


Figure 4.21 The effect of outer bracing on 30 story building in X direction

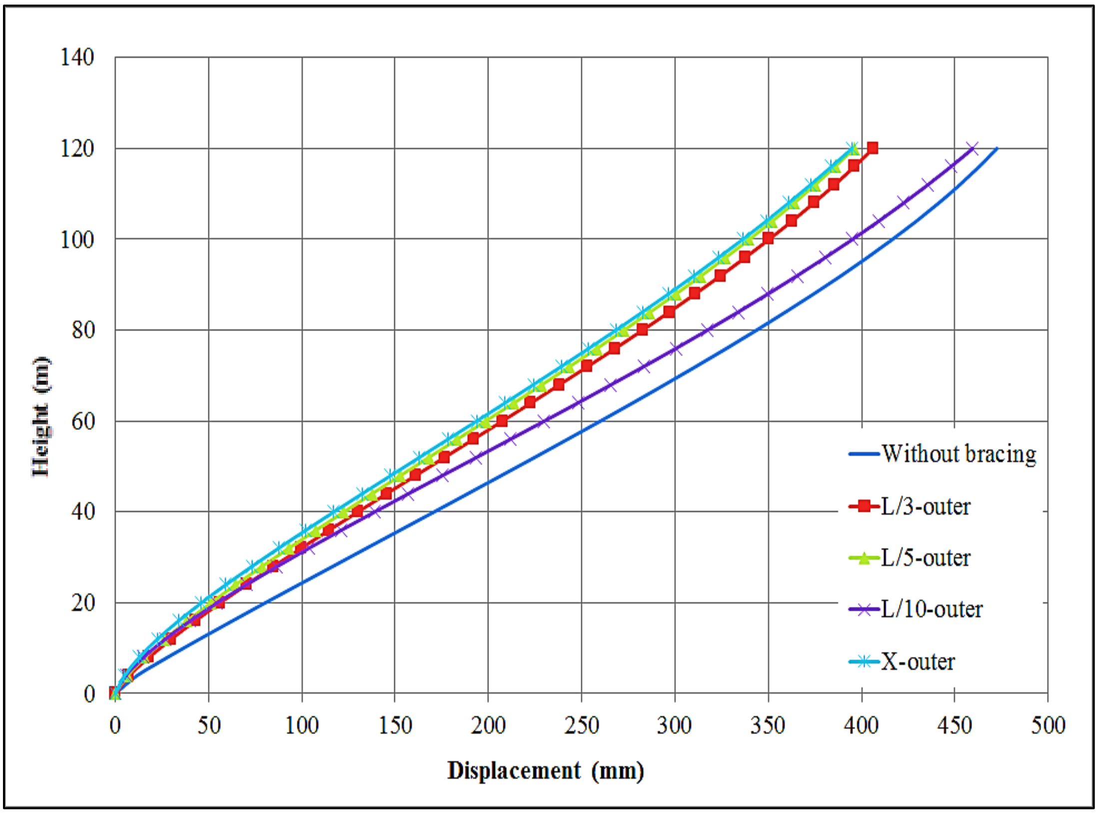


Figure 4.22 The effect of outer bracing on 30 story building in Y direction

4.2.2 Circular buildings

The figures below show the variation of story displacements for the 10, 20 and 30 story original and retrofitted frames of circular buildings, respectively. As seen from the figures, the structures were subjected to ground motion, that it has been applied by the program. It was observed that all structures under investigation were considerably affected by the given earthquake. Moreover, the use of concentrically and eccentrically braced in a different location for the purpose of retrofit decreased remarkably the value of the maximum story displacements as compared to original frames, especially in the case of retrofitted with braces systems.

The maximum story displacement was affected by the number of stories, bracing type and bracing location. For example, in the case of 10 story frame of circular plan with x bracing in middle span, the maximum story displacement was smaller than other frames, by increasing numbers of stories the maximum story displacements were also increased. The circular building has excited result showing the effect of bracing could change depending on the location, bracing type and building height. The results and figures is shown below for circular plan.

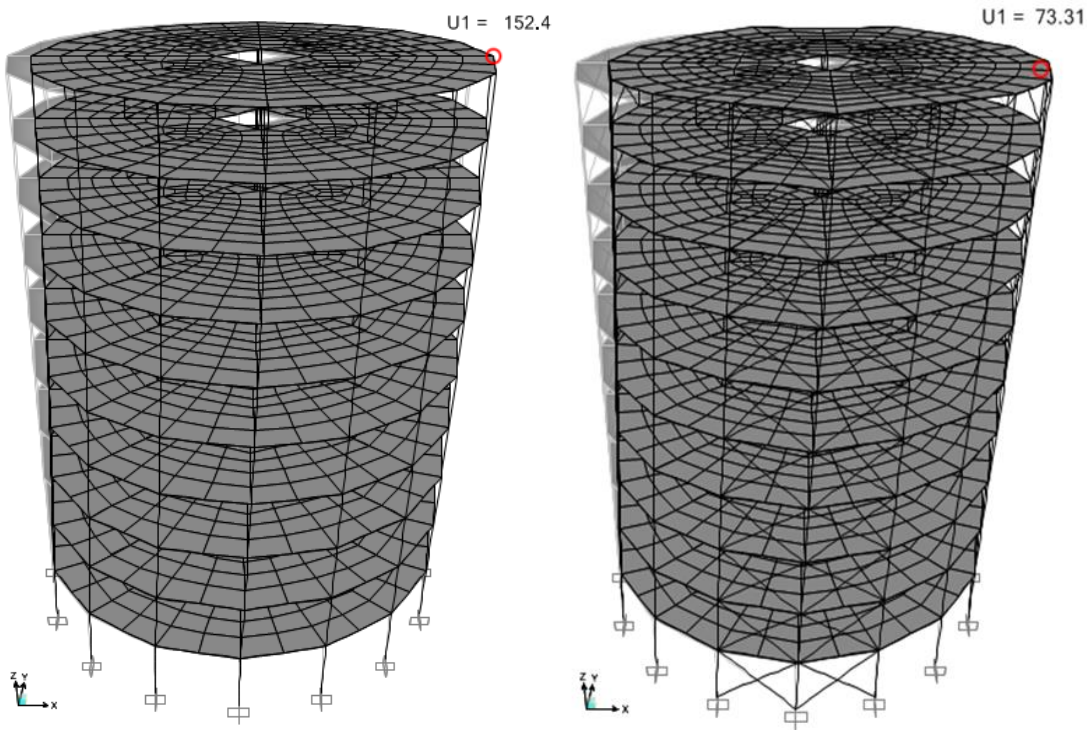


Figure 4.23 The bracing performance for 10 story building under seismic load.

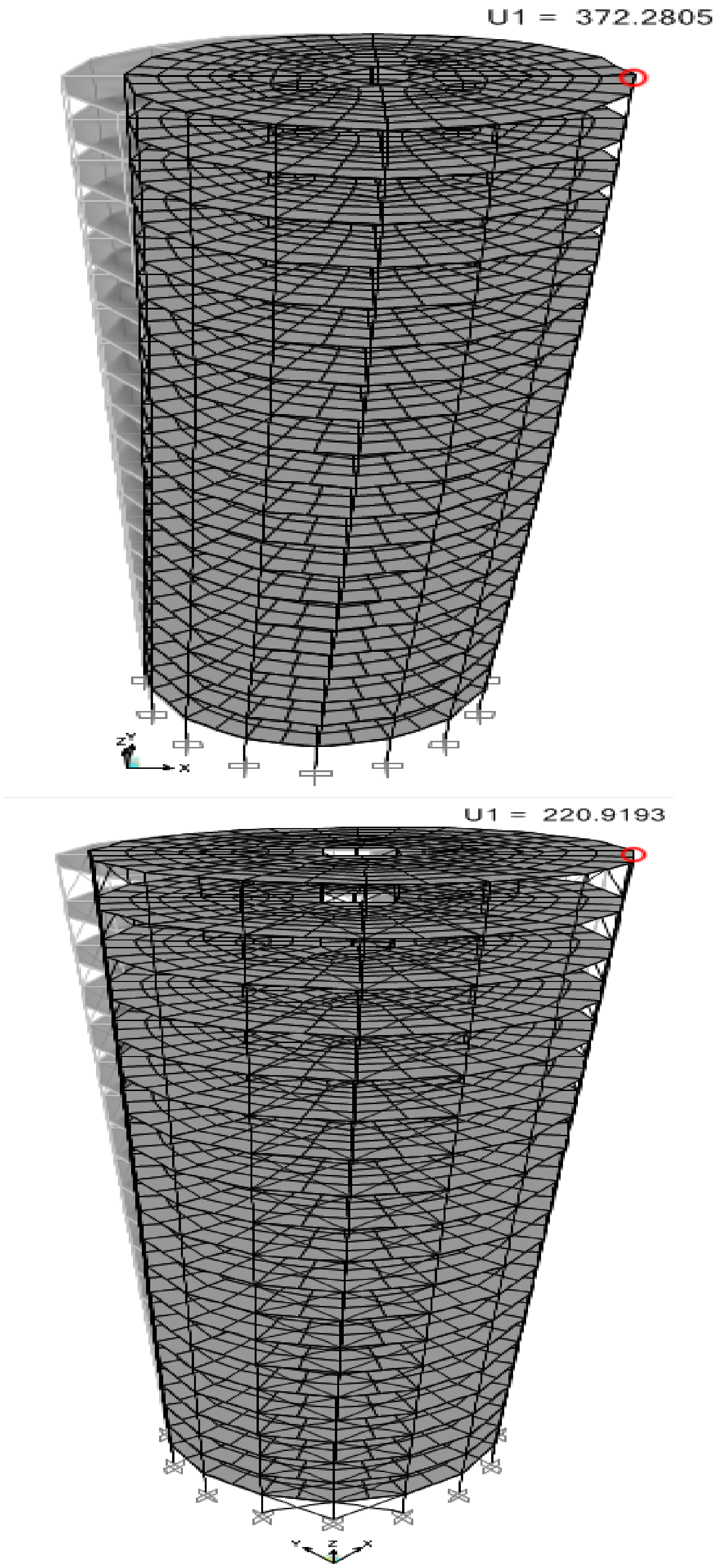


Figure 4.24 The bracing performance for 20 story building under seismic load, and showing the reduced in maximum displacement.

Table 4.19 Displacement result of bracing on a 10 story circular building

Height (m)	Without	X-out	X-mid	X-cor	L/5-out	L/5-mid	L/5-cor
0	0	0	0	0	0	0	0
4	16.33	5.32	6.13	7.58	8.11	8.23	9.79
8	38.23	12.15	13.26	17.49	17.92	17.68	22.31
12	59.43	19.72	20.55	28.09	27.85	26.90	35.03
16	79.17	27.70	27.76	38.76	37.64	35.68	47.37
20	97.21	35.87	34.71	49.03	47.11	43.87	58.95
24	113.30	44.04	41.27	58.66	56.07	51.34	69.54
28	127.15	52.05	47.31	67.44	64.32	57.94	78.94
32	138.47	59.73	52.69	75.20	71.67	63.51	86.89
36	146.97	66.90	57.30	81.77	77.88	67.87	93.13
40	152.40	73.31	60.90	86.96	82.65	70.79	97.40

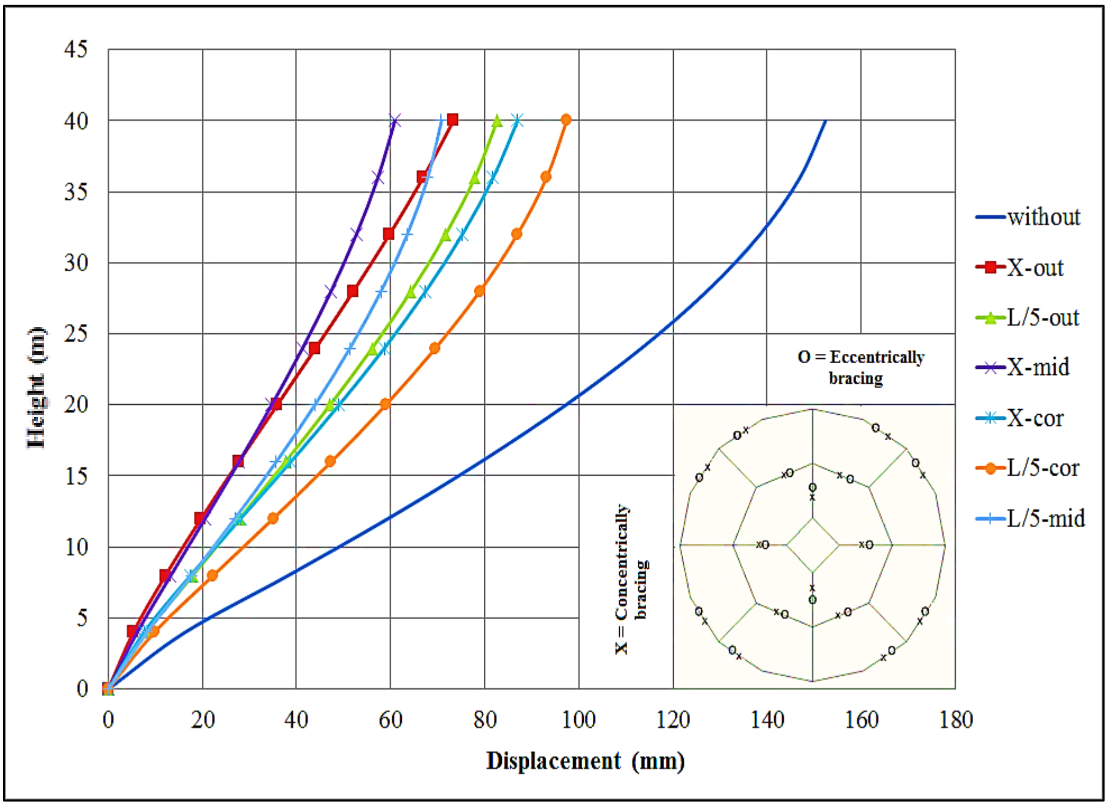


Figure 4.25 The effect of bracing on the displacement for 20 story circular plan

Table 4.20 Displacement result of bracing on a 20 story circular building

Height (m)	Without	X-out	X-mid	X-cor	L/5-out	L/5-mid	L/5-cor
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	18.17	5.48	6.40	8.01	8.33	8.56	10.28
8	43.36	13.13	14.32	19.13	19.11	18.93	24.12
12	69.11	22.25	22.99	31.84	30.83	29.77	39.16
16	94.68	32.52	32.19	45.60	43.26	40.91	54.92
20	119.80	43.65	41.74	59.93	56.19	52.17	71.04
24	144.30	55.44	51.50	74.52	69.49	63.45	87.21
28	168.10	67.70	61.38	89.09	83.02	74.69	103.23
32	191.13	80.29	71.30	103.49	96.65	85.80	118.94
36	213.30	93.07	81.21	117.57	110.26	96.76	134.21
40	234.53	105.93	91.03	131.25	123.74	107.49	148.97
44	254.70	118.75	100.70	144.45	137.00	117.94	163.14
48	273.72	131.44	110.18	157.12	149.95	128.03	176.67
52	291.47	143.92	119.39	169.20	162.52	137.72	189.48
56	307.87	156.11	128.30	180.63	174.61	146.94	201.51
60	322.83	167.97	136.85	191.34	186.15	155.64	212.68
64	336.25	179.45	145.01	201.28	197.07	163.76	222.89
68	348.01	190.52	152.72	210.37	207.30	171.23	232.05
72	358.00	201.13	159.94	218.59	216.76	177.98	240.08
76	366.07	211.30	166.63	225.93	225.39	183.93	246.92
80	372.28	220.91	172.69	232.40	233.05	188.97	252.55

Table 4.21 Displacement result of bracing on a 30 story circular building

Height (m)	Without	X-out	X-mid	X-cor	L/5-out	L/5-mid	L/5-cor
0	0	0	0	0	0	0	0
4	23.89	5.51	6.34	8.08	8.20	8.46	10.73
8	57.46	13.53	14.45	19.57	19.21	18.96	25.42
12	92.48	23.38	23.61	32.99	31.61	30.27	41.65
16	128.15	34.77	33.64	47.85	45.18	42.23	58.99
20	164.17	47.42	44.36	63.72	59.68	54.69	77.09
24	200.35	61.09	55.62	80.30	74.93	67.54	95.68
28	236.53	75.61	67.31	97.35	90.80	80.67	114.58
32	272.56	90.82	79.35	114.70	107.16	94.02	133.63
36	308.33	106.58	91.67	132.20	123.92	107.54	152.73
40	343.74	122.78	104.22	149.75	140.98	121.20	171.82
44	378.69	139.30	116.96	167.27	158.26	134.94	190.83
48	413.09	156.06	129.85	184.69	175.66	148.76	209.71
52	446.86	172.96	142.85	201.97	193.12	162.62	228.42
56	479.90	189.93	155.94	219.06	210.56	176.48	246.92
60	512.13	206.89	169.09	235.94	227.93	190.32	265.15
64	543.44	223.79	182.25	252.56	245.16	204.09	283.06
68	573.76	240.58	195.39	268.91	262.20	217.77	300.61
72	602.98	257.21	208.48	284.95	279.01	231.31	317.76
76	631.05	273.63	221.49	300.66	295.55	244.69	334.46
80	657.87	289.80	234.38	315.98	311.76	257.86	350.66
84	683.38	305.69	247.13	330.90	327.59	270.79	366.32
88	707.51	321.26	259.70	345.35	343.02	283.45	381.41
92	730.19	336.49	272.06	359.30	357.99	295.80	395.88
96	751.36	351.35	284.18	372.70	372.47	307.80	409.69
100	770.94	365.83	296.02	385.51	386.43	319.40	422.79
104	788.87	379.91	307.56	397.69	399.83	330.56	435.12
108	805.08	393.61	318.76	409.22	412.65	341.22	446.64
112	819.50	406.91	329.60	420.08	424.85	351.36	457.31
116	832.11	419.86	340.06	430.31	436.42	360.92	467.10
120	843.01	432.40	350.09	439.91	447.28	369.85	476.01

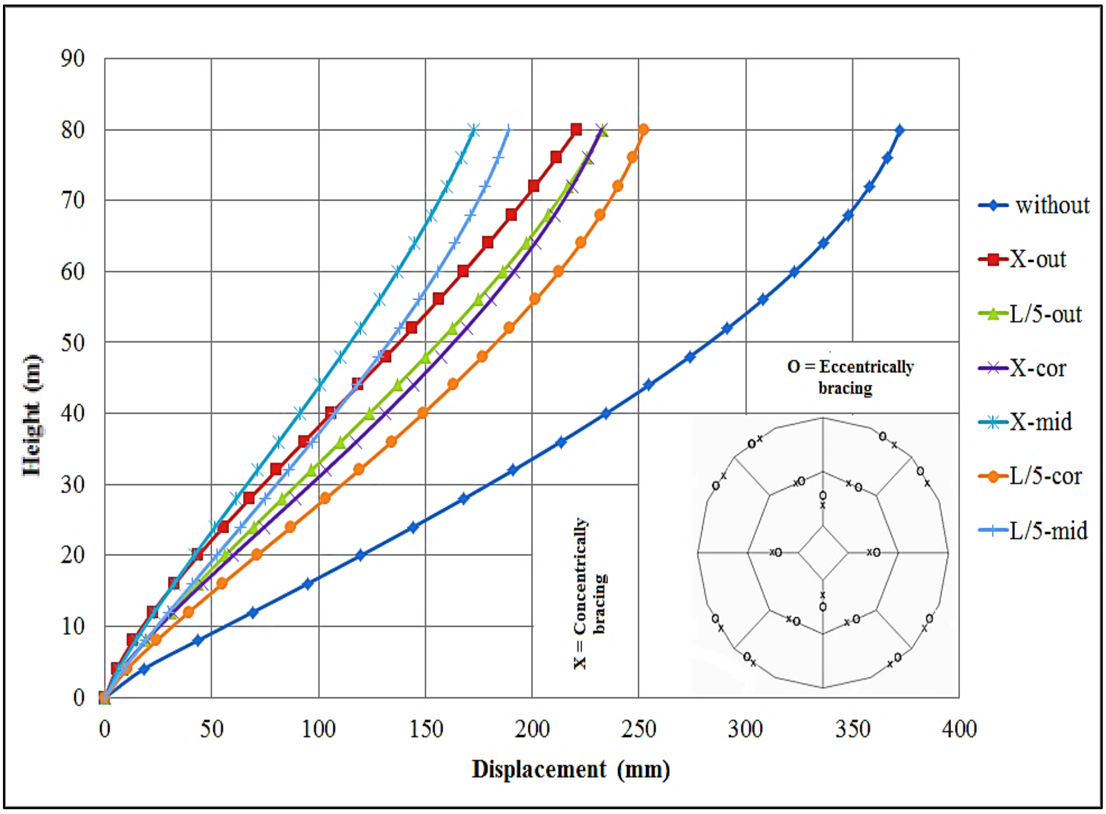


Figure 4.26 The effect of bracing on the displacement for 20 story circular plan

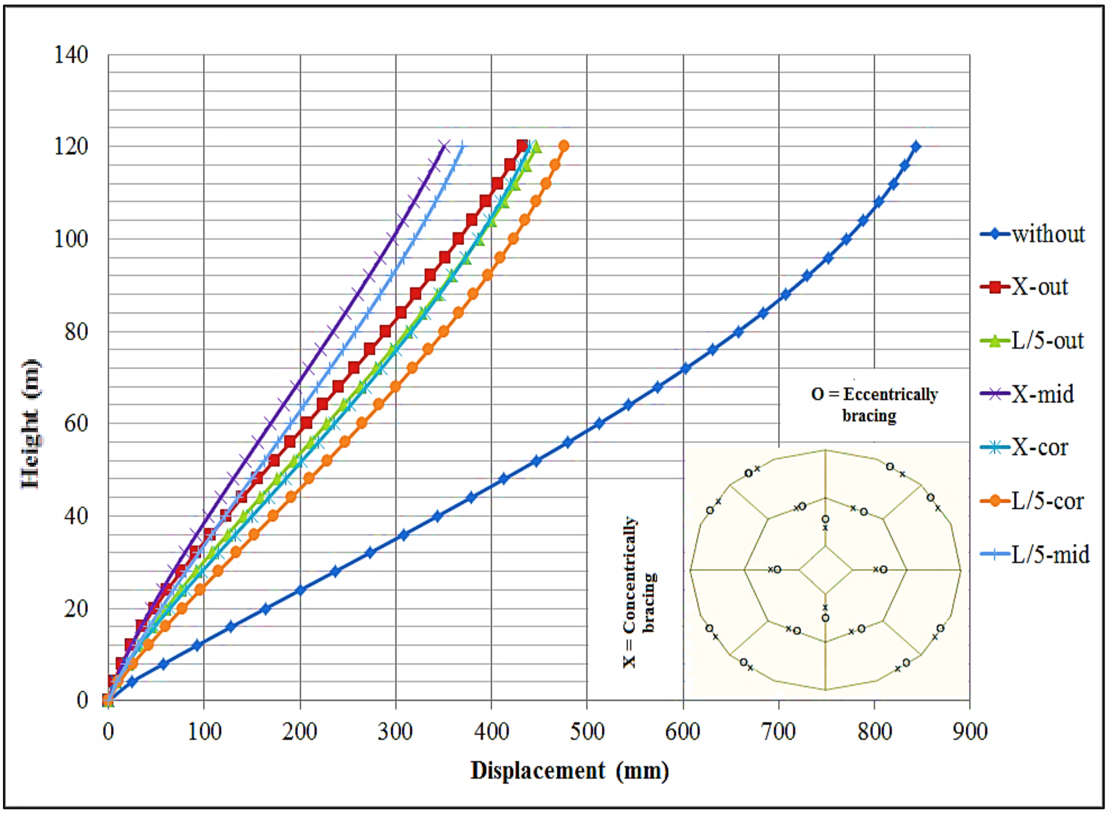


Figure 4.27 The effect of bracing on the displacement for 20 story circular plan

4.3 Discussions

According to the findings obtained from the response spectrum analysis, both the braced frame systems had significantly lower story displacements than the existing frame under the all earthquake ground motions. For rectangular building as an example, (see Figure 4.11), at the 10 story building in the X-axis under the ground motion, the maximum displacement of the existing frame was obtained at 21.2 cm, while the maximum displacement of the inner concentrically x braced frame equal to 7.4 cm and inner eccentrically braced frames (L/5) was equal to 8.6 cm. In the case of the 20 story building in the X-axis (see Figure 4.15), the maximum displacement of the existing frame was achieved as 57.1 cm while the maximum displacement of inner concentrically x braced and inner eccentrically braced frames (L/5) was obtained as 21 cm and 22.8 cm, respectively. In the case of the 30 story building in the X-axis (see Figure 4.19), the maximum displacement of the existing frame was achieved as 136.8 cm while the maximum displacement of inner concentrically (x) braced and inner eccentrically braced frames (L/5) was obtained as 39.1 cm and 41.5 cm, respectively. As a result, it was appeared that the use of the concentrically braced frames was better than the eccentrically braced frame, in the case of using brace frame systems were decreased significantly the maximum displacement of the original frames.

The graph and the results of circular buildings, is showing that the middle location of bracing for each type, it's the best performance of the buildings to decrease the maximum displacement compared with other location of bracing under the seismic load. In 10 story building (see Figure 4.25), the displacement of concentric bracing (x) type decreased by percent equal to 61% in the middle span and 56% in the outer span than structures without bracing, for eccentrically bracing type the displacement decreased by 55% in the middle span and 47% in the outer span than structure without bracing, The lowest effect on the displacement for each type of bracing that appear in the span between the core and the middle span.

In 20 story building (see Figure 4.26), the effect of bracing location appear more clearly than 10 story building, the middle span gives better performance for decreased the displacement than other location for each type of bracing, the concentrically bracing X decreased the displacement with 55% and 50 % for eccentrically bracing compared with without bracing and still the lowest location effect in the span between the core and the middle span.

In 30 story building still have the same performance with the 20 story building (see Figure 4.27), the middle span gives a better result about the location of bracing with decreased the displacement for concentrically bracing 59% and 56% for eccentrically bracing compared with the structure without bracing, the lowest decreased for displacement the eccentrically bracing system in the span between the core and the middle span with percent equal to 43% than building without bracing.

In previous figures was observed, that all structures under retrofitted if exposed to an earthquake, it will dramatically affect, Furthermore, the use of bracing retrofit in the structure reduced in dramatically the value of the maximum displacements of the story. The frame type and the number of stories were also affected on the maximum story displacement; it means in the thirty stories of models the value of the maximum displacement was greater than the other story in the comparison, for rectangular building the results are;

1. Figure 4.11, the maximum displacement of the existing frame without bracing was 21.2 cm of 10 story building in X-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 7.4 cm and EBFs with link length (L/3), (L/5) and (L/10) was 9.5 cm, 8.6 cm and 9.3 cm, respectively.
2. Figure 4.12, the maximum displacement of the existing frame without bracing was 9.7 cm of 10 story building in Y-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 5.8 cm and EBFs with link length (L/3), (L/5) and (L/10) was 6.9 cm, 6.4 cm and 7.5 cm, respectively.
3. Figure 4.13, the maximum displacement of the existing frame without bracing was 21.2 cm of 10 story building in X-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 9.4 cm and EBFs with link length (L/3), (L/5) and (L/10) was 10.8 cm, 10.1 cm and 10.7 cm, respectively.
4. Figure 4.14, the maximum displacement of the existing frame without bracing was 9.7 cm of 10 story building in Y-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 6.8 cm and EBFs with link length (L/3), (L/5) and (L/10) was 7.6 cm, 7.3 cm and 8.2 cm, respectively.

5. Figure 4.15, the maximum displacement of the existing frame without bracing was 57.1 cm of 20 story building in X-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 21 cm and EBFs with link length (L/3), (L/5) and (L/10) was 24.3 cm, 22.8 cm and 26 cm, respectively.
6. Figure 4.16, the maximum displacement of the existing frame without bracing was 24.9 cm of 20 story building in Y-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 17.2 cm and EBFs with link length (L/3), (L/5) and (L/10) was 18.7 cm, 18cm and 20 cm, respectively.
7. Figure 4.17, the maximum displacement of the existing frame without bracing was 57.1 cm of 20 story building in X-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 29.1 cm and EBFs with link length (L/3), (L/5) and (L/10) was 31 cm, 30 cm and 34 cm, respectively.
8. Figure 4.18, the maximum displacement of the existing frame without bracing was 24.9 cm of 20 story building in Y-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 19.8 cm and EBFs with link length (L/3), (L/5) and (L/10) was 20.8 cm, 19.8 cm and 23.6 cm, respectively.
9. Figure 4.19, the maximum displacement of the existing frame without bracing was 136.8 cm of 30 story building in X-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 39.1 cm and EBFs with link length (L/3), (L/5) and (L/10) was 43.4 cm, 41.5 cm and 49.7 cm, respectively.
10. Figure 4.20, the maximum displacement of the existing frame without bracing was 47.3 cm of 30 story building in Y-axis, when subjected to earthquake. While, the maximum value for inner CBFs was 35.5 cm and EBFs with link length (L/3), (L/5) and (L/10) was 37.2 cm, 36.4 cm and 39.9 cm, respectively.
11. Figure 4.21, the maximum displacement of the existing frame without bracing was 136.8 cm of 30 story building in X-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 60.9 cm and EBFs with link length (L/3), (L/5) and (L/10) was 66.2 cm, 63.6 cm and 81.7 cm, respectively.
12. Figure 4.22, the maximum displacement of the existing frame without bracing was 47.3 cm of 30 story building in Y-axis, when subjected to earthquake. While, the maximum value for outer CBFs was 39.4 cm and EBFs with link length (L/3), (L/5) and (L/10) was 40.6 cm, 39.6 cm and 46 cm, respectively.

For circular building the results are;

1. Figure 4.25, the maximum displacement of the existing frame without bracing was 15.2 cm of 10 story building, when subjected to earthquake. While, the maximum value for outer CBFs was 7.3 cm, middle span was 6.1 cm and the span between the core and the middle span was 8.7 cm. Also, the maximum displacement for EBFs with link length (L/5) in the outer span was 8.3cm, middle span was 7.1 cm and for the span between the core and the middle span was 9.7 cm.
2. Figure 4.26, the maximum displacement of the existing frame without bracing was 37.2 cm of 20 story building, when subjected to earthquake. While, the maximum value for outer CBFs was 22.1 cm, middle span was 17.3 cm and the span between the core and the middle span was 23.2 cm. Also, the maximum displacement for EBFs with link length (L/5) in the outer span was 23.3 cm, middle span was 18.9 cm and for the span between the core and the middle span was 25.3 cm.
3. Figure 4.27, the maximum displacement of the existing frame without bracing was 84.3 cm of 30 story building, when subjected to earthquake. While, the maximum value for outer CBFs was 43.2 cm, middle span was 35 cm and the span between the core and the middle span was 44 cm. Also, the maximum displacement for EBFs with link length (L/5) in the outer span was 44.7 cm, middle span was 37 cm and for the span between the core and the middle span was 47.6 cm.

From the results above, the effect of bracing appear clearly in decreasing the maximum displacement. In the rectangular building the concentrically braced frame reduced significantly the maximum displacement, regardless of bracing location and direction of seismic load. In circular building, bracing location gives effect to decrease the maximum displacement irrespective of type of bracing. The ratio of performance increment for rectangular and circular buildings is given in Table 4.22, Table 4.23 and Table 4.24

Table 4.22 Performance increment of rectangular building in X-axis

Number of story	L/3 inner	L/5 inner	L/10 inner	X inner	L/3 outer	L/5 outer	L/10 outer	X outer
10	55%	59.3%	56.5%	65%	48.8%	52.1%	49.5%	55.7%
20	57.5%	60.1%	54.4%	63.2%	45.8%	47.6%	40.4%	49%
30	68.3%	69.7%	63.7%	71.4%	51.6%	53.5%	40.4%	55.5%

Table 4.23 Performance increment of rectangular building in Y-axis

Number of story	L/3 inner	L/5 inner	L/10 inner	X inner	L/3 outer	L/5 outer	L/10 outer	X outer
10	29.7%	34.2%	22.9%	40.4%	22.2%	25.2%	16.1%	29.5%
20	25.1%	27.5%	19.7%	31.1%	16.6%	20.3%	4.9%	20.3%
30	21.4%	22.9%	15.5%	24.9%	14.1%	16.2%	2.7%	16.4%

Table 4.24 Performance increment of circular building

Number of story	X out	X mid	X cor	L/5 out	L/5 mid	L/5 cor
10	51.9%	60%	42.9%	45.8%	53.6%	36.1%
20	40.7%	53.6%	37.6%	37.4%	49.2%	32.2%
30	48.7%	58.5%	47.8%	46.9%	56.1%	43.5%

CHAPTER FIVE

CONCLUSION

5.1 Conclusion of Study

This study achieved the modelling, analysis and design of response spectrum analysis, in high rise steel building. The performance characteristics were evaluated according to the response spectrum analysis. In the area of civil engineering, this study constitutes an example of arising applications of the dynamics analysis. The bracing system elements have been chosen to resist seismic load in this study. SAP2000 version 14 (CSI, 2009), was used to performing the analysis and design. All results obtained from the program were exported to Microsoft Excel (2010) to draw all the graphics required for the study. In comparing the result from dynamic analysis of this study, the following conclusions can be reached:

1. After checking the results, data indicate that the use of the concentrically and eccentrically braced frames reduces the value of maximum displacement significantly. Also, the maximum displacement of the frame with concentrically braced was smaller than other frames, and the increase in the number of story's leads to an increment in the displacements.

2. In general the effect of concentrically bracing on the rectangular buildings is bigger than the effect of eccentrically bracing in increment the performance of buildings, and to decrease the maximum displacement, regardless of, the location of bracing in the building and the direction of lateral loads. The maximum displacement in the long direction is bigger than the displacements in the short direction of the buildings.

3. Eccentrically braced frame with link length equal to L/5 for the beam length L, and to give the best performance to reduce the displacement from that's having a

link length equal to L/3 and L/5 in all rectangular building cases. Inspire of, the location of bracing in the building and the direction of lateral loads.

4. The results of the circular buildings show that the performance of the buildings be affected by the location of bracing, more than the bracing types, where was the middle location best performance against the displacement and more than the other bracing location like the outer span or the span between the core and the middle span.

5.2 Future studies.

This study employed a number of steel frames and plan with a limited hypothesis of ground motion. So, the proposed results which had been calculated in this study needs to be validated by further case studies such as;

1. Future studies should be done in finding the optimum braced frame structure have the advantage of all types. In addition, for a cost-efficient design.

2. Analysis and design can be extended to include different story heights for large towers that require changing in system of structure.

3. The accuracy of lateral load patterns that used in predicting seismic demands should also be investigated

4. Develop the available methods of analysis and design buildings or find new methods for that, also can improve the code for the design and analysis seismic load.

5. Apply a deep study on this subject by using a large number of frames with different earthquake records and testing new forms of structures and new type of bracing.

6. Search the effect of use bracing type in this study on the deflection of beam and column buckling and make comparison between it.

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