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BEHAVIOUR OF FRAME STRUCTURES WITH CONCENTRIC DIAGONAL BRACINGS UNDER LATERAL LOADING

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Behaviour of Frame Structures with Concentric Diagonal Bracings under Lateral Loading

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Supervisor
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Guler MUHYADDIN

ABSTRACT

Behaviour of Frame Structures with Concentric Diagonal Bracings under Lateral Loading

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M.Sc. in Civil Engineering

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In this study, nonlinear static analysis was performed to compare the structural response of different type of moment resisting frame buildings with and without concentric diagonal braces (CBs) subjected to lateral loading. For the purpose of this study, moment resisting frame buildings with 4, 8, 12, and 16 stories were taken into consideration. The buildings had the same plan, which included three bays on each direction were selected as a case study. The existing steel frames were designed according to two different cases. They were referred to as a) flexible moment-resisting frames and b) rigid moment-resisting frames. Then, concentric braces were inserted in the middle bay of the existing frames. For the braced frame structures, diagonal type configuration was used. A total of 16 different steel frame structures were under investigation. Capacity curve, total base shear, interstorey drift index, and global damage index were evaluated for each frame system. Depending upon the design properties of the frames, the results exhibited a substantial improvement in the earthquake performance of the frames with the incorporation of diagonal type braces.

Keywords: Diagonal brace, Earthquake, Frame, Nonlinear analysis, Structural response

ÖZET

MERKEZİ DİYAGONAL ÇAPRAZ ELEMANLI ÇERÇEVE YAPILARIN YANAL YÜKLER ALTINDAKİ DAVRANIŞI

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84 Sayfa

Bu çalışmada, farklı özelliklere sahip moment aktarabilen çerçeve sistemli yapılar ile merkezi diyagonal çelik çapraz ilaveli yapıların yanal yükler altındaki yapısal tepkileri lineer olmayan statik analiz kullanılarak karşılaştırmalı olarak incelenmiştir. Bu amaçla, 4, 8, 12 ve 16 katlı moment aktarabilen çerçeve sistemli yapılar araştırmada kullanılmıştır. Yapılar aynı kat planına sahip olup, her iki yönde de üç açıklıktan oluşmaktadır. Mevcut çelik yapılar iki farklı durum için tasarlanmıştır. Bunlar esnek moment aktarabilen çerçeveli ve rijit moment aktarabilen çerçeveli yapılar olarak sıralanmaktadır. Merkezi diyagonal çelik çaprazlı yapıların oluşturulmasında, merkezi çaprazlar her bir mevcut yapının orta açıklığına yerleştirilmiştir. Böylece, araştırmada 16 farklı çelik çerçeveli yapı incelenmiştir. Her bir çerçeve sistemi, kapasite eğrisi, toplam taban kesme kuvveti, göreli kat ötelemesi indeksi ve hasar indeksi gibi parametreler kullanılarak değerlendirilmiştir. Yapıların tasarım özelliklerine bağlı olarak, diyagonal çaprazların sisteme ilavesinin yapıların deprem performanslarında önemli iyileşmelere sebep olduğunu göstermiştir.

Anahtar kelimeler: Diyagonal çelik çapraz, Deprem, Çerçeve, Lineer olmayan analiz, Yapısal tepki

To my beloved parents,

brother and sisters

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Finally, and the most thoughtfully difficult, but the most emotionally vital, I say. I have to pronounce the last but the greatest thanks. This is turned to my siblings for their sacrifices and for the love and encouragements proved to me during my studies.

TABLE OF CONTENTS

CONTENTS Page

LIST OF TABLES

TABLES Page

LIST OF FIGURES

LIST OF FIGURES Page

LIST OF SYMBOLS/ ABBREVIATIONS

AISC……………………..…. American Institute of Steel Construction

BF-BF………………………. BF-BF Braced frames in both horizontal directions

BFs…………………….……. Braced frames

BRBFs……………...……. Buckling restrained braced frames

CBFs……………….….…. Concentrically braced frames

CBs…………………………. Concentric diagonally braces

CP…………………………. Collapse prevention

FEMA…………………..…. The Federal Emergency Management Agency

IQ……………………..……. Immediate occupancy

LLRS……………………..…. Lateral resisting structure system

LRFD……………………..…. Load and Resistance Factor Design

LS…………………….……... Life safety

MDOF……………………...…. Multi degree of freedom

MRFs……………………….. Moment-resisting frames

NEHRP……………………...National Earthquake Hazards Reduction Program

NLLink……………………..…. Nonlinear link

NSP……………………….…….Pushover analysis

OCBFs……………...………. Ordinary concentric braced frame

PBPD………………….……. Performance-based based plastic design

SCBFs………………………. Special concentric braced frames

SDOF………………………...… Single degree of freedom

SMA……………………..…. Shape memory alloy

T……………………………. Fundamental period of vibration

UB-BF………………..…….Unbraced frames in one horizontal direction &braced frames in the other direction

UBC……………………...…. Uniform Building Code

UFs…………………………. Unbraced frames

UF-UF…………………...…. Unbraced frames in two horizontal directions

CHAPTER 1

INTRODUCTION

1.1 General

Structures in seismically spry zones are disposed to extreme destruction and also breakdown through earthquakes as a result of vast lateral warps. The flexibility of steel moment-resisting frames perhaps the consequence in an excessive lateral drift caused non-structural destruction nether tough ground motion (Asgarian and Moradi, 2011).

Steel frame structural systems have been extensively used in the United States for mid- to high-rise structures. A great majority of these systems constructed before 1994 comprised of steel moment resisting frames to offer lateral resistance throughout an earthquake (McCormick et al., 2007). The existence of the 1994 Northridge earthquake and 1995 Hyogoken–Nanbu (Kobe) earthquake led to unpredicted destruction to many of these systems due to fracture of welded beam to column connections causing excessively large lateral displacements (Nakashima et al., 1998). To prevent upcoming difficulties related with geometric nonlinearities and weak fracture of the beam–column connection in steel moment-resisting frames, investigation in the United States has dedicated on understanding the nonlinear and brittle performance of these steel frame structures. Important efforts were assumed to develop different connection geometries and configurations to mitigate these problems (Nakashima et al., 2000). Numerous causes for the exhausted behaviour of ordinary steel braced frames in the latest earthquakes be can be well known as limited ductility and low energy dissipation capacity because of braces buckling, defeat of connections and unsymmetrical behaviour of the braces in tension and compression. Observant the defects for ordinary concentrically braced frames, seismic design needs for braced frames were altered and the idea of special concentric braced frames was established (Sabelli et al., 2003). In spite of the improved behaviour of the special concentric braced frames above ordinary braced frames, investigation acts have been realized to investigation for additionally sacrifice and best behaviour concentric braced frames (Sabol, 2004).

As a further result of the Northridge and Hyogoken–Nanbu earthquakes, the structural engineering community has focussed on a further performance-based seismic design approach in order to prevent a recurrence of similar damage and economic losses in future earthquakes. Performance-based design provides engineers with the means to design and analyse building structures such that they have a predictable and reliable performance in the event of an earthquake (Hamburger et al., 2003). The Federal Emergency Management Agency (FEMA)/SAC steel project resulted in the progress of guidelines for the design of steel moment frames, which limited interstory displacement to given performance levels (FEMA 2000). However, such a comprehensive study has yet to be completed for concentrically braced frame systems. There exists a need to investigate improved design and retrofit measures for current concentrically braced frame systems to ensure that these systems fit in accordance with performance-based design parameters. One means of improving the performance of concentrically braced frame systems in terms of limiting interstory drift levels is through the use of innovative materials in the bracing system (McCormick et al., 2007).

Concentrically braced frames are those braced frames in whatever the center lines of members which match a joint intersect at a point to create a erect truss system that defies lateral loads.

Braced frames were developed in the 1960s and 1970s, along with the promulgation of more detailed seismic regulations. Braced framing systems proved popular in regions of high seismicity because materials savings could be achieved with respect to moment-resisting frames and control of frame drift due to high earthquake-induced inertial forces could be efficiently realized (Shafei et al., 2006).

Through an extreme earthquake, bracing members in a concentric braced frame are exposed to huge deformations in cyclic tension and compression into the post-buckling range. As a consequence it reserved cyclical rotations happened at plastic hinges in much the equivalent way as they do in beams and columns in moment frames. Actuality, braces in a typical concentric braced frame can be projected to yield and buckle at temperate story drifts of about 0.3 % to 0.5 % (Shafei et al., 2006).

In an extreme earthquake, the braces might suffer post buckling axial deformations ten to twenty times their yield deformation. To endure such enormous cyclic deformation's early flop, the bracing members and their connections should be appropriately comprehensive (Shafei et al., 2006).

The concentric braced frame (CBF) is a lateral force-resisting system that is characterized by high elastic stiffness. High stiffness is achieved by the introduction of diagonal bracing members that resists lateral forces on the structural frame by evolving internal axial actions and proportionately tiny flexural actions. Diagonal bracing members and their connections to the framing system form the core units of a CBF (Shafei et al., 2006). Concentrically braced frames (CBFs) are generally considered less ductile seismic resistant structures than other systems due to the brace buckling or fracture when subjected to large cyclic displacements. Nevertheless, it has been estimated that CBFs comprise about 40% of the newly built commercial constructions in the last decade in California (Uriz, 2005). This is attributed to simpler design and high efficiency of CBFs compared to other systems such as moment frames, especially after the 1994 Northridge earthquake. However, recent analytical studies have shown that CBFs designed by conventional elastic design method suffered severe damage or even collapse under design level ground motions (Sabelli, 2000). In addition, the confidence level (FEMA, 2000) to achieve collapse prevention performance objectives for typical CBFs can be extremely and unacceptably low when compared with special moment frames (Mahin and Uriz, 2004).

The design of braced frames in areas of intense seismicity is achieved conforming to the AISC-Seismic Provisions. A concise debate of the seismic provisions of different versions of this handbill, containing the major attributes of braced frame design, is as follows:

1- The difference between the two ordinary concentric braced frame (OCBF) and special concentric braced frame (SCBF) is in particularization of the connections, and a number of prescriptive demands for SCBF planned to empower them to reply to seismic forces with larger ductility.

2- Concentrically braced frames are predictable to sustain an inelastic response through huge earthquakes. Especially intended diagonal braces in these frames can assist plastic deformations and spend hysteretic energy in a steady manner during following cycles of buckling in compression and yielding in tension. The favoured design approach is, hence to certify that plastic deformations only happen in the braces, leaving the columns, beams, and connections not harmed, so permitting the building to endure hardy earthquakes without gravity-load resistance.

3- Braced frames with single diagonals are further authorized by AISC-Seismic Provisions. Nevertheless there is a burdensome penalty as the braces should be designed to challenge 100% of the seismic force in compression, unless multiple single-diagonal braces are stated for a granted frame line. Beams and columns in braced frames ought to be designed to stay elastic as braces have attained their highest tension or compression capacity (1.1Ry times the nominal strength where Ry is the ratio of the predictable yield strength to the lowest specified yield strength) to prevent response in all parts but the braces.

4- Compactness necessities for braces are the equivalent for OCBFs and SCBFs.

5- The plastic hinge that creates at mid-span of a buckled brace possibly will elaborate huge plastic rotations that could be leading to local buckling and quick mislay of compressive capacity and energy dissipation characteristic through periodic cycles of inelastic deformations. Locally buckled braces can as well suffer low-cycle fatigue and fracture later a few cycles of extreme inelastic deformations, especially whenever braces are cold-formed rectangular hollow sections. For these cause's braces in SCBFs have to satiate the width-to-thickness ratio limits for compacted sections. For OCBFs, braces can be compact or non-compact, but not slender (Shafei et al., 2006)

6- SCBFs are intended to exhibit improved post-buckling capacity over OCBF. In an SCBF, the braces still buckle in compression but are designed to maintain a greater post-buckling capacity and thus continue to contribute to overall frame capacity. Additional special detailing of the connections that prevent local connection buckling failures and member failures even when there is overall buckling of the compression brace are also required in SCBFs. The required brace and connection detailing can be quite restrictive, resulting in braces fabricated from large rolled shapes.

7- The SCBF ductility comes from axial inelasticity of the braces in tension and compression. Compression buckling, however, results in degradation of the brace stiffness and strength. Eventual plastic hinge formation can lead to brace fracture. Buckling of the braces, thus limits the performance of the CBF system. The 1994 UBC acknowledged these findings: The code recognized among OCBFs and SCBFs during design forces and particularization needs; OCBFs were intended for big base shears with the suspense of low ductility requests; SCBFs had been minor needed base-shear capacity and were dealt with similarly ductile systems that had to provide cyclical expeditions into the post-buckling compass. The difference between OCBFs and SCBFs created the 1994 UBC is a little less comprehensible. The force request for OCBFs corps changeless, but in the LRFD provisions some ductile detailing needs were added. The necessities for SCBFs stayed changeless. The distinctions between the ordinary and special types are hence limited to: slenderness limits, OCBF braces's capacity reduction, brace compactness and stitch necessities' permissible form's column necessities and the waiving of specific needs for one- and two-story OCBFs.

The 2005 AISC seismic provisions (AISC, 2005) create a further rational differentiation among the two techniques. OCBFs are anticipated to comprise a higher elastic force capacity (due to the higher design base shear and the decreed deduction in calculated brace capacity) and to provide cyclical buckling of braces in the connection design. Expeditions into the tensile yielding range necessity not be thought over SCBFs are predictable to accomplish trilinear hysteretic behaviour by obliging cyclical brace buckling in addition to resisting forces equivalent to the yielding capacity of the braces. The force level equivalent to the yield mechanism discovers the highest forces that element of the system, like the connections, are needful of oppose. As capacity design is used for SCBFs, AISC Seismic 2005 applies an overstrength factor (Ry) to account for predictable yield strength and strain hardening. SCBF necessities for braces are intentional to avoid unacceptable modes of brace behaviour. Critical investigations on bracing systems designed in austere conferring with earlier code necessities prophesied to brace omissions lacking the improvement of consequential energy dissipation (Hassan and Goel 1991; Tang and Goel 1989). Brace failures happened most often at plastic hinges (concentrated areas of curvature and inelastic strain susceptible to local buckling due to lack of compactness); braces are intentional to avoid unacceptable modes of brace behaviour. Analytical investigations on bracing systems designed in austere conferring with earlier code necessities prophesied to brace failures lacking the improvement of significant energy dissipation (Hassan and Goel 1991; Tang and Goel 1989). Brace failures happened most often at plastic hinges (concentrated areas of curvature and inelastic strain susceptible to local buckling due to lack of compactness); plastic hinges in buckled braces occur at the ends of a brace and at the brace midspan. Critical models of bracing systems that were formed to certify unchangeable ductile behaviour displayed full and unchangeable hysteresis without rupture when exposed to the equivalent ground motion records as the prior concentrically braced frame designs. Equivalent results were discovered in full-scale tests by Mahin and Yang (2005), Wallace and Krawinkler (1985), Tang and Goel (1989), and Uriz (2005).

1.2 Objective and scope

The principal purpose of this study was to compare the seismic performance of flexible and rigid moment resisting framed buildings and the same moment resisting framed buildings added with diagonally CBs subjected to lateral loading. Exploring the effect of improved nonlinear behaviour of CBs on performance and response of the building structures was monitored. Moreover, the effect of buckling of the CBs on the response and behaviour of the building structures when loaded with a large force were studied and finally judging on which rehabilitation strategy that would improve the seismic performance of the existing buildings were performed. The case study structures considered in this investigation were eight steel moment-resisting frame (MRF) buildings having four, eight, twelve, and sixteen stories height. The buildings have the same plan, which consists of three bays on each direction. The buildings were assumed to have a uniform mass distribution over their height and a non-uniform lateral stiffness distribution. The existing structures were first designed by Santa-Ana and Miranda (2000). They were categorized as flexible and rigid frames. Then, diagonal braces were inserted into each frame system. Totally, 16 different frame systems were studied. Performance characteristics in terms of capacity curve, local and global index were evaluated and discussed comparatively.

1.3 Outline of the thesis

The major objective of this thesis is to provide a description through nonlinear static analysis of the various frame systems and assess their efficiency.

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review and background: This chapter traces the historical background on practical application and previous studies on moment resisting frames, braced frames, and their different types. Also, the negative consequences during severe earthquakes of frame system discussed.

Chapter 3-Methodology: In this chapter, type of the frame systems and strategies that was used in this study is presented and the type of analysis procedures that has been carried out with the assumptions for modeling is discussed. Also, it describes and discusses the analytical model used in this research.

Chapter 4-Result and discussion: This chapter presents and discusses the sequels attained from nonlinear static analysis for assessing the structural performance of each frame system considered in this study in terms of capacity curves, interstorey drift index, and etc.

Chapter 5-Conclusion: General conclusions are drawn regarding the overall results of the study.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

2.1 Moment resisting frames

Steel moment frames have been in usage for further than one hundred years, dating to the earlier use of structural steel in construction. Steel building construction of the frame transporting the vertical duties started with the Home Insurance Building in Chicago, a 10-story building built in 1884 with a height of 138 ft., frequently credicted with existence the first skyscraper (Fig. 2.1). These and other elevated buildings in Chicago credited with a complete generation of elevated structures built with load bearing steel frames encouraging concrete floors and non-load bearing; unreinforced masonry in fills walls at their circumferences conceiving in these early structures typically used “H” shapes originated up from plates, and “L” and “Z” sections.

Figure 2.1 The Home Insurance Building – Chicago, IL, 1885, an early skyscraper. (Ronald et al., 2009)

This fundamental construction method stayed trendy for high-rise construction during the 1930s, although by the early 1900s, rolled “H” shape sections started to observe enlarging usage in place of the built-up sections, in specific for slighter framing. It varied very lofty structures, involving New York’s Empire State Building, for several years the worldwide highest buildings are of this formation type.

Next World War II, it got to be wasteful to build perimeter walls absent of infill unreinforced masonry, especially for tall buildings, and more modernistic glass and aluminium curtain wall systems were taken by adoption as a portion of the unused modernist architectural form. The bigger windows possible with these unused curtain wall systems made hefty gusseted framing connections undesirable, and designers started to design connections lacking of gussets, applying angels or split tees to combine top and bottom beam flanges to columns.

In the 1950s, as welding was established into building construction, the angles and plit tees were substituted by flange plates that were shopped welded to the column flanges, and then riveted to the beam flanges. By the 1960s, riveting had come to be uneconomical and was substituted by high-strength bolting. At last in the early 1970s, engineers started to use the connection type familiar nowadays as the welded unreinforced flange - bolted web, containing field-welded, accomplished joint penetration groove welds to join beam flanges to columns, with shop-welded, field-bolted shear plates combining beam webs to columns. In the 1960s and 1970s, Professor Egor Popov at the University of California at Berkeley and further investigators started to carry out cyclic laboratory testing of steel moment framing and observed that a number of controlled on the proportioning and particularization of these structures was required to attain superior inelastic behaviour in hardy earthquakes. Little by little during the 1970s and 1980s, the building codes started to accept these investigator's suggestions and request particular design, organization and detailing of steel moment frames used for seismic reluctance in places of high seismic hazard. Frames complying with these design principles were first indicated as Ductile Moment Resisting Space Frames, and subsequently lastly in the 1988 Uniform Building Code, as Special Moment-Resisting Space Frames, which were allocated the highest response modification factor, Rw. The term “special” was adopted, both because Special principles are needful in designing, and Special behaviour during extreme earthquakes. At the beginning, the requirement for MRF was to provide connections such that accomplished of progression of the strength of the connected members. However, later requirements introduced that

(a) Weak beam/ strong column ratio,

(b) Balance shears strength in the panel zones, and

(c) Section compactness.

Must be provided, most of the steel structures constructed in 1960 to 1970 in western US were moment resisting frames and provided moment resisting connection and great distribution of lateral force and redundancy. However, engineers by 1980's begun to decrease the redundancy and to provide an economic meaning for the building structures through decreasing the moment resisting frame bays. In the aftermath of Northridge earthquake in Los angles, the brittle fracture of many modern special moment resisting frames had surprised the engineers and accelerated the research programs toward developing more robust moment resisting frames (Ronald et al., 2009).

Even so, by the 1980s, designers began to economize their plans and decrease costly field welding by applying fewer bays of moment-resisting framing that occupied heavier beams and columns, consequent in fewer unnecessary structures with extra concentrated lateral force resistance. In highest occasions, a number of elevated structures were supplied with individual a single bay of moment-resisting framing on apiece side of the construction. Succeeding the 1994 Northridge earthquake in the Los Angeles zone. Engineers were amazed to find that some of new special moment-resisting frame structures had practiced brittle rupturing of their welded beam-to-column connections. Equivalent destruction happened one year later, in the 1995 Kobe earthquake in Japan. Succeeding these findings a syndicate of qualified organizations and investigators noted as the SAC Joint Venture engrossed in a federally supported, multi-year program of investigation and progress to discover the reasons of this unforeseen behaviour and to improve suggestions for extra strong moment-resisting frame construction. The SAC investigation behaved at a cost of $12 million over eight years, ensued in the basis for the present design provisions for moment-resisting frames included in AISC 341, AISC 358, and AWS D1. 8. (Ronald et al., 2009).

2.2 Braced frames

Steel moment-resisting frames are disposed to large lateral displacements through critical earthquake ground movements, and desire special care to border destruction to non-structural elements as well as to prevent difficulties linked with P-Δ effects and brittle or ductile fracture of the beam to column connections (FEMA, 2000). As a result, engineers in the US have more and more sour to concentrically brace steel frames as a cost-effective means for resisting earthquake loads. nevertheless destruction to concentrically braced frames in former earthquakes, for example the 1985 Mexico (Osteraas, 1989), 1989 Loma Prieta (Kim, 1992), 1994 Northridge (Tremblay, 1995; Krawinkler, 1996), and 1995 Hyogo-ken Nanbu (AIJ/Kinki

Branch Steel Committee, 1995; Hisatoku, 1995; Tremblay, 1996) earthquakes, shows that buckling of the diagonal members and miserable detailing of the connections (e.g. column to base, brace to beam, brace to column, beam to column) may erode seismic performance as a whole (Broderick et al., 1994; Elnashai et al., 1995; Nakashima et al., 1998; Watanabe et al., 1998b; Naeim et al., 2000). Damage practiced during previous earthquakes worldwide shows that steel multi-storey building structures generally display an sufficient seismic response (e.g. Di Sarno and Elnashai, 2009). This is because of the favourable mass-to-stiffness ratio of base metal, and the increased energy absorption of structural ductile systems occupied. Nevertheless, proportionately the latest earthquakes, e.g. those in the 1994 Northridge (California), 1995 Kobe (Japan) and 1999 Chi-Chi (Taiwan), have indicated that poor detailing of connections (e.g. beam-to-column, brace-to-beam, brace-to-column, and column-to-base) and buckling of diagonal braces can excavate the seismic performance of the structure as whole (Broderick et al., 1994; Elnashai et al., 1995; Nakashima et al., 1998; Watanabe et al., 1998; Naeim et al., 2000).

Figure 2.2 shows the delivery of destruction level and the damage to structural members and connections with regard to structural sort as examined in the repercussion of the 1995 Hyogoken-Nanbu (Kobe) earthquake (Youssef et al., 1995). Disabled buildings are categorized as having unbraced (UFs) or braced (BFs) frames. Hence, taking into consideration the two main framing orientations of a building, the examined structures contain the following designations: UF-UF (unbraced frames in two horizontal directions), UB-BF (unbraced frames in one horizontal direction and braced frames in the other direction), and BFBF (braced frames in both horizontal directions) (Youssef et al., 1995). Beams contained virtually solely of wide-flange sections, also rolled or built-up. For columns, wide-flange (H) sections were used most widely; square-tube (S) sections were as well employed in some structural systems. Taking into consideration the 988 damaged steel structures,

(a) 432 (43.7%) are UF-UF,

(b) 134 (13.6%) are UF-BF,

(c) 34 (3.4%) are BF-BF, and

(d) 388 (39.3%) having unidentified framing systems.

This information shows that the mainstream of smashed buildings had unbraced moment resisting frames (MRFs) as earthquake resistant system (Youssef et al., 1995).

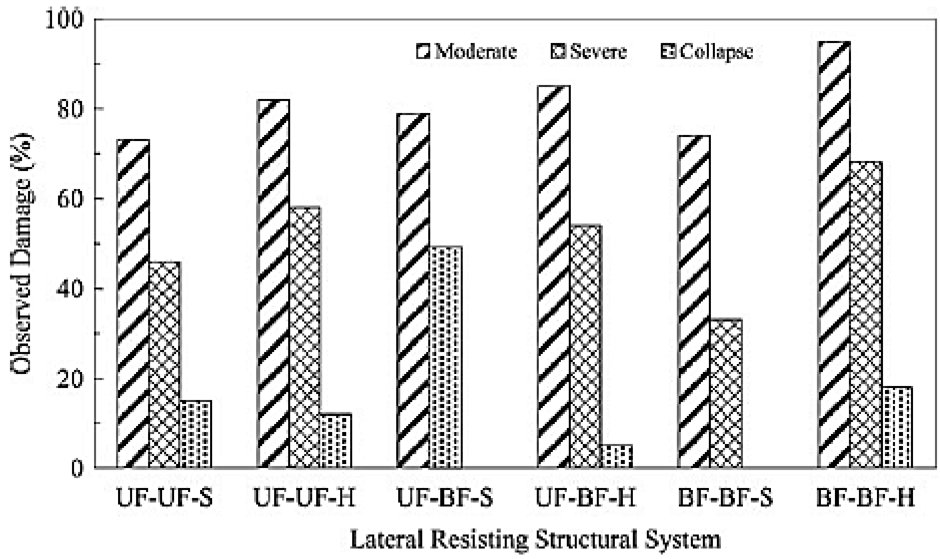


Figure 2.2. Distribution of damage level with regard to structural type (Di Sarno and Elnashai, 2009)

Location of the damage, which is beams, columns, braces, beam to column connection and column bases, with the type of frame is shown in Figure 2.3.

Following observations made from the composed data are as follows (FEMA 355E, 2000):

(a) Columns in UFs sustained the most destruction comparative to other frame elements (in terms of the number of buildings), although braces in BFs were the most often disabled structural element,

(b) Damage to beam-to-column connections and column bases was also important in UFs,

(c) Damage to beam-to-column connections was most important for UFs hiring hollow section (square-tube) columns; and

(d) Damage to columns was most substantial for UFs with wide flange members. The discussion of the overhead observed data is representative of usual structural response of steel buildings disabled by moderate-to-severe earthquake ground motions (FEMA 355E, 2000).

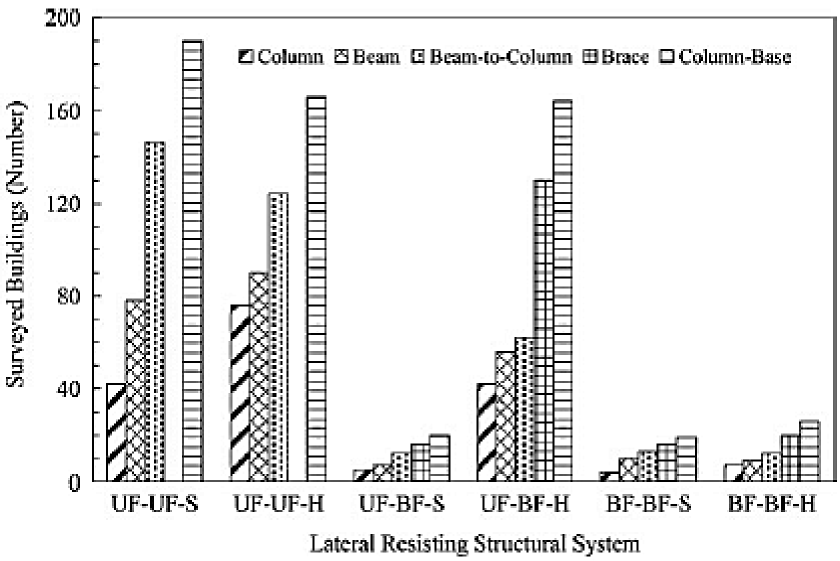


Figure 2.3 Damage to structural members and connections with regard to structural type (Di Sarno and Elnashai, 2009)

The incidence of buckling, frequently in the plastic range in multi-storey buildings, deteriorates since the capacity of the structure and may leads to unexpected vicissitudes in the dynamic characteristics of the lateral resisting structure system (LLRS)(Di Sarno and Elnashai, 2009).

Figure 2.4 shows the brittle fracture for beam to column and braces to column connections that have resulted in decreasing the performance and energy dissipation capacity under earthquake excitation. As a consequence beam-to-column connections and braces may be negligent in ductile MRFs or concentrically braced frames (CBFs) if they are not sufficiently capacity designed (Bruneau et al., 1998; Nakashima et al.,2000; Tremblay, 2002; Broderick et al.,2005).

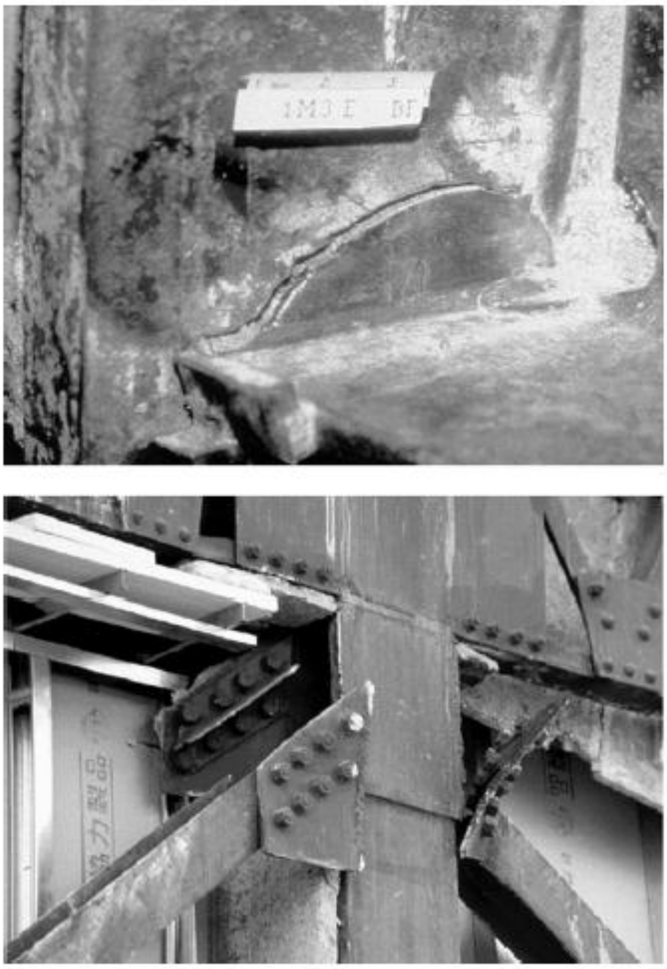


Figure 2.4 Fracture in beam-to-column connections in the Northridge earthquake (*top*) and web tear-out in bolted brace-to-column connections during the 1995 Kobe earthquake (*bottom*) (Di Sarno and Elnashai, 2009)

In retort to various practical and economic matters engineers are revolving to the use of braced steel frames. Whenever hysteretic dampers are utilized, it is anticipated that the braces can intensification 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 on structural and nonstructural component are conceived to be smaller than their capacity when global modification is applied as shown in Figure 2.5 (Bozorgnia and Bertero, 2004; Di Sarno and Elnashai, 2009).

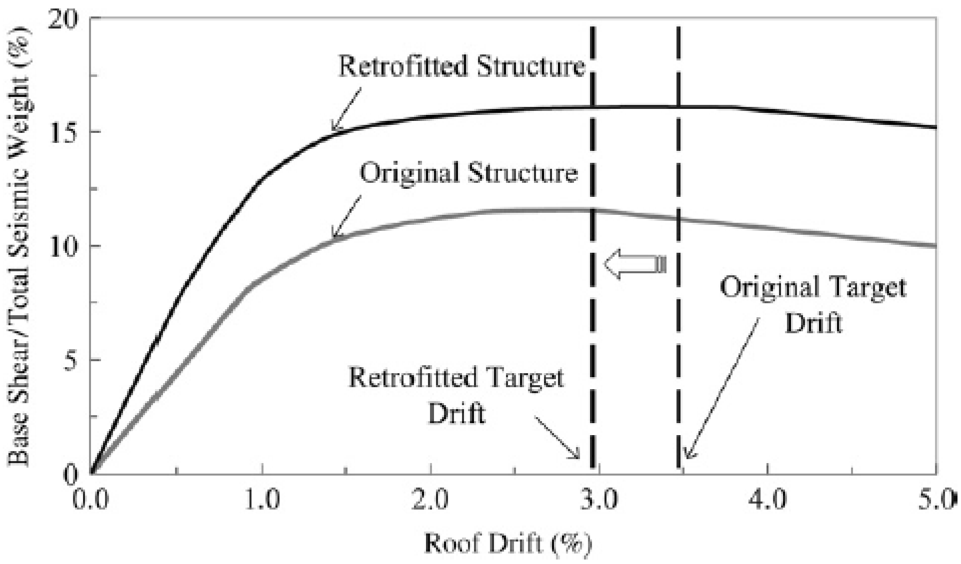


Figure 2.5 Characteristics of global intervention attitudes in seismic retrofitting of structures (Di Sarno and Elnashai, 2009)

During severe earthquakes, large amount of kinetic energy would be fed to the structures; all building codes recognize that it is not economic to dissipate energy only through the elastic capacity of the materials. Thus, the best strategy to dissipate energy is to accept that yielding occurs in the structure but in such a way that plastic deformation would be concentrated at controlled locations or structural fuses and major structural members remain elastically (Deulkar et al., 2010).

In traditional braced frames, braces are considered the structural fuses 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 idea of buckling restrained brace (BRB) borne out to improve compressive capacity and achieve more favorable behavior, BRBs exhibit stable and balanced hysteresis behaviour by cooperative ductile compression yielding before the onset of buckling (Asgarian and Amirhesari, 2008; Mahmoudi and Zaree, 2010).

2.2.1 Concentrically braced frame systems

Concentrically braced frames erect truss systems that repel lateral loads in the elastic range basically during axial forces in members. Members meet at a point or with small eccentricities that are not a cause of inelastic deformation. In the inelastic range, braced frames may concern the flexure of frame members, nevertheless the inelastic drift is predictable to be mainly a consequence of brace axial deformation, and with the exception of in certain formations that are not suggested (SEAOC, 2001).

Up to the 1994 UBC, concentrically braced frames had been dealt with by codes as basically elastic truss systems. Post-elastic behaviour was only deliberated in recommending a reduction in calculated brace strengths, which leads to increasing the elastic force capability of these systems. Consequent investigation carried out at the University of Michigan revealed that these systems, if they had accurate quantity of members and specifying of connections, could accomplish in a ductile manner (Astaneh et al., 1985; Hassan and Goel, 1991; Goel, 1992). These further ductile braced frame systems can accomplish tri-linear hysteretic behaviour, with the three varieties of behaviour being the elastic, postbuckling, and tensile yielding ranges (Bruneau et al., 1998; AISC 2005). Numerous scholars have examined the factors that may have donated to the witnessed overstrength. Osteraas and Kraeinkler (1999) accompanied a thorough study of overstrength concentric braced frames designed following the permissible stress design provisions with seismic loads per UBC seismic zone 4 and soil type S2.

Rahgozar and Humar (1998) also reported that the main parameter regulatory these factors in braced frame structures are the slenderness ratio of bracing members. Executing pushover analyses, Kim and Choi (2005) estimated the overstrength, ductility and response modification factors of the chevron type concentric braced frames with varied stories and span lengths.

The studies done by Disarno and Elnashai (2008) explain that in CBFs with stainless steel braces and columns, the escalation in overstrength is about 40% with respect to the configuration in mild steel.

Consistent with Davaran and Hoveidae opinion (2009), the type of mid-connection detail of X concentric braced frame could advance the response modification factor and the overstrength factor to about 28% and 5%, separately, in excess of the one with common mid-connection detail.

Mahmoudi et al. (2011) research tries 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, invert 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.

McCormick et al. (2007) studied three- and six-story concentrically braced frames with superelastic shape memory alloy (SMA) braces to estimate their seismic performance in comparison to traditional systems. SMAs are distinctive metallic alloys that have the aptitude to experience bulky deformations whereas returning back to their original undeformed shape given that recentering abilities to the braced frame. Comprehensive analytical models of the frames with SMA braces are established and two sets of ground motions are accustomed estimate the structures with regard to interstory drift and residual drift. The outcomes propose that the SMA braces are active in restraining interstory drifts and residual drifts through an earthquake, partially, owing to the recentering nature of superelastic SMAs.

Moghaddam and Hajirasouliha (2006) investigated the potentialities of the pushover analysis to approximate the seismic deformation demands of concentrically braced steel frames. Consistency of the pushover analysis has been confirmed by conducting nonlinear dynamic analysis on 5, 10, and 15 story frames exposed to 15 artificial earthquake annals signifying a design spectrum. It is publicized that pushover analysis with prearranged lateral load pattern offers uncertain evaluations of inter-story drift. To overwhelm this shortage, a basic analytical model for seismic response forecast of concentrically braced frames is suggested. In this approach, a multistory frame is condensed to an equivalent shear-building model by carrying out a pushover analysis. A conventional shear-building model has been adjusted by hosting additional springs to represent flexural displacements along with shear displacements. It is publicized that altered shear-building models have an enhanced evaluation of the nonlinear dynamic response of real framed structures equated to nonlinear static processes.

Moghaddam et al. (2005) studied the structural possessions which are changed with the intention of inefficient material is steadily shifted from strong to weak areas of a structure. This procedure is sustained till a state of uniform deformation is attained. It is publicized that the seismic performance of such a structure is ideal, and performs generally improved than those made by conventional methods. With the intention of avoiding difficult analysis of the frame models, a corresponding procedure is presented for performing the optimization technique on the adjusted reduced shear-building model of the frames, which is publicized to be precise sufficient for design dedications.

Hajirasouliha et al. (2010) suggested a simplified analytical model for seismic response prediction of concentrically braced frames. In the anticipated attitude, a multistory frame model is condensed to an equivalent shear-building one by carrying out a static pushover analysis. The conventional shear-building model has been amended by presenting additional springs to account for flexural displacements besides shear displacements. The sufficiency of the modified model has been confirmed by conducting non-linear dynamic analysis on 5, 10, and 15 stories concentrically braced frames exposed to 15 artificial earthquake records signifying a design spectrum. It is publicized that the suggested amended shear-building models offer an enhanced assessment of the non-linear dynamic reaction of the original framed structures, as paralleled to the conventional models. Whereas simplifying the analysis of concentrically braced frames to a large extent, and therefore decreasing the computational efforts considerably, the suggested method is precise enough for practical applications in performance assessment and earthquake-resistant design.

Mahmoudi et al. (2010) tried to assess the response modification factors of conventional concentric braced frames (CBFs) in addition to buckling restrained braced frames (BRBFs). Since, the response modification factor is determined by on ductility and overstrength, the static nonlinear analysis has been completed on building models containing single and double bracing bays, multi-floors and diverse brace alignments (chevron V, invert V and X bracing). The CBFs and BRBFs values for factors such as ductility, overstrength, force reduction because of ductility and response modification have been measured for all the buildings. The outcomes displayed that the response modification factors for BRBFs were greater than the CBFs one. It concluded that the number of bracing bays and height of buildings have had bigger effect on the response modification factors.

Shih-Ho Chao et al. (2008) studies have specified that the confidence level to accomplish collapse prevention performance purposes for typical SCBF can be tremendously and unsatisfactorily low when compared with special moment frames (SMFs). The results present of a study in which a newly developed performance-based plastic design (PBPD) methodology was useful to CBF with buckling type braces. Initially the method was established and effectively applied to moment frames and more lately prolonged to other steel framing systems too. 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. Plastic design is then done to detail the frame members and connections to attain the planned yield mechanism and behavior. Outcomes of extensive inelastic dynamic analyses carried out on example frames considered by the PBPD method displayed that the frames achieved all the desired performance objectives, together with the intended yield mechanisms and the story drifts. Reliability-based estimation in compliance with FEMA 351 showed that the PBPD frames have confidence levels against global collapse much greater than those of equivalent SCBFs designed by recent practice.

In the study of Xiaodong et al. (2009), nonlinear dynamic analysis is applied for 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 occasions 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 moreover, carried out to authenticate the theoretical remarks and count the stiffness/strength requests of gravity columns to evade drift concentration. The analysis consequences aid the tendencies and opinions acquired from the theoretical formulation. Eventually numerical imitations are applied with the seismic force reduction factor Ds, a slenderness ratio of braces λ, and number of stories, as main analysis variables. The consequences prove that a few (e.g. four) fixed-base gravity columns can cause breakdown avoidance for sensibly designed low-rise CFBs under the set of large ground motions.

In the study of Tremblay (2002), 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 parameters that were observed are the buckling strength of the bracing members, the brace post-buckling compressive resistance at various ductility levels, the brace maximum tensile strength comprising strain hardening effects, and the lateral deformations of the braces upon buckling. Equations are planned for each of these parameters. Furthermore, the maximum ductility that can be reached by rectangular hollow bracing members is inspected.

CHAPTER 3

METHODOLOGY

3.1 Description of the analytical models

The multi degree of freedom (MDOF) structures considered in this study were eight steel moment-resisting frame (MRF) buildings having four, eight, twelve, and sixteen stories high. These frames were first designed by Santa-Ana and Miranda (2000). The buildings have the same plan, which consists of three bays on each direction as shown in Figure 3.1. The buildings were assumed to have a homogeneous mass circulation over their height and a non-homogeneous lateral stiffness circulation. Steel members in the buildings were designed using the lateral load distribution specified in the uniform building code (UBC, 1994). The member stiffness was achieved to acquire fundamental periods of vibration for each structure typical of those acquired from actual earthquake records. As a result, two frames with different dynamic properties, namely flexible and rigid frames were considered. Furthermore, as reported in the study of Santa –Ana and Miranda (2000), with the exception of beam-to-column connections in the top floor, the steel section of structural members was chosen such that the sum of plastic section modulus of the columns framing into each beam-column joint was larger than the sum of plastic section modulus of the beams framing into the interchangeable joint.

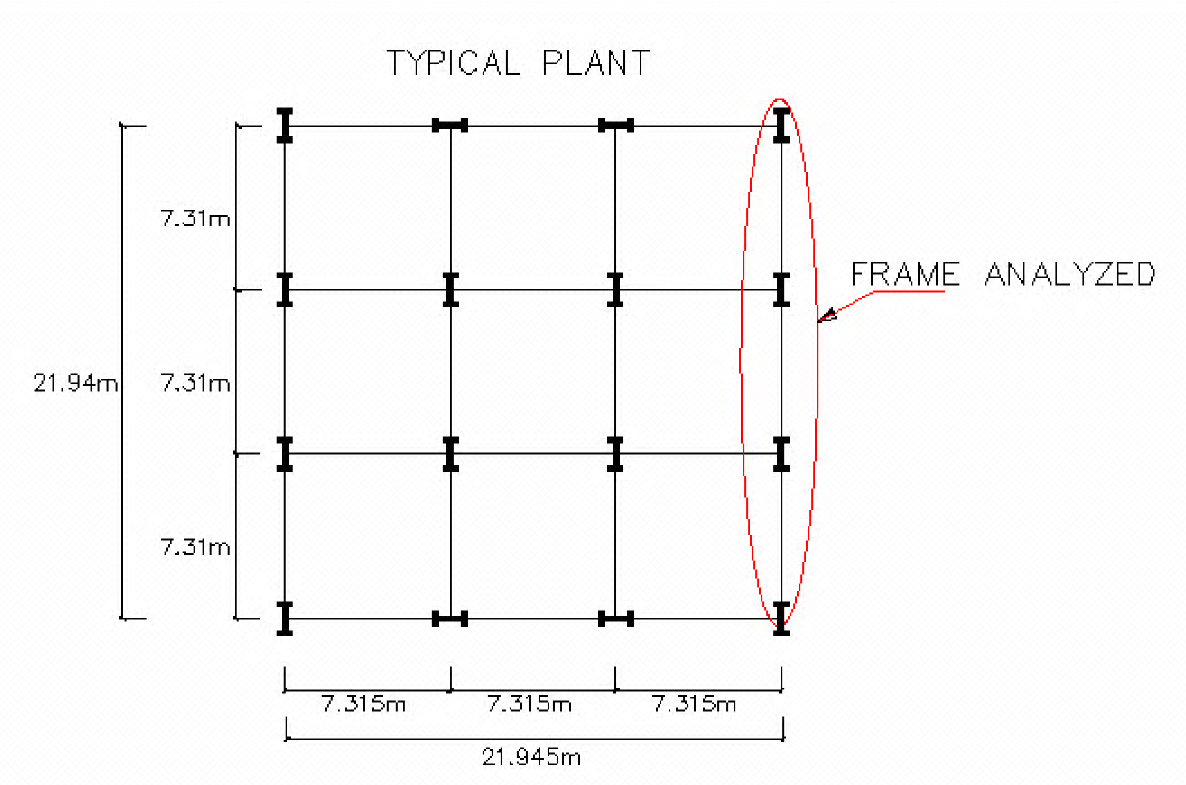


Figure 3.1 Plan view of the multi-story buildings (Santa-Ana and Miranda, 2000)

The beams and columns were built with W profiles. The storey height in the models was 3.66 m for all the floors except in the first floor in which the storey height was 5.49 m. Section profiles for the unbraced frames are shown in Figures 3.2 and 3.3, and dynamic properties of MRFs are presented in Table 3.1.

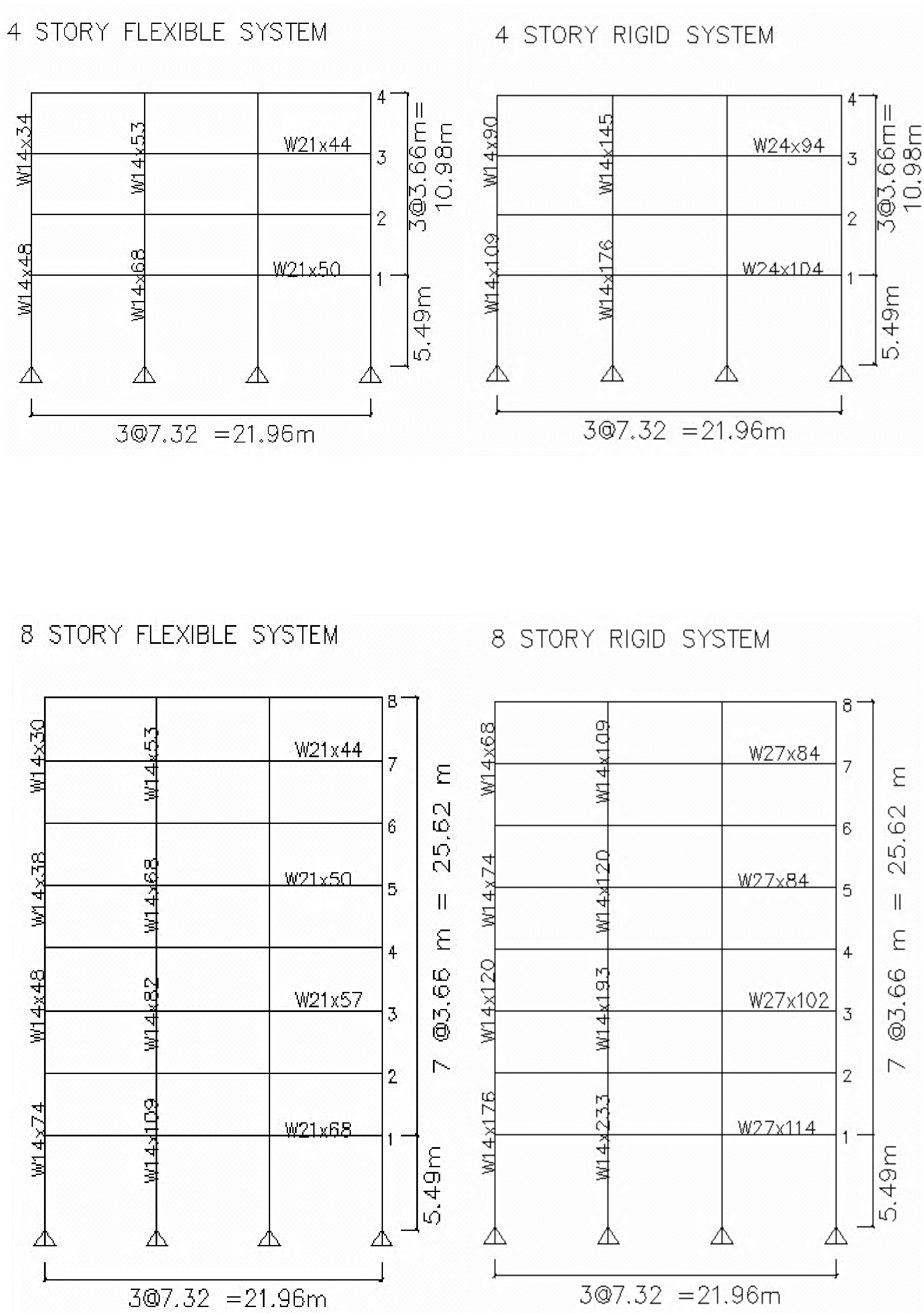


Figure 3.2 Elevations view of 4 and 8 stories flexible and rigid frames under consideration (Santa-Ana and Miranda, 2000)

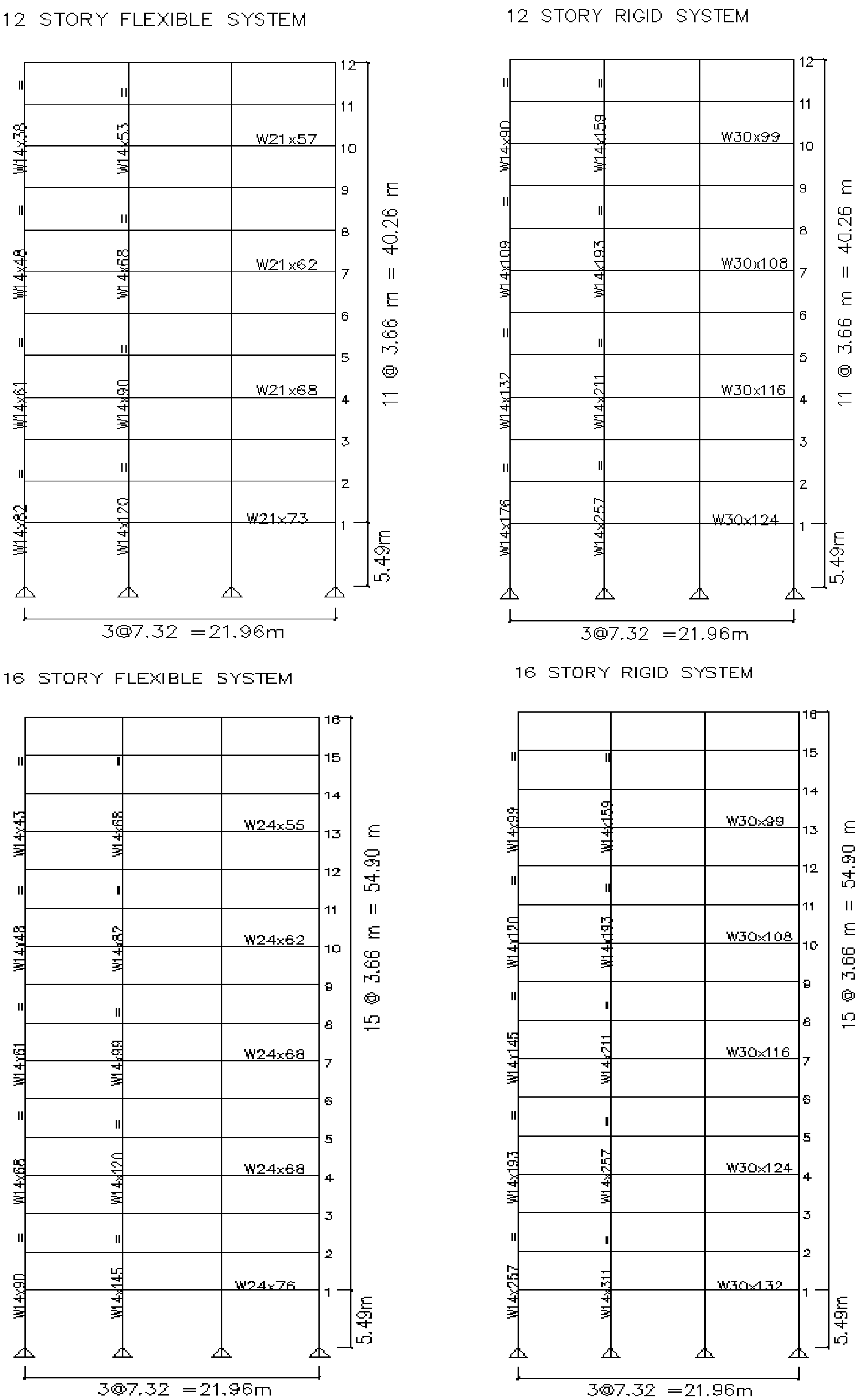


Figure 3.3 Elevations view of 12 and 16 stories Flexible and rigid frames under consideration (Santa-Ana and Miranda, 2000)

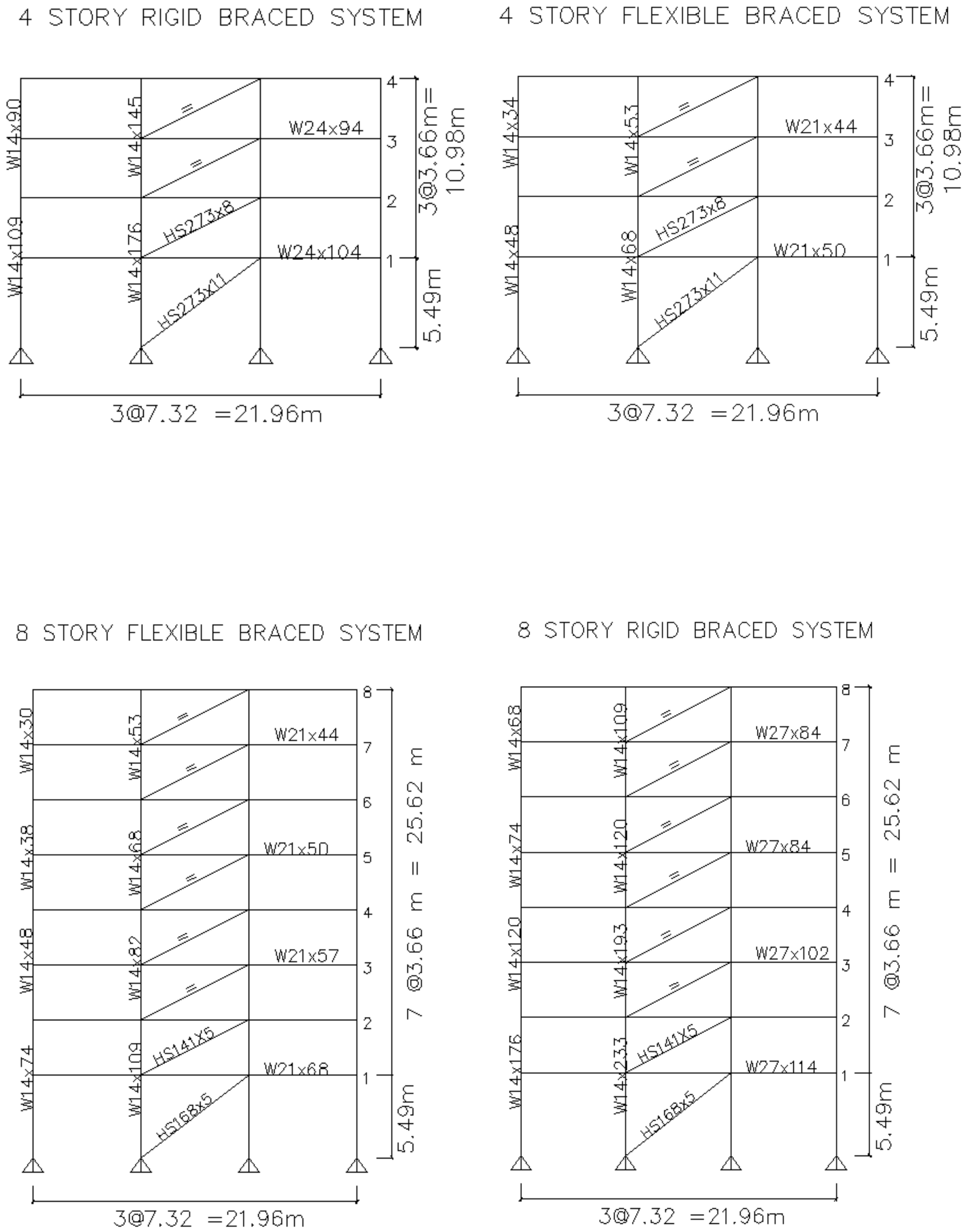


Figure 3.4 Elevations view of 4 and 8 stories flexible and rigid frames with diagonal braces analysed in this study

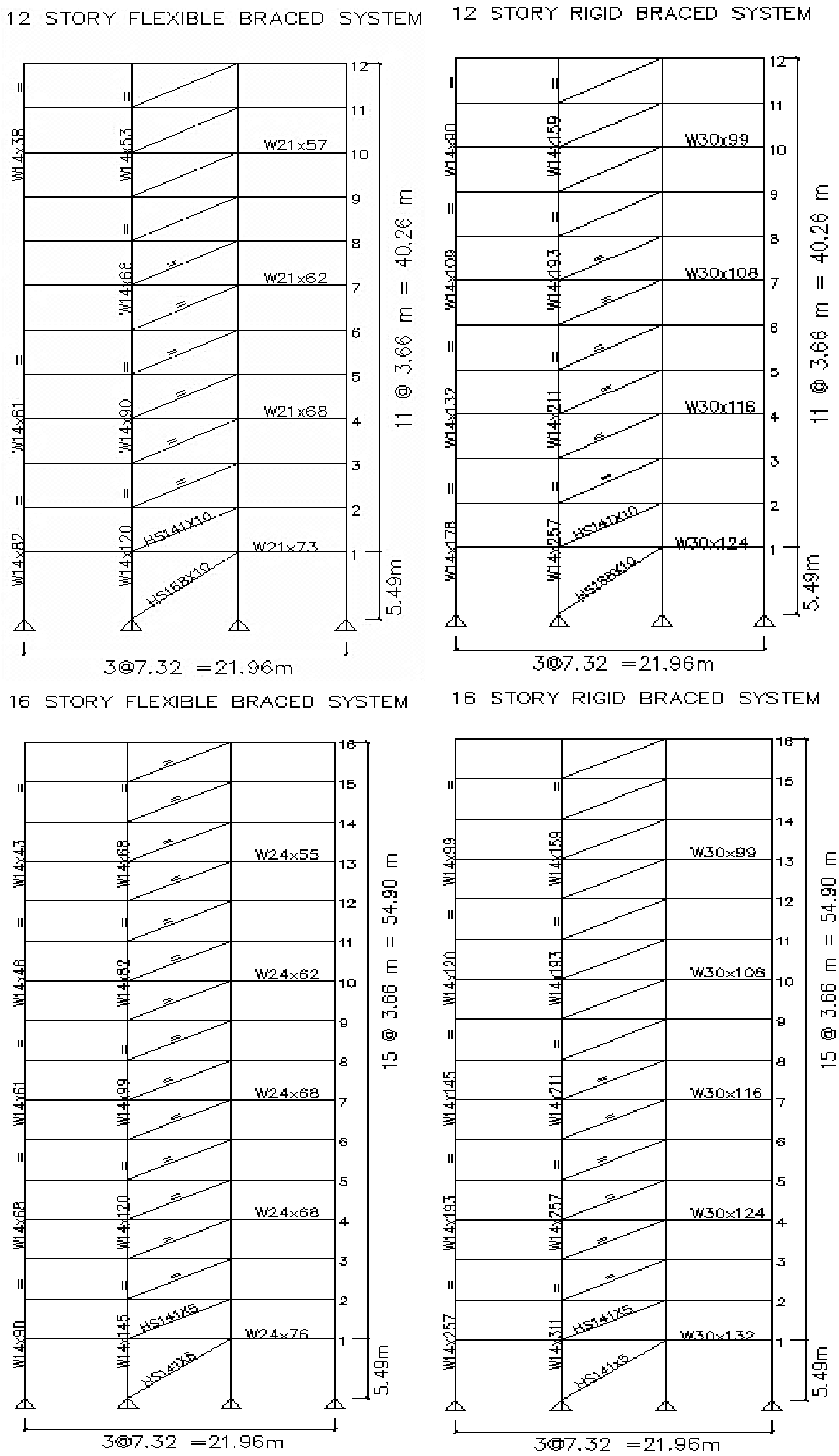


Figure 3.5 Elevations view of 12 and 16 stories flexible and rigid frames with diagonal braces analysed in this study

Table 3.1 Dynamic properties of the unbraced and braced frames

Buildings	T_1 (s)	T_2 (s)	T_3 (s)
4 story flexible unbraced	1.54	0.344	0.17
4 story flexible braced	0.374	0.123	0.08
4 story rigid unbraced	0.847	0.1926	0.0923
4 story rigid braced	0.269	0.089	0.05
8 story flexible unbraced	1.97	0.616	0.326
8 story flexible braced	0.949	0.327	0.186
8 story rigid unbraced	1.081	0.349	0.17
8 story rigid braced	0.664	0.235	0.131
12 story flexible unbraced	2.53	0.839	0.45
12 story flexible braced	1.296	0.44	0.245
12 story rigid unbraced	1.35	0.44	0.237
12 story rigid braced	0.854	0.29	0.169
16 story flexible unbraced	2.91	1.011	0.55
16 story flexible braced	1.823	0.627	0.347
16 story rigid unbraced	1.659	0.58	0.322
16 story rigid braced	1.249	0.44	0.24

By considering two sets of target fundamental periods, the moment resisting frames with different number of stories were designed by Santa-Ana and Miranda (2000). These sets of target fundamental periods were chosen to approximately provide upper and lower bounds of those recently obtained from earthquake records of

California (Goel and Chopra, 1997). It is necessary to note that even the rigid MRFs considered herein have fundamental periods longer than those obtained using the expression recommended by the UBC (Uniform Building Code, 1994) for MRF buildings (Santa-Ana and Miranda, 2000). Moreover, it was perceived that the first two modes captured most of the response of the structure which was about 97%. Also it was observed that the fundamental periods of the unbraced frames were shorter than braced frames which it was an indication that the braced frames were stiffer than unbraced frames. The flexural moment capacity of beams and columns was decided using real yield strengths of 337.8 and 399.9 MPa, separately.

Then, concentrically lateral braces were inserted to middle bays of these frames as seen in Figures 3.4 and 3.5. Total of 16 cases were analyzed in this study (8 unbraced cases and 8 braced frames with concentrically lateral brace). The dynamic properties of these braced frames are also given in Table 3.1. As seen from Table 3.1, the fundamental periods of the unbraced frames were shorter than braced frame which was also an indication that the braced frames were stiffer than unbraced frames.

In the nonlinear static analysis (pushover analysis), concentrated incremental static lateral loads were requested at every floor applying triangular distribution during the height in the structure. The structure models were based upon the centerline dimensions that beam, and columns span between the crossings at the intersections of beam and column centerlines, and no fundamental points or insertion points were used for the modeling objective.

3.2 Nonlinear behaviour of structural elements

The nonlinear behaviour of a building structure depends on the nonlinear responses of the elements that are used in the lateral force resisting system. Hence, before applying any nonlinear analysis method on a building structure, the nonlinear behaviour of such elements must be clearly specified and assessed.

In FEMA 356 (FEMA 356, 2000), the generalized load deformation relation of a structural member while exhibiting nonlinear behavior is shown in Figure 3.6. After the member yields (when applied load/yield load proportion (Q/Qy) is equal to 1), the consequent strain hardening gives the strain hardening in the load-deformation relation as the member deforms toward the anticipated strength. The horizontal axis of this diagram may either declare curvature or strain.

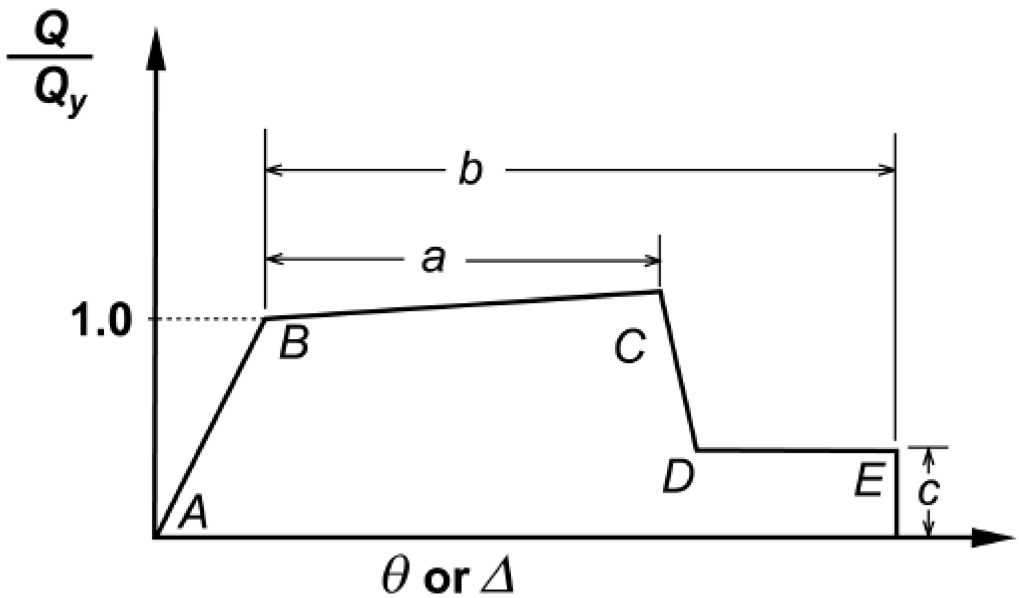


Figure 3.6 The generalized load deformation relation while exhibiting nonlinear behavior of a structural member (FEMA 356, 2000)

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C coincides to the deformation at which significant strength degradation starts. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

ATC-40 and FEMA-356 codes also define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 3.7, the acceptance criteria on a force versus deformation diagram are given. In this diagram, the points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention, separately.

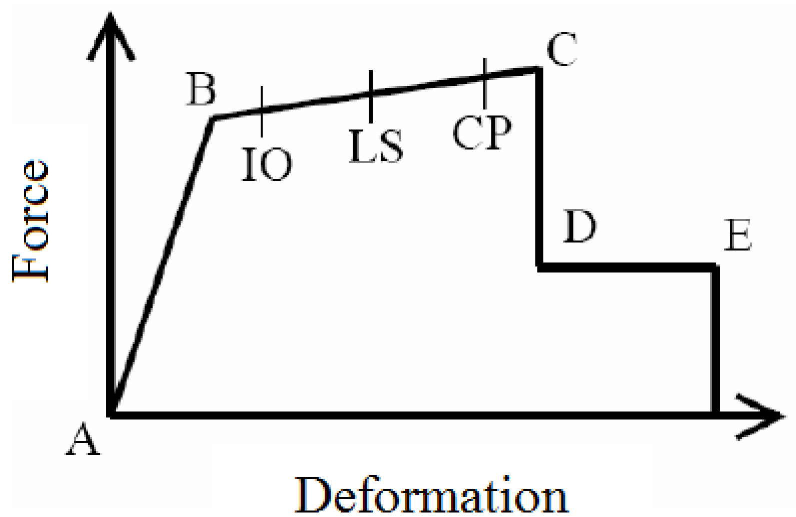


Figure 3.7 Acceptance criteria on a force versus deformation diagram (FEMA 356, 2000)

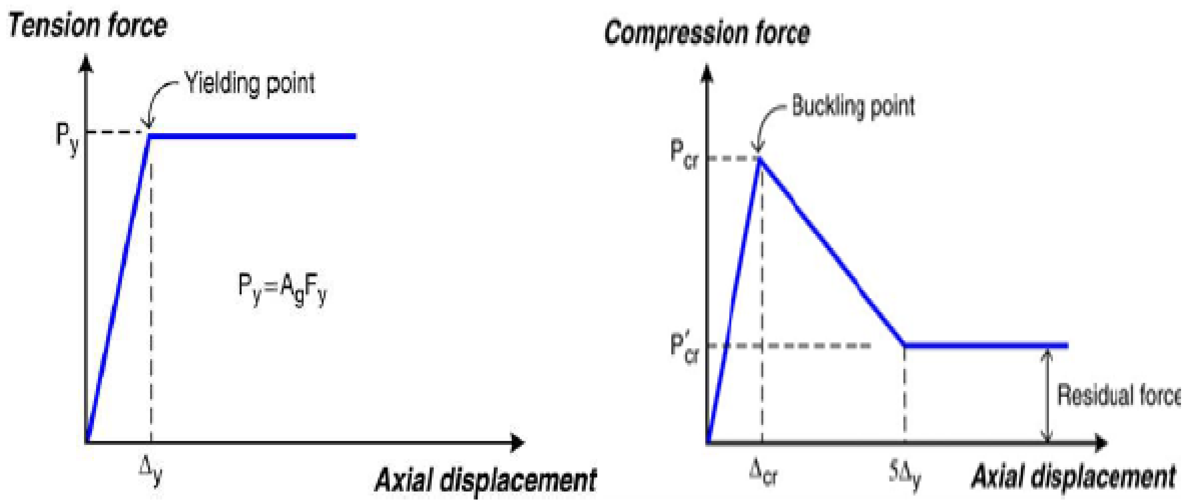
Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element.

There are three kinds of hinge on properties in SAP2000. They are default hinge on properties, user-defined hinge on properties and generated to hinge on properties. Solely default hinges on properties and user-defined hinge on properties can be allocated to frame elements. Whenever these hinge properties (default and user-defined) are allocated to a frame element, the program automatically produces a new generated hinge on property for every single hinge.

Default hinges on properties could not be adjusted, and they are section dependent. Whenever default hinges on properties are used, the program unifies its built-in default criteria with the clarified section properties for every element to propagate the eventual hinge properties. The built-in default hinges on properties for steel, and concrete members are based on ATC-40 (ATC 40, 1996) and FEMA-273 (FEMA273, 1997) norms.

User-defined hinge on properties can be depended on default properties, or they can be fully user-defined whenever user-defined properties are not depended on default properties, then the properties can be observed and adjusted originated to hinge on properties are used throughout the analysis. They could be examined, but they could not be adjusted. In this study, axial force-moment interaction (P-M3) according to FEMA356 (FEMA356, 2000) was defined for determining the nonlinear hinge properties of a columns, such hinges required that the axial force vs. moment interaction diagram to be calculated. When an axial force and corresponding moment value of a loading was formed outside the plotted interaction diagram, this column exhibited nonlinear behavior. Plastic hinge moment capacity (M3) according to FEMA356 (FEMA356, 2000) was introduced for plastic hinges of the beam elements.

For modeling, the nonlinear behavior of CBs (Figure 3.8), the phenomenological model suggested by Jeol and Jain (1980), which also existed in FEMA 274 (1997), was used. The values of the modeling parameters were selected based on Table 5-8 of FEMA 273 (1997).



a) Tension brace b) Compression brace

Figure 3.8 Simplified analysis model for force–displacement relationship of brace (Kim and Choi, 2005)

3.3 Nonlinear static pushover analysis

The static pushover analysis is appropriate a common instrument for seismic performance assessment of living and modern structures. The prediction is that the pushover analysis will supply sufficient information on seismic requests required by the design ground motion on the structural system and its components.

The pushover analysis of a structure is a static non-linear analysis under stable vertical loads and progressively increasing lateral loads. The corresponding static lateral loads nearly constitute earthquake induced forces. A plot of the total base shear versus top displacement in a structure is acquired by this analysis that would designate any early failure or weakness. The analysis is conveyed out up to failure; therefore it authorizes perseverance of collapse load and ductility capacity. In this simplified method, a capacity curve is obtained which shows the relation of base shear and roof displacement. This curve represents the behaviour of the building structure under increasing base shear forces. As the capacities of the members of the lateral force resisting system exceed their yield limits during the increasing of the base shear forces, the slope of the force-deformation curve will change, and hence the nonlinear behavior can be represented (Altuntop, 2007).

In the pushover analysis, the applied lateral forces to a model are increased in a regular manner depending on the initial load pattern. Hence, a triangular load pattern determined from the first mode shape of the structure was utilized as an initial load pattern as shown in Figure 3.9. Member forces are calculated for each step and the stiffness of the members whose capacities are exceeded is changed according to the hinge properties in the next step of the analysis. This process ends when the structure becomes unstable. Figure 3.10 displays a typical pushover curve as an example.

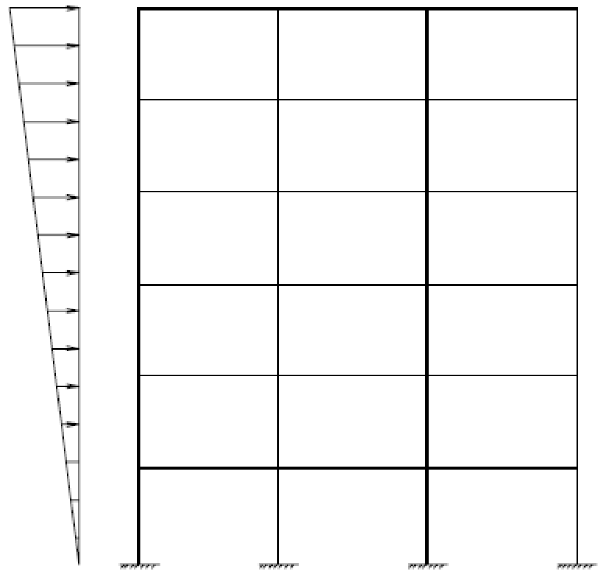


Figure 3.9 Load pattern for nonlinear static pushover analysis (Yang and Wang, 2000).

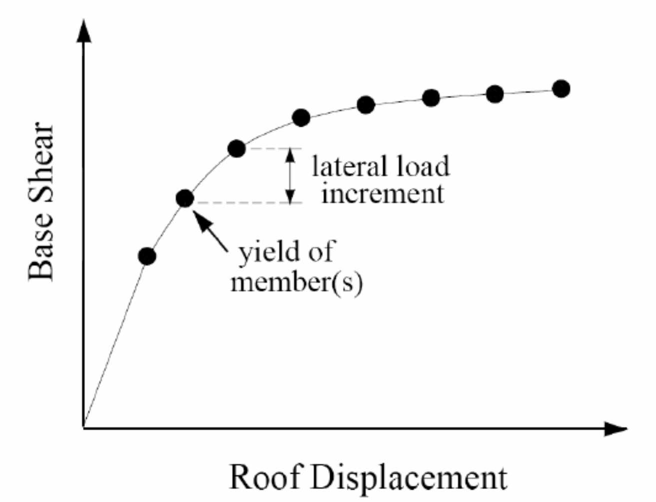


Figure 3.10 An example pushover curve of a building structure (Sermin, 2005)

In the pushover analysis, monotonically growing lateral forces are executed to a nonlinear mathematical model of the building up to the time that displacement of the authority node at the roof level excels the target displacement. The lateral forces ought to be executed to the building using issues or portraits that fastened, although closely the likely distribution of inertial forces in the design earthquake. The currently NEHRP guidelines such as FEMA 273 (1997), FEMA 356 (2000) show that, for a precise earthquake, the building ought to have adequate capacity to resist a particularized roof displacement. This is named the target displacement and is clarified as a guess of the likely building roof displacement in the design earthquake.

The basic query in the implementation of the pushover analysis is the quantity of the target displacement at which seismic performance evaluation of the structure is to be carried out. Target displacement attends as an approximation of the global displacement of the structure is predictable to occurrence in a design earthquake. It is the roof displacement at the centre of mass in the structure (Krawinkler and Seneviratna, 1997).

In the pushover analysis, it is supposed that the target displacement in the MDOF structure can be appraised as the displacement request for the corresponding equivalent SDOF system altered to the SDOF domain during the use as a shape factor. This hypothesis which is regularly a guess can only be acceptable within limitations and exclusively be acceptable within limitations and only if extreme attention is taken in taking in the projected SDOF displacement demand all the important ground motion and structural response symptoms that considerably influence the highest displacement of the MDOF structure. Basic principle in this access is the hypothesis that the highest

MDOF displacement is regulated by a single shape factor lacking regards to higher mode effects. Subordinate to the Non-linear Static Procedure, a model immediately including inelastic material response is replaced to a target displacement, and resulting from inner deformations and forces are concluded. The mathematical model of the building is exposed to monotonically increase lateral forces or displacements up to the time of either a target displacement are passed or the building crashes. The target displacement is deliberate to denote the maximum displacement probable to be experienced through the design earthquake. The pushover analysis can be performed considering the control over the force or displacement (Krawinkler and Seneviratna, 1997).

Force control option was useful when the magnitude of the load was known clearly, and the structure was expected to support that load. The displacement control was useful when the magnitude of the load was unknown and displacements were searched (Krawinkler and Seneviratna, 1997).

In the current study, the pushover analysis was conveyed out in these following steps:

- The model representing the building structure was created and vertical loads (dead load and live load), member properties and member nonlinear behaviors were defined and assigned to the model,
- Hinge properties were defined and these properties were assigned to the member ends,
- Lateral load patterns to be used in the pushover analyses were assigned,

- An initial force controlled pushover loading to be used for the lateral load increment analyses, was applied to the model as a pushover case. This pushover load case was composed of the dead loads and reduced live loads,

- A new displacement controlled pushover case was defined considering the lateral load pattern which was defined above for the incremental pushover analysis starting from the initial pushover case.

CHAPTER 4

RESULTS AND DISCUSSION

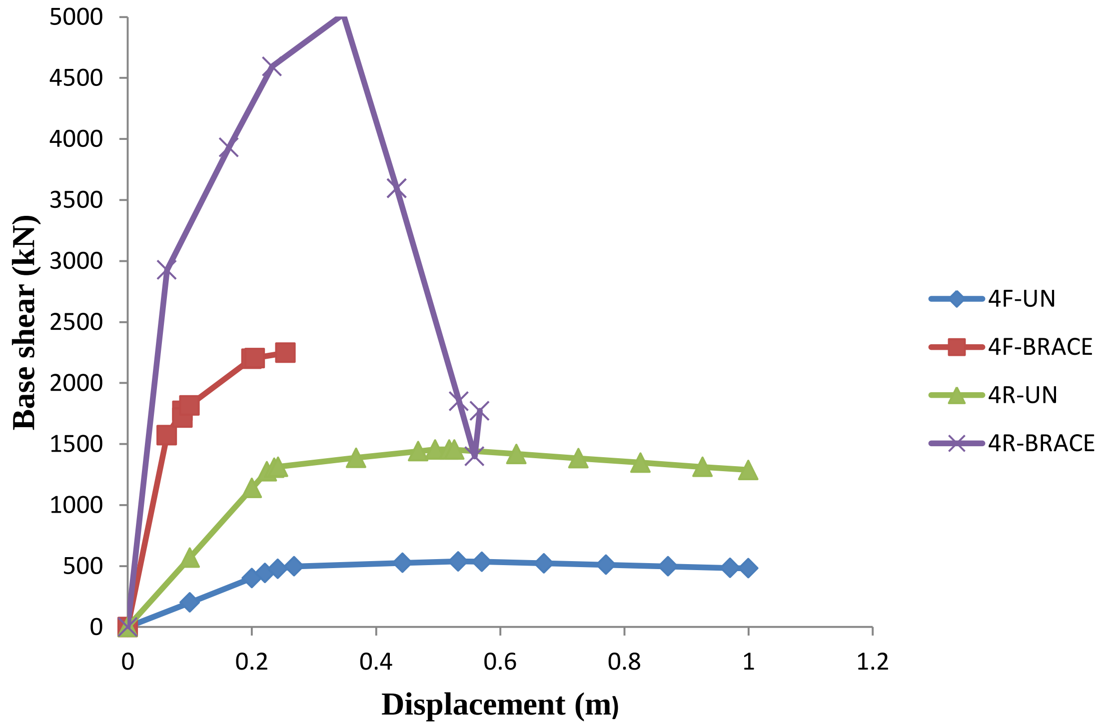
4.1 General

In this section, the results for unbraced frames (UFs), concentrically braced frames with lateral bracing obtained from nonlinear static and dynamic analysis were given and discussed comparatively. In the present study, a total of 16 different cases were taken into account and structural performance of unbraced and concentrically diagonal brace systems having different number of stories and different type of frame systems with and without braces were evaluated. Performance characteristics in terms of capacity curves, inter story drift index, global damage index, base shear, and target displacement were given.

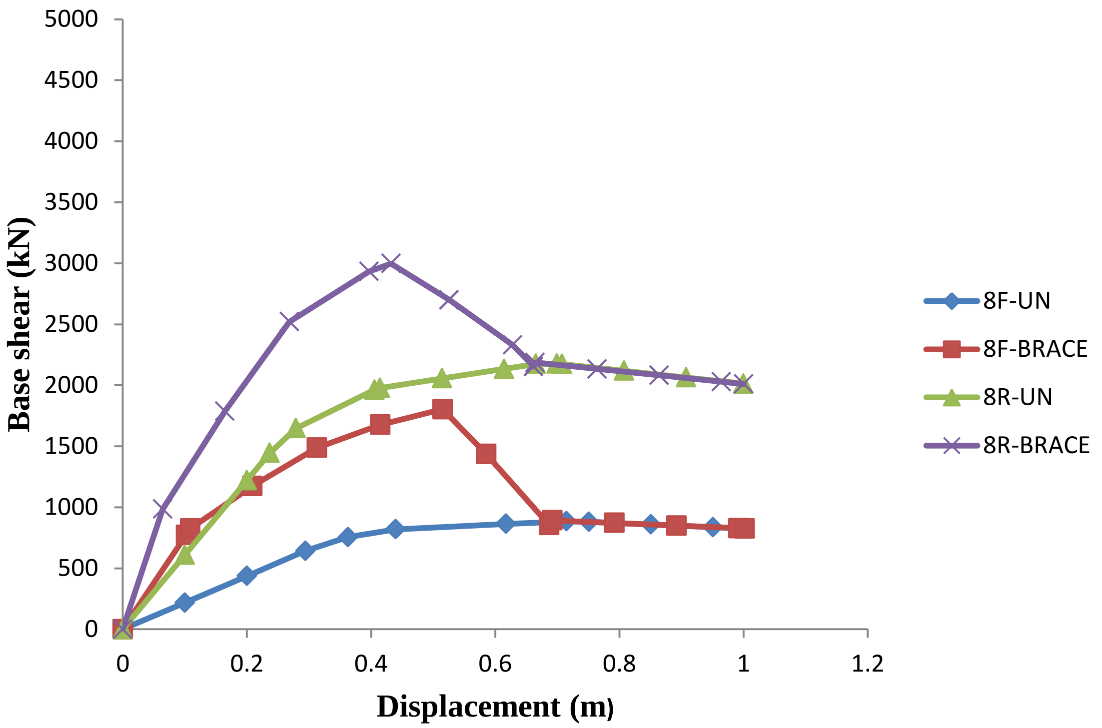
4.1.1 Capacity curves

The capacity curves (pushover curves) were evaluated for different frame type. Figure 4.1 shows the comparison of the capacity curves of flexible and rigid frames with and without diagonal type braces. It was pointed out that for both flexile and rigid frame systems, the concentrically braced frames (CBFs) were much stiffer and showed a better performance compared to the unbraced frames (UFs), also putting the stiffness of the concentrically braced frames in perspective with the unbraced frames; it was apparent that in general the former was much stiffer than later. However, in some cases, there was a considerable difference in the pushover curves, and that was due to a difference in the number of stories and difference in the frame type.

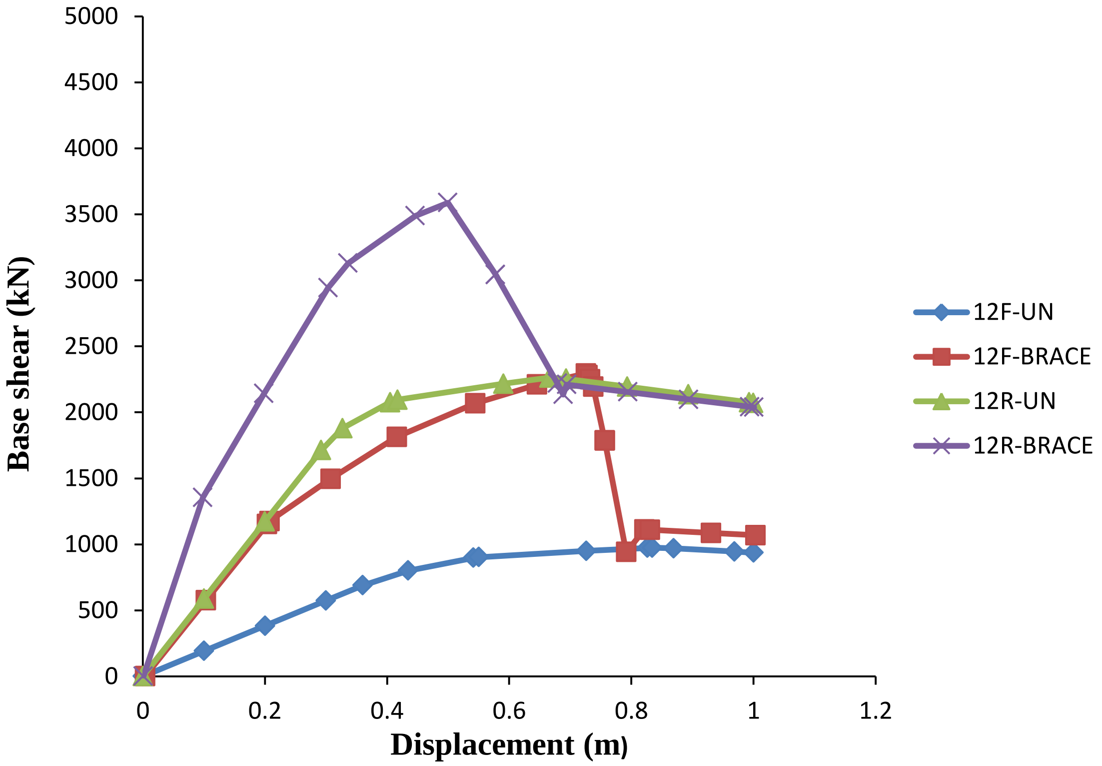
It was observed that the capacity curves in general for unbraced frames were bilinear since at the beginning the structure was globally in the elastic stage and provided a linear elastic slope, and then when the base shear was exceeded, some structural members (beams and columns) would yield and induce a change in the slope of the capacity curve. However, in the case of the braced frames, the preliminary change in the elastic slope was due to the yielding of the braces, and the other changes was due to the yielding of the structural members. Hence the length of the second slope was the deferment between the yielding of the brace and the structural members.



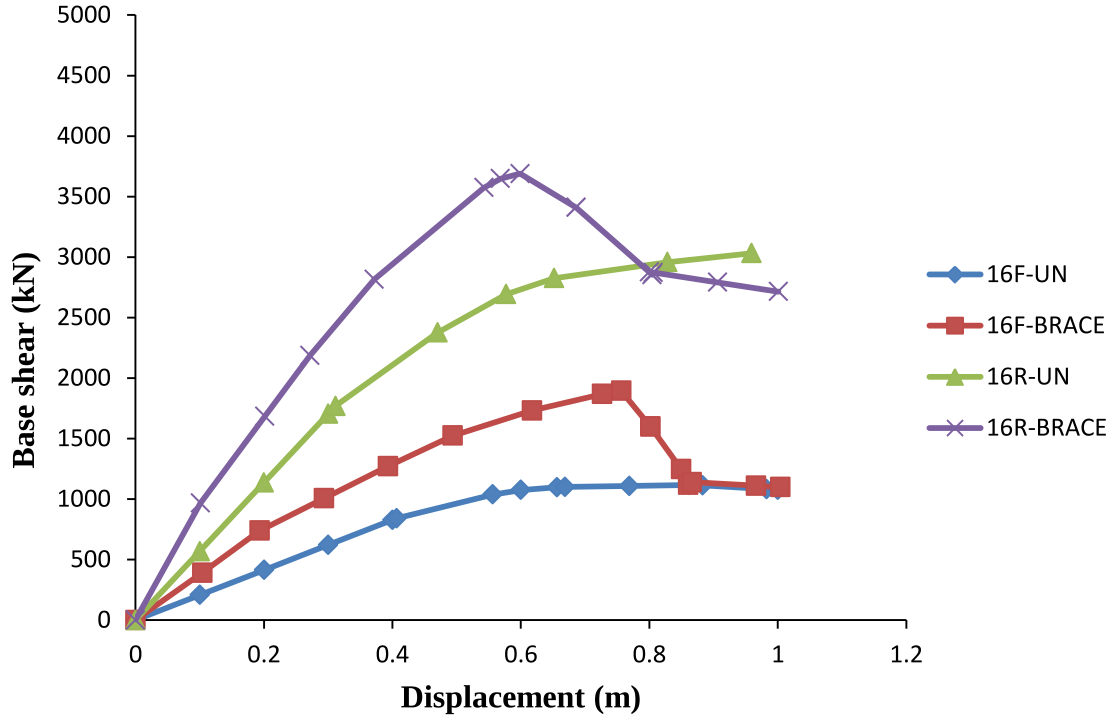
(a) Four story building



(b) Eight story building



(c) Twelve story building



(d) Sixteen story building

Figure 4.1 Capacity curves for 4, 8, 12, and 16 stories unbraced and braced flexible and rigid frames

The values of target displacements were evaluated from FEMA 356 coefficient method as given in Table 4.1. Generally, the rigid frames had smaller values of target displacements. The use of CB considerably decreased the value of target displacement compared to unbraced frames, especially in the case of rigid CB frames. It is the roof displacement at the center of mass of the structure.

In the other hand, number of stories have effect on the this value, by increasing number of stories the value of target displacement is increased, for example in the case of sixteen story frame the target displacement is greater than other buildings, and this is because of the fact that target displacement is based on the height of the structure as a parameter.

Table 4.1 Target displacements obtained for the braced and unbraced moment resisting frames

Buildings	**Target displacement(m)**
4 story flexible unbraced	0.271
4 story flexible braced	0.047
4 story rigid unbraced	0.15
4 story rigid braced	0.022
8 story flexible unbraced	0.376
8 story flexible braced	0.191
8 story rigid unbraced	0.201
8 story rigid braced	0.122
12 story flexible unbraced	0.493
12 story flexible braced	0.278
12 story rigid unbraced	0.252
12 story rigid braced	0.159
16 story flexible unbraced	0.548
16 story flexible braced	0.389
16 story rigid unbraced	0.318
16 story rigid braced	0.235

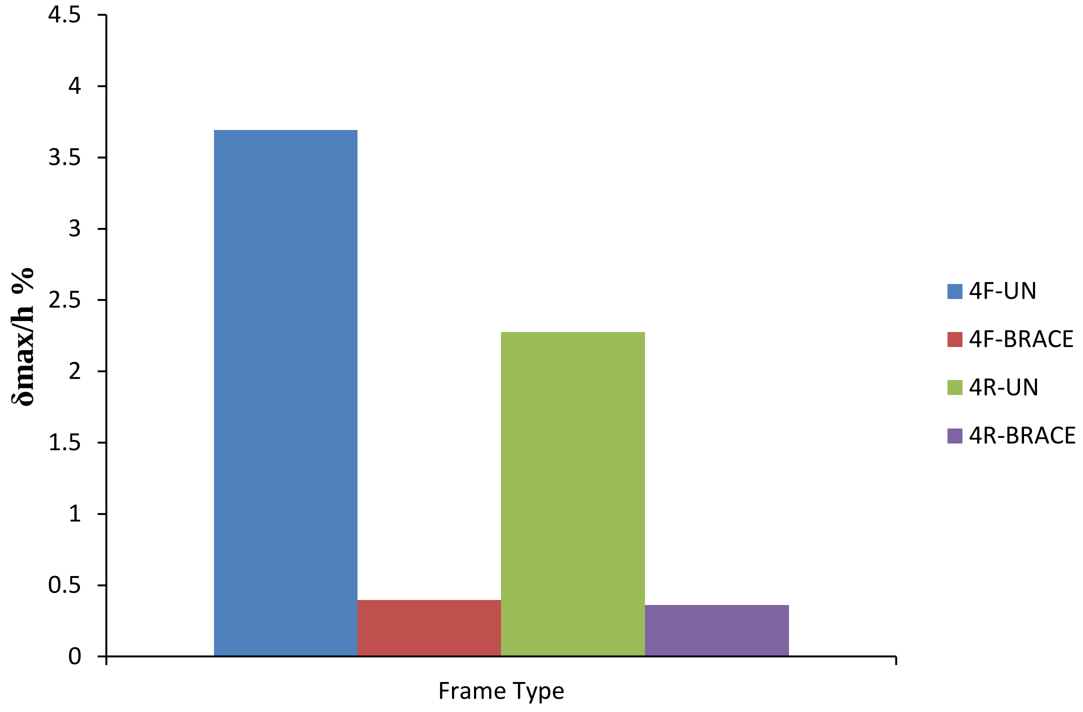
4.1.2 Interstorey index

The maximum inter storey drift (δmax) divided by the storey height (h) is defined as *the maximum inter storey index*. This index is a good indication of the damages experienced by the structural members.

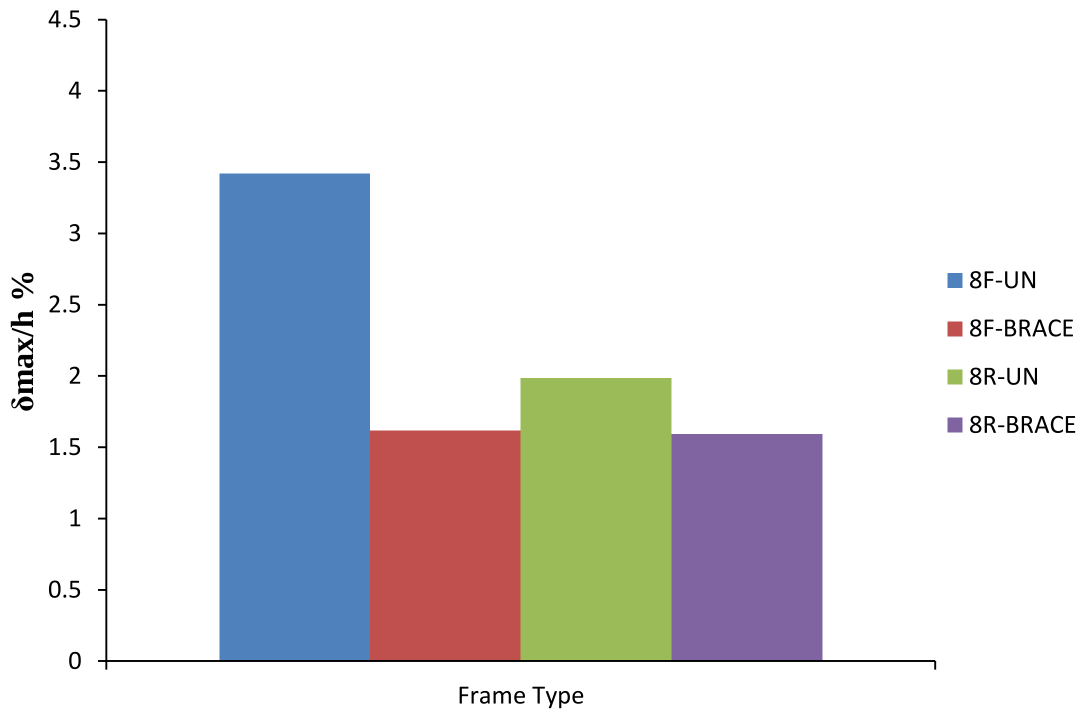
The maximum inter storey index was assessed for both unbraced frame (UF) and concentric braced (CB) frames subjected to lateral loading. Figure 4.2 compares maximum inter storey index for UF and CB frames with different frame property.

In case of unbraced condition and braced one, for both flexible and rigid systems, braced frames showed a better performance compared to UFs. MRFs braced with concentric braces retained a relevant amount of strength even after the initial buckling of a brace. In addition to these, it was observed that frames with concentric braces exhibit buckling deformations as a result; a rapid dropping in strength of frames occurred at close steps to failure.

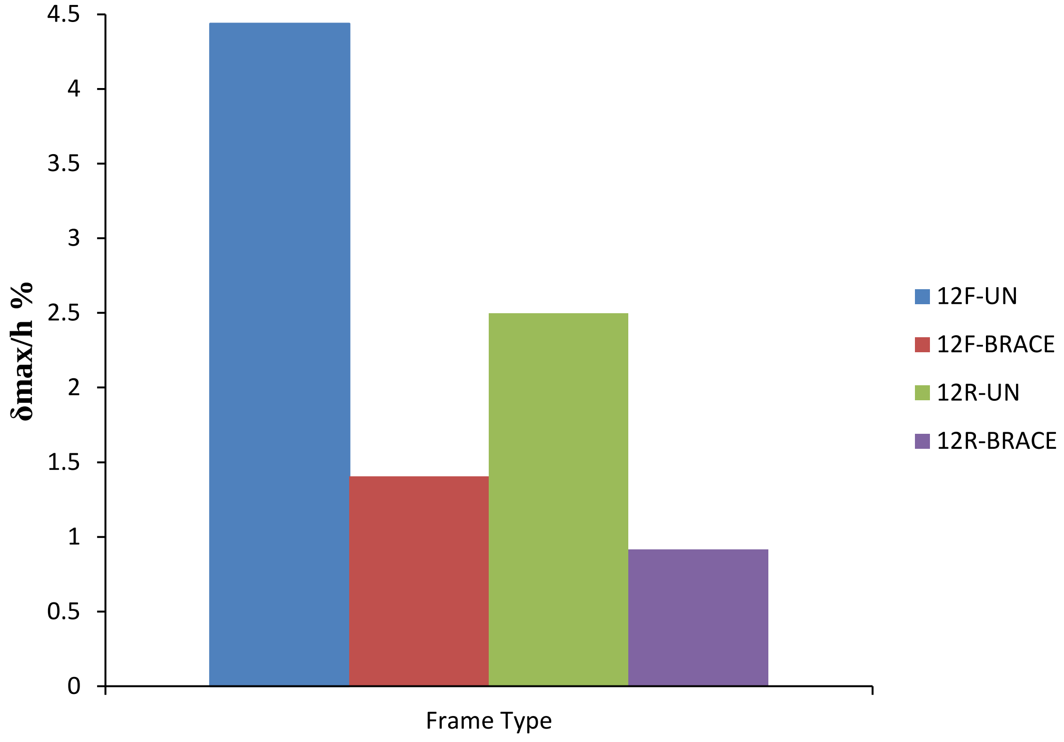
It was also observed from the figure that there was a difference between the inter storey indexes of the stiff and flexible frames equipped with CB. However, stiff braced frames were performing better than the flexible braced frames. For example, the inter storey index in the flexible unbraced frames was more in comparison to the braced one, and inter storey index in the stiff unbraced frames was more compared to the braced one. Even so, the differences in the interstorey index for the stiff braced frame were smaller than the interstorey index for the flexible braced frame.



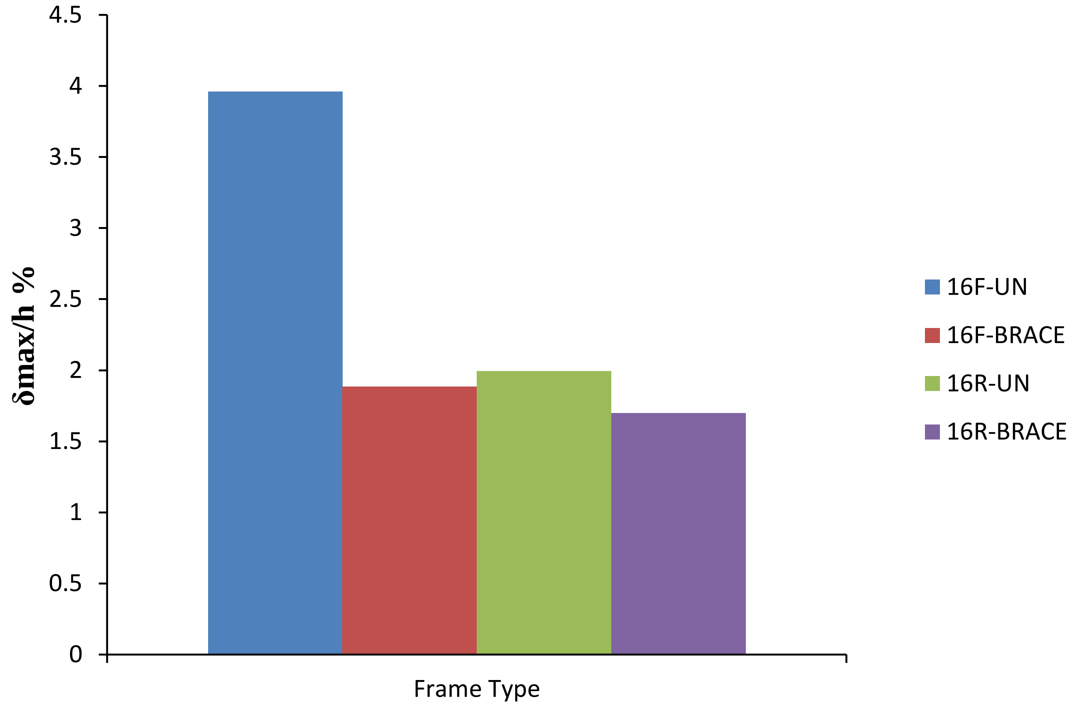
(a) Four story building



(b) Eight story building



(c) Twelve story building



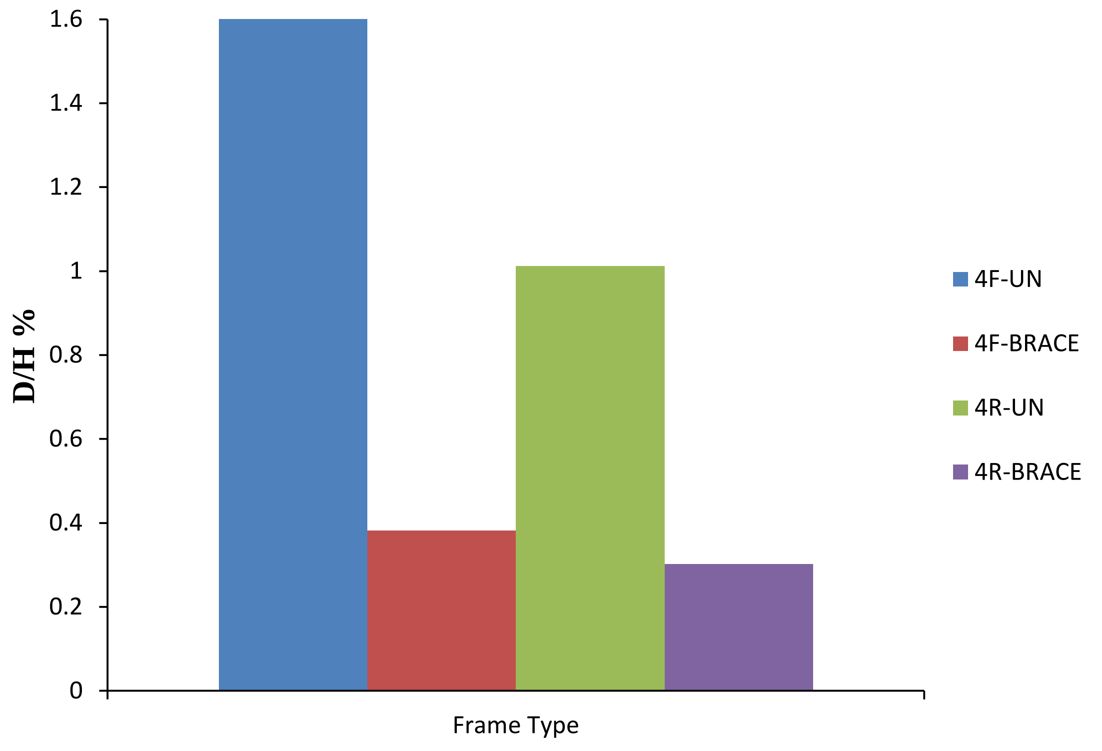
(d) Sixteen story building

Figure 4.2 Maximum interstorey indexes for different MRFs

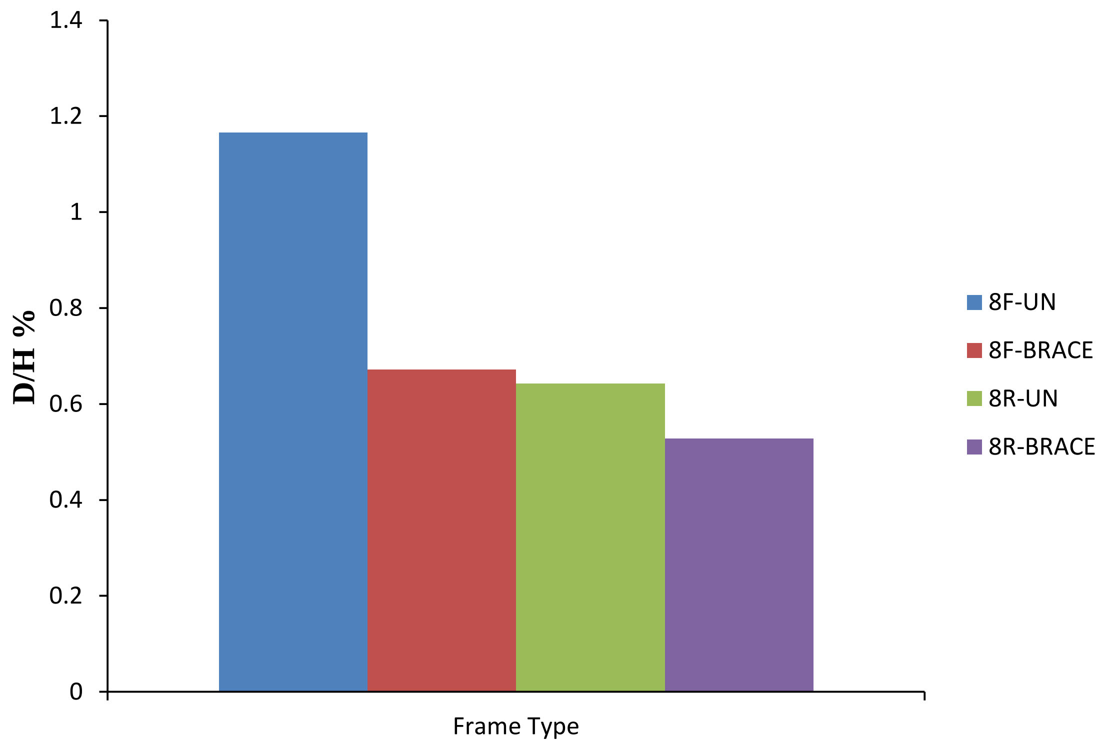
4.1.3 Global damage index

The ratio of the roof displacement (D) over the total height of the building (H) is defined as the *global damage index*. Figure 4.3 compare the global damage index for UF and CB frames with different frame property. Comparison of global damage index of the frames revealed that the global index for UF was considerably greater than that for CB frames, and CB frames showed better performance in comparison to unbraced frames (UFs). The use of CBs resulted in reductions of 15-25%. The magnitude of these global deformations depends mainly upon number of stories and especially characteristics of frame (flexible or rigid systems).

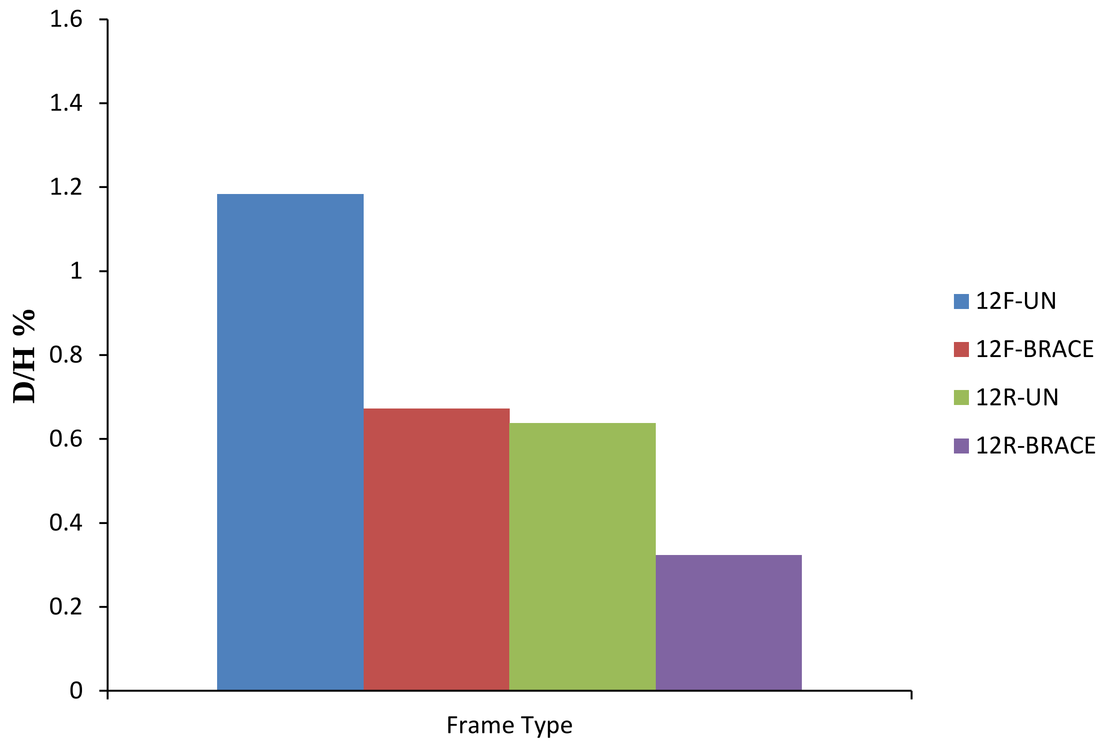
From the result of this index, it was evident that the number of story of structure has great effect on this index, for example, in the case of four-story buildings; the global index is higher than the other structures. Moreover, the inclusion of CB into the same structure resulted in lower index. It was observed that this index had a tendency to diminish with the use of rigid type of frames.



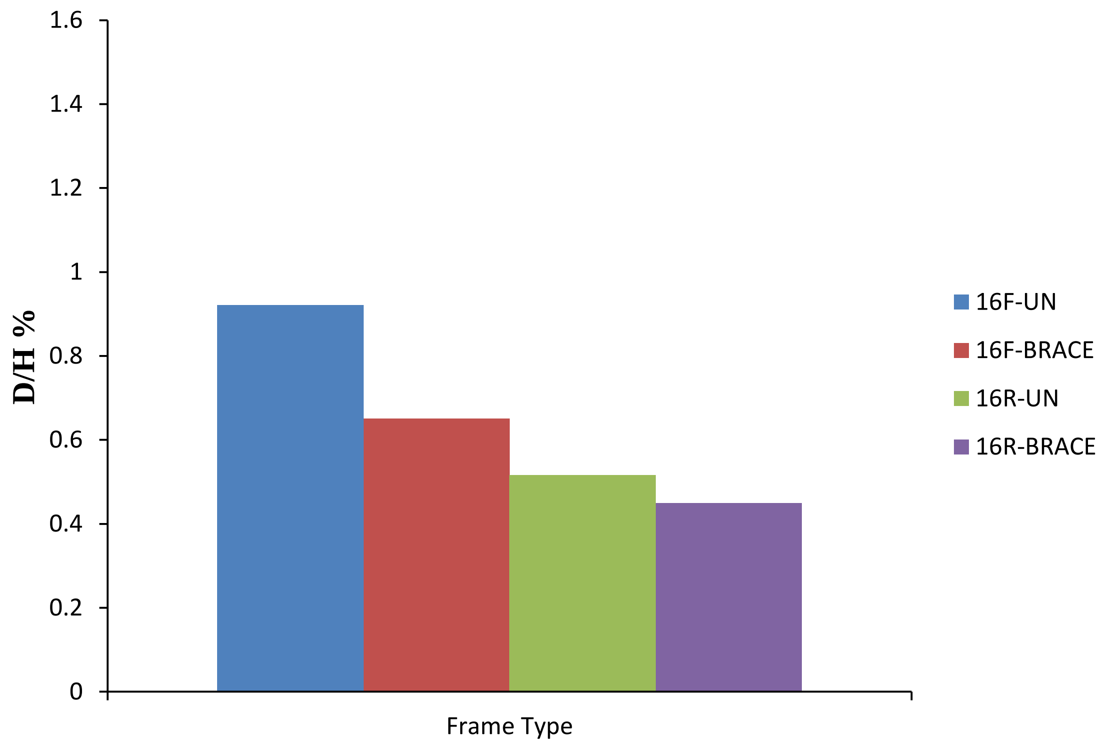
(a) Four story building



(b) Eight story building



(c) Twelve story building



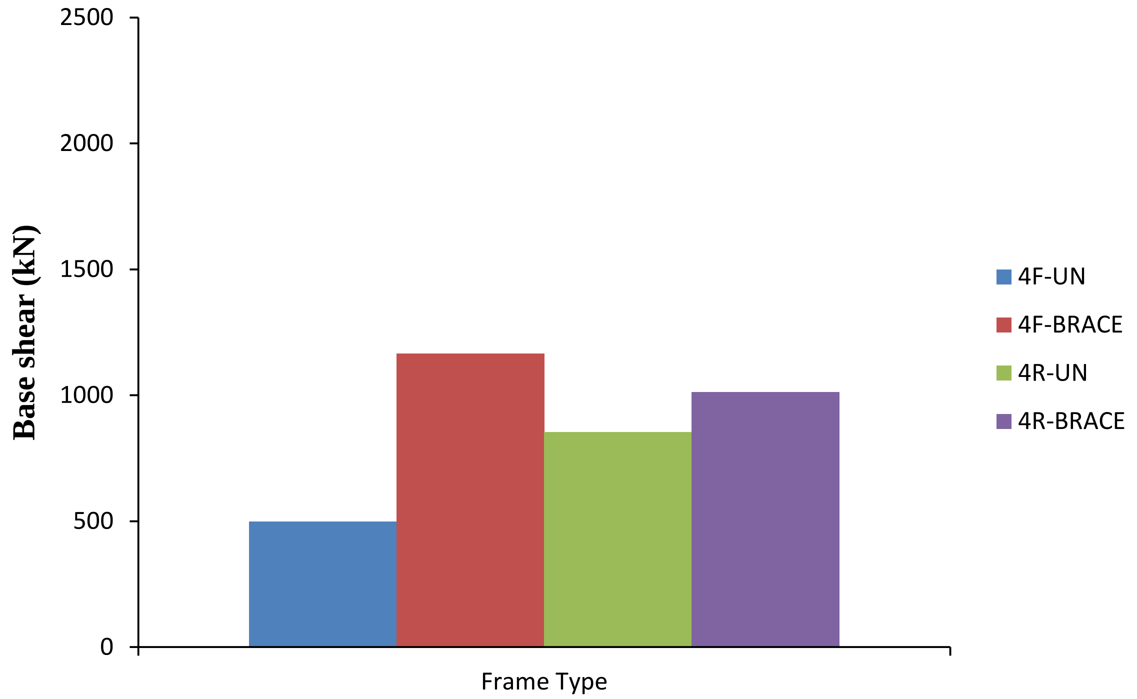
(e) Sixteen story building

Figure 4.3 Global damage indexes for MRFs

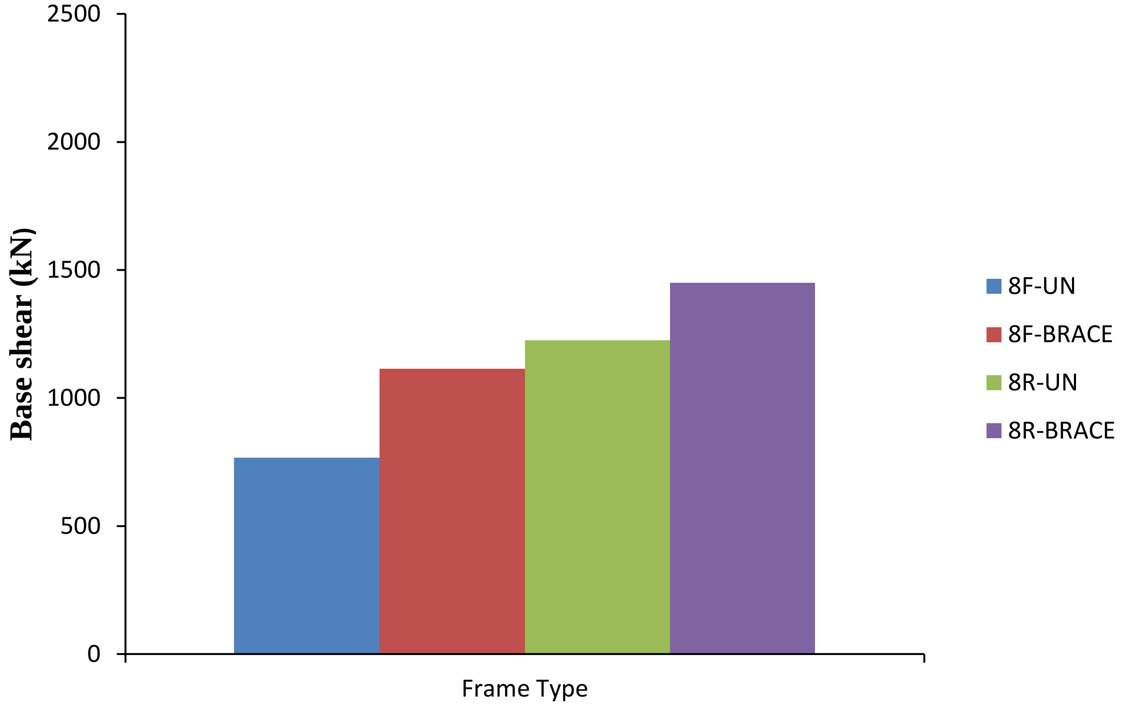
4.1.4 Total Base shear

The base shear at the target displacement of the pushover analysis was evaluated for both UF and CB frames. Figure 4.4 compares the base shear values for different frame type and story level. Strength of frame, peak ground acceleration, and earthquake type, site conditions affected the variety of base shear forces. Position of the fundamental period of the frame with respect to earthquake excitation acceleration spectrum defined this variation in the elastic stage.

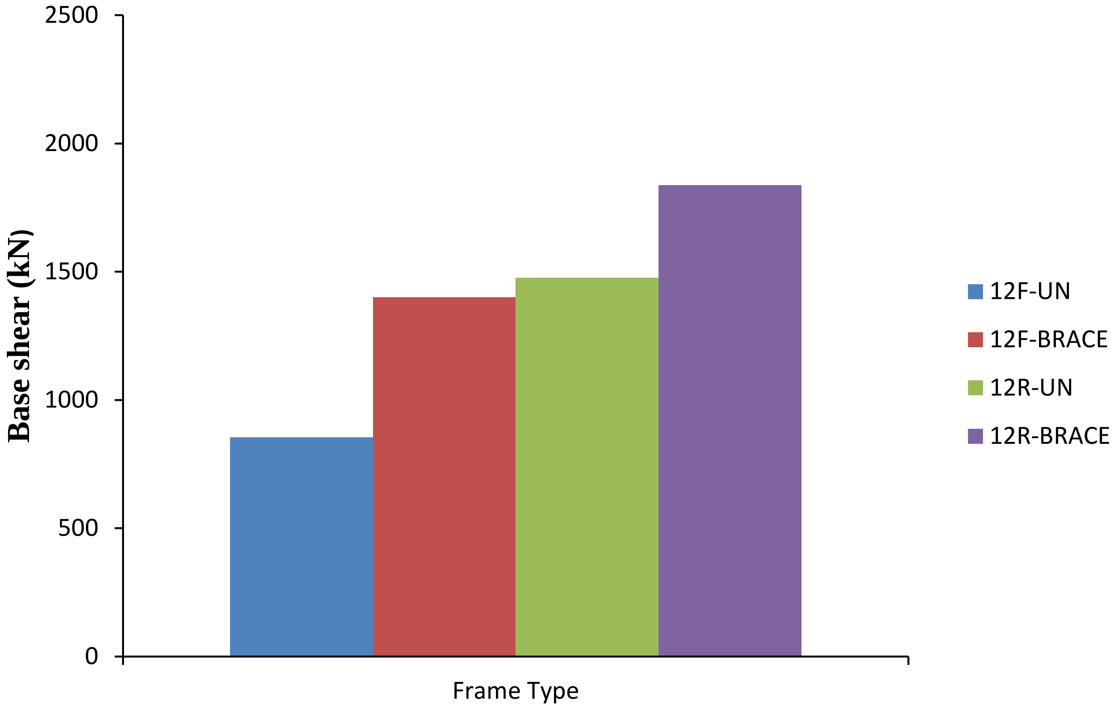
Total base shear was increased in the presence of braces, but the columns were not influenced so much by this increment, because most of the shear forces were supported by the braces. The base shear is also affect by type of frame and number of stories of the structure, for example, in the case of stiff CB frames the base shear is greater than flexible CB frames and by increasing number of stories the base shear is also increased. Thus, the largest base shear was observed in the case of sixteen stories stiff CB frames.



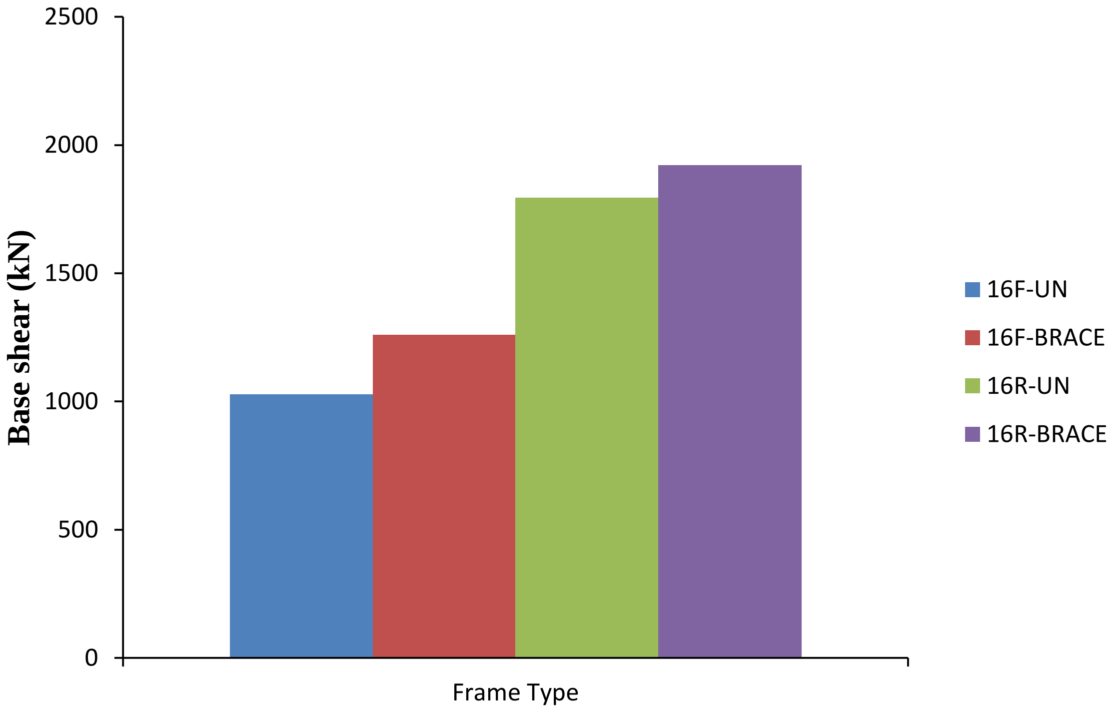
(a) Four story building



(b) Eight story building



(c) Twelve story building



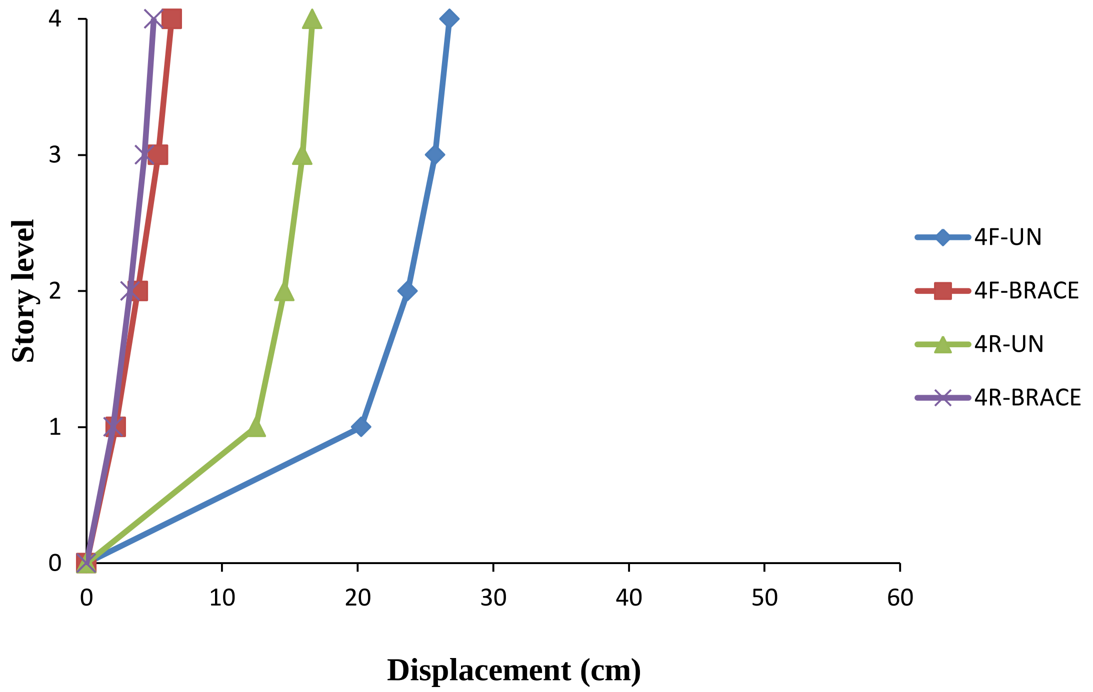
(d) Sixteen story building

Figure 4.4 Total base shears for different MRF

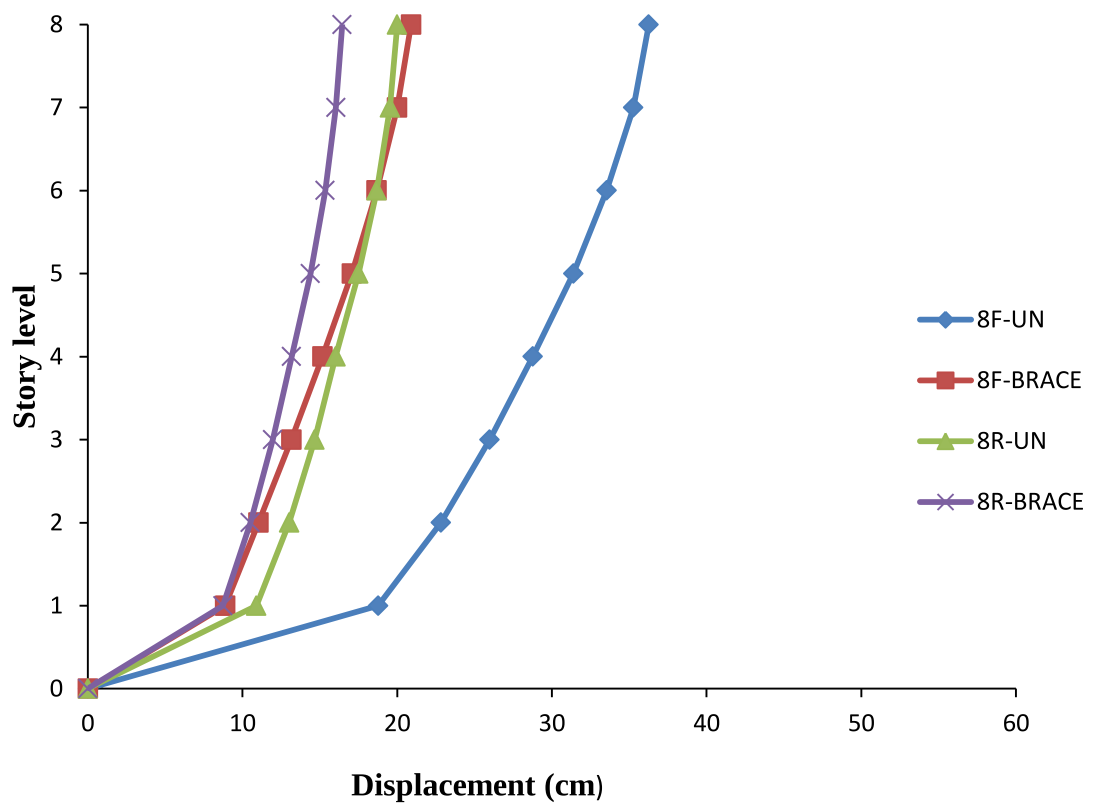
4.1.5 Variation of storey displacement

Figure 4.5 shows the deflected shape of UF and CB frames at various circumstances at the instance corresponding to the target roof displacement. The use of CB considerably decreased the value of maximum storey displacements compared to unbraced frames, especially in the case of rigid CB frames; more uniform response of the frame along the height of the structure could be observed and there was not concentration of large deformation in one storey or without an abrupt change in the drift pattern with respect to the level of deformation.

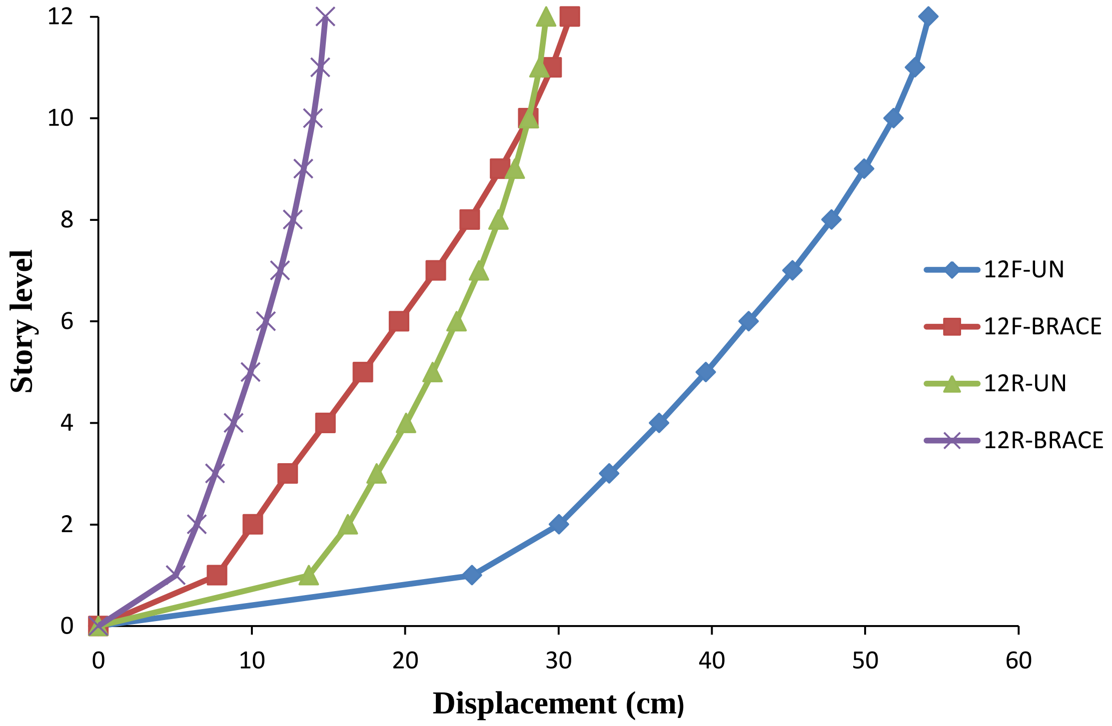
The maximum storey displacement is also influenced by the number of stories and frame type. For example, in the case of four stories stiff CB frames the maximum storey displacement is smaller than other frames, by increasing number of stories the maximum storey displacements are also increased.



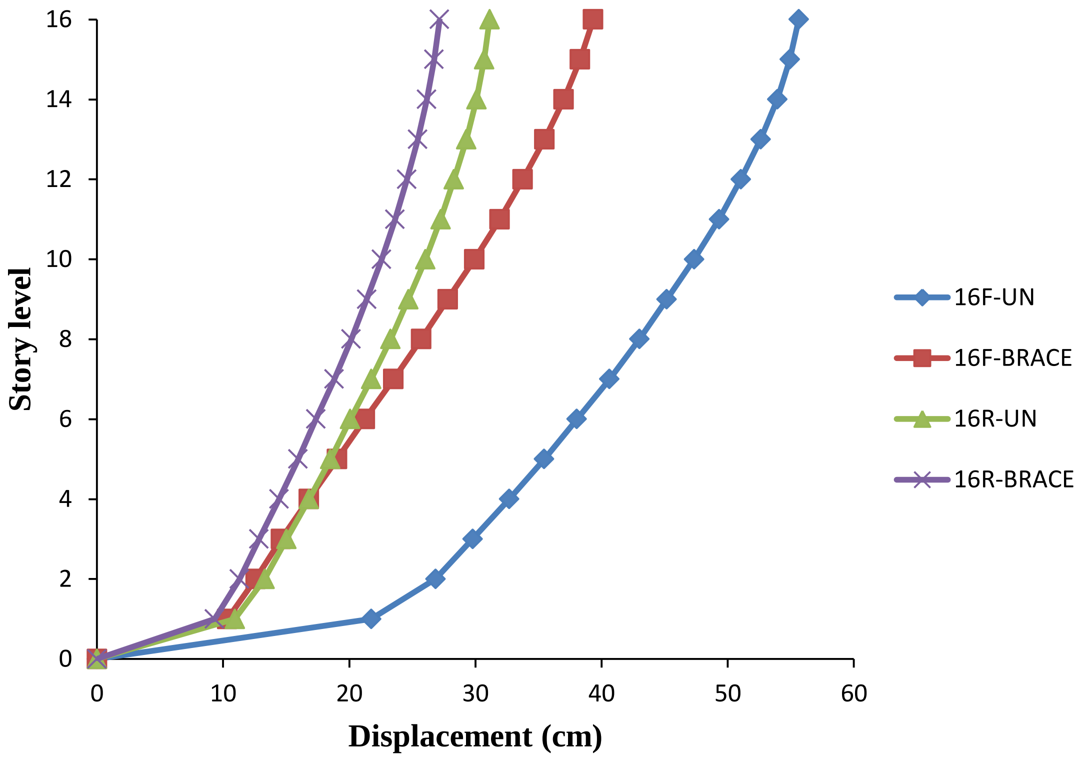
(a) Four story building



(b) Eight story building



(c) Twelve story building



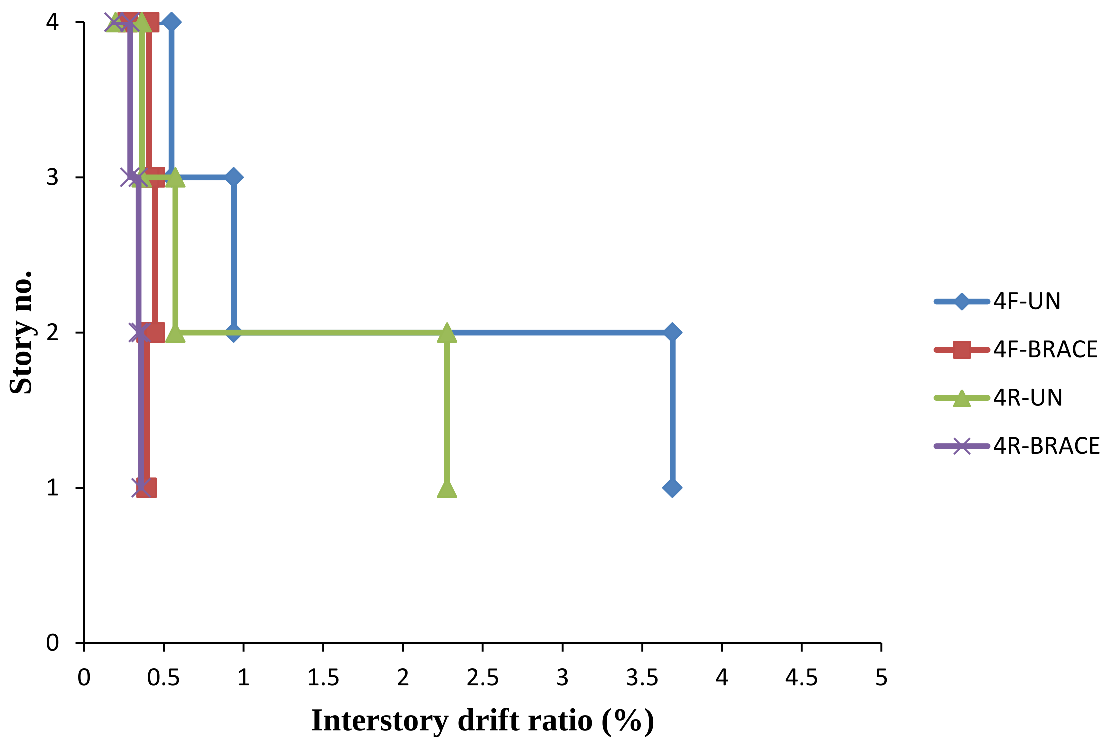
(d) Sixteen story building

Figure 4.5 Deflected of 4, 8, 12, and 16 storeys unbraced and braced flexible and rigid frames

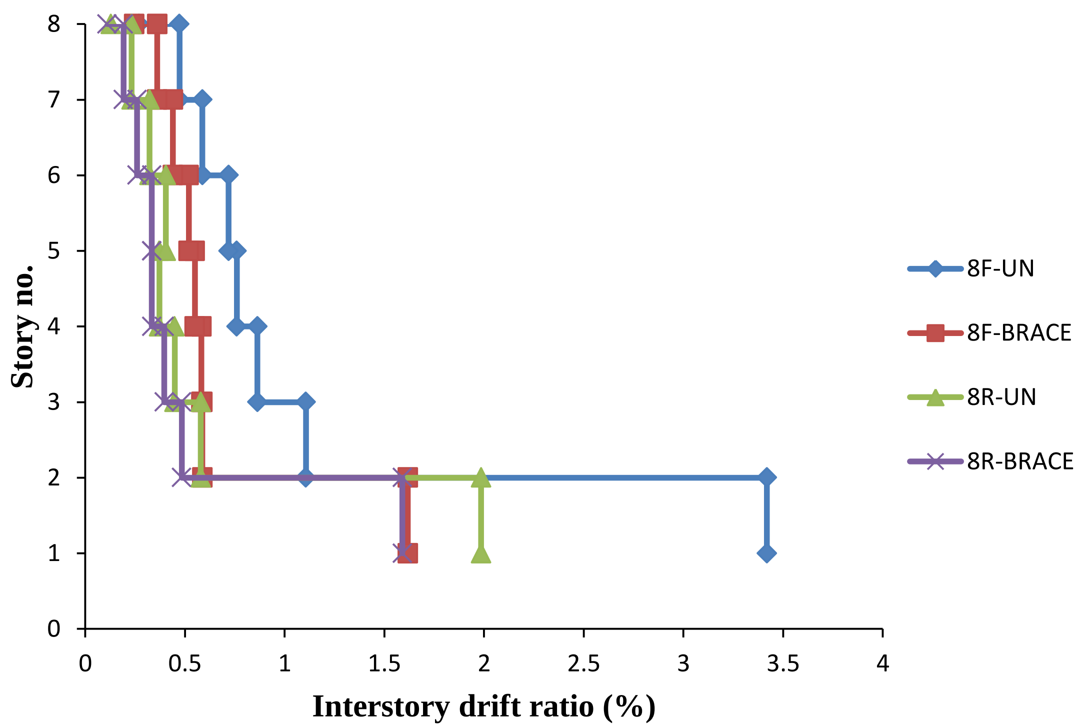
4.1.6 Inter-story drift ratio

Prior investigations have underlined the deed that steel buildings can skill significant lateral deformations following an earthquake ground motion (Pampanin et al., 2003; Garcia and Miranda, 2006). Therefore, Inter-story drift demands over height in the unbraced and braced frames are also evaluated as seen in Figure 4.6.

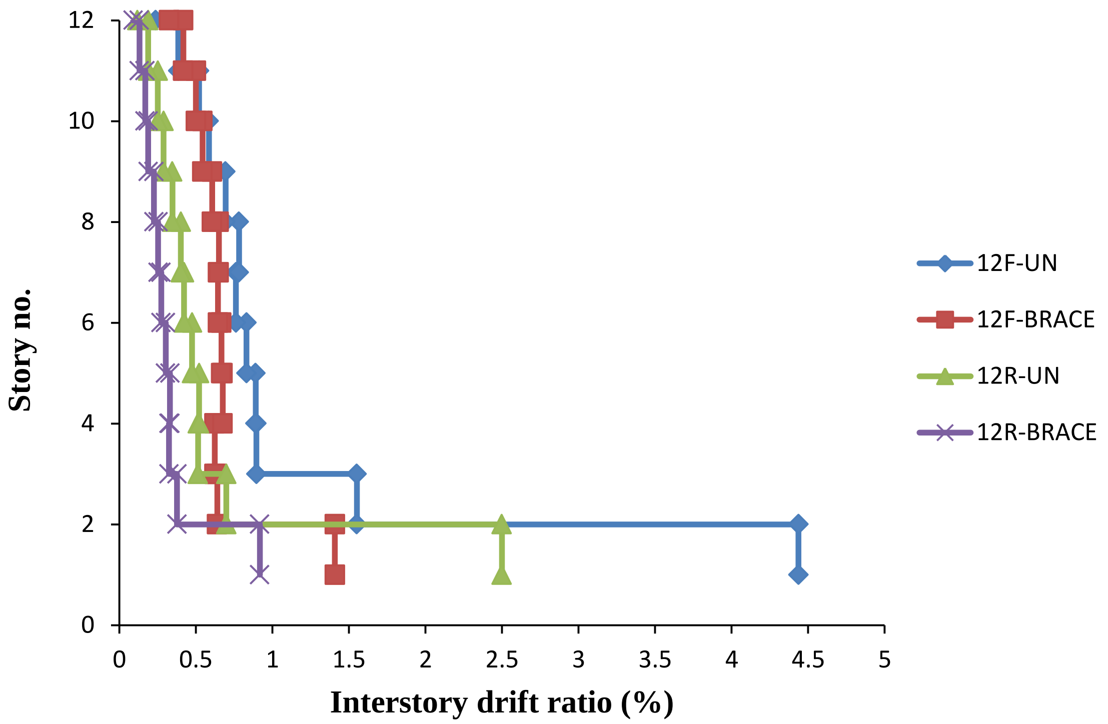
Generally, it can be noticed that the supplement of braces decreases considerably the drifts in the frames. For example, the drift demand complementary to the unbraced flexible eight-storey frame is 3.4% while the drift demands complementary to the eight-storey frames equipped with CBs are considerably smaller and have peak inter-story drift of 1.6%. Figure 4.6 shows that the use of stiff frames is better than flexible frames and in the case of CB frames, the stiff diagonally braced, storey drift demands are significantly smaller.



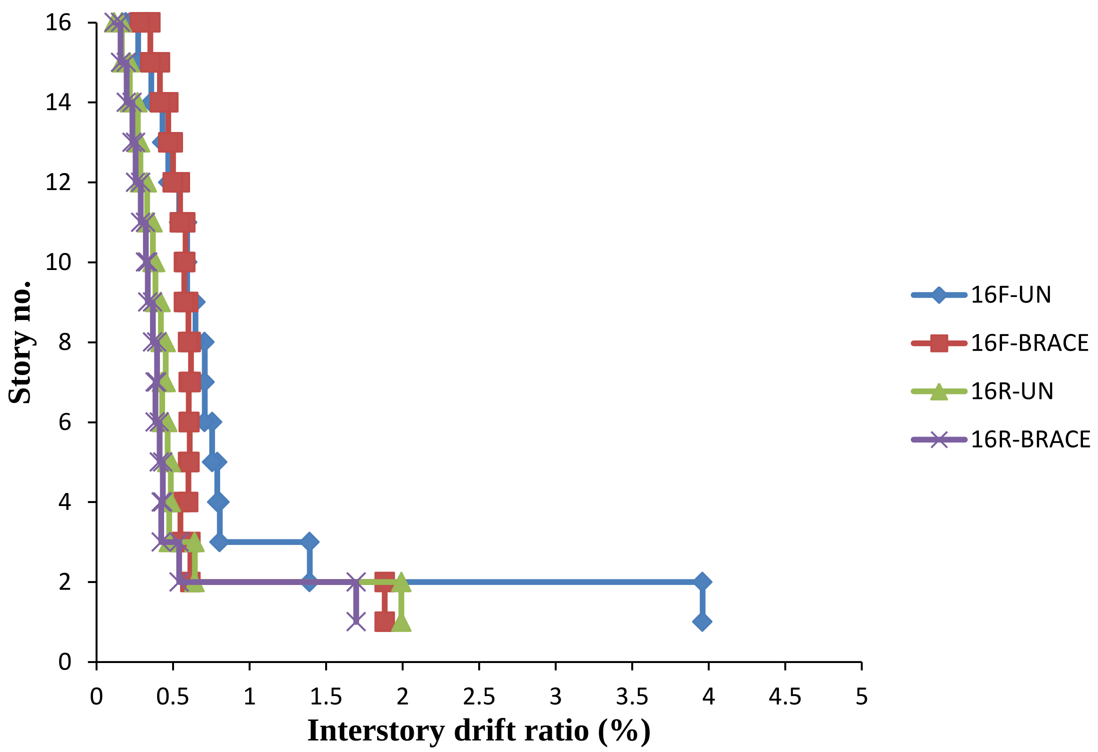
(a) Four story building



(b) Eight story building



(c) Twelve story building



(d) Sixteen story building

Figure 4.6 Interstorey drift ratio for different frames

CHAPTER 5

CONCLUSIONS

The study described herein investigated the structural performance of different type of moment resisting frame buildings and those equipped with concentrically diagonal braces (CBs) subjected to lateral loading. Based on the results of this study, the following conclusions can be drawn:

- Depending upon the design properties of the bare frames, CB frames provided smaller interstorey drift index compared to UFs. The results of the performed nonlinear static analysis indicated that CB frames were effective in diminishing drifts since the reduction of interstorey drifts with respect to the original frames was on average equal to 50%. Similarly, the use of CBs considerably reduced the global damage index of both flexible and especially rigid frames.

- From the capacity curves, it was observed that the base shear, which is the capacity of the frame to resist lateral loads, was considerably increased in the presence of braces. That is much more pronounced in the case of stiff braced frame systems.

- The target displacement was reduced significantly by using concentrically lateral bracing. Moreover, CB frames were much stiffer and showed a better performance compared to unbraced frames. Thus, the results of the performed nonlinear static analysis indicated that as the rigidity of the frame increased, smaller target displacement values were achived.

- Comparison of the performance of rigid MRFs in perspective with flexible MRFs, it was evident that the stiff systems were performing better than flexible ones. In the case of stiff ones, most of the plastic hinges were concentrated in the beam elements, and most of the columns remained in the elastic stage. However, in the case of flexible ones, many column elements entered the inelastic range of deformation that might result in an undesirable mode of failure.
- In these analyses, it was verified that this pushover methodology allows evaluating the performance of different structures through control of their displacements (at local and global levels), still giving additional information about the ductility and the resistant capacity of frames designed with different features.

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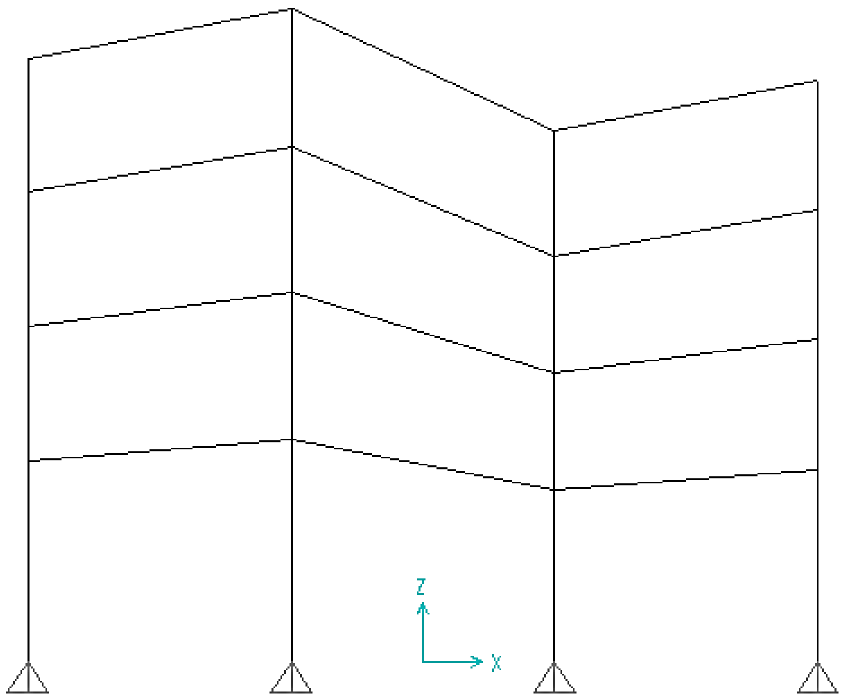
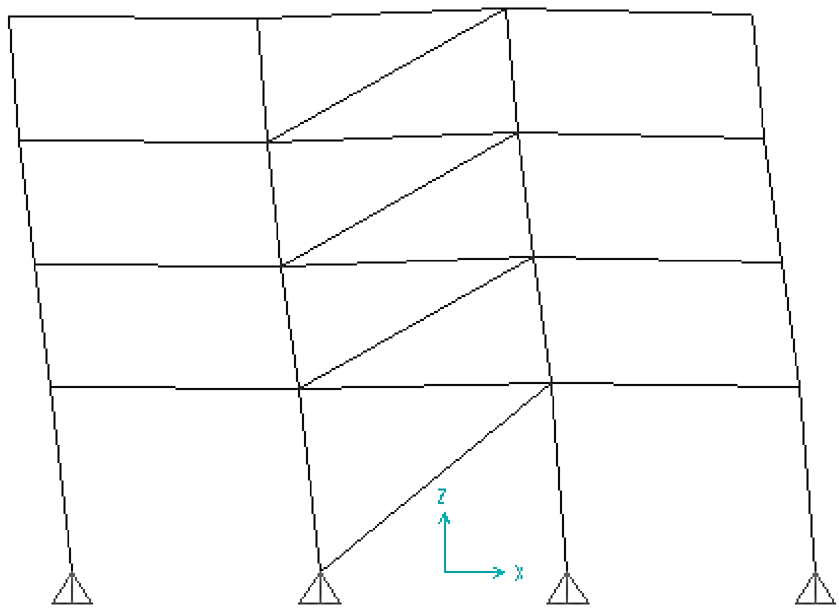
APPENDIX

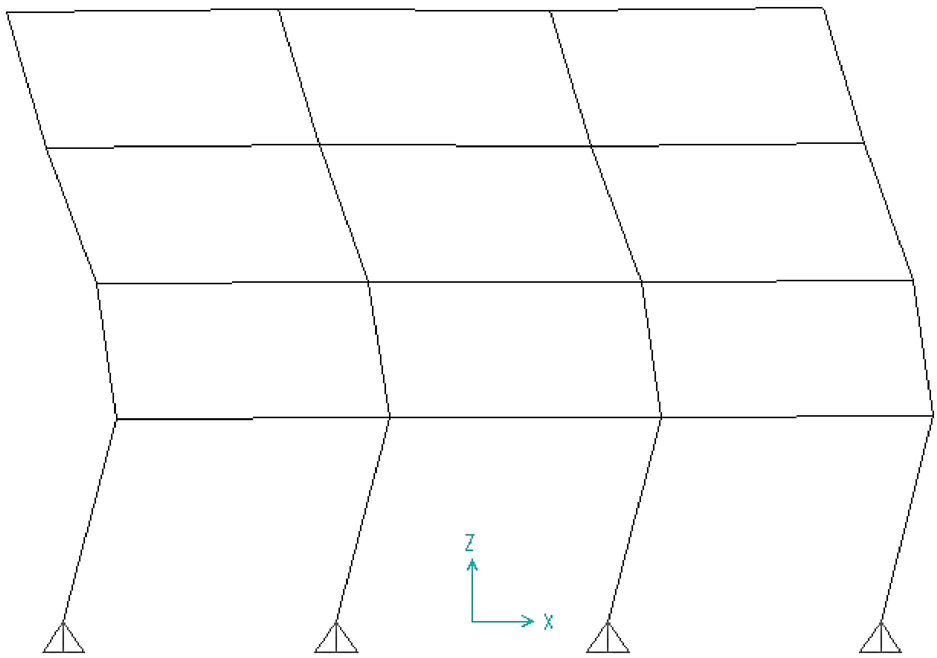


(a) at $T_{5} = 0.078$ s

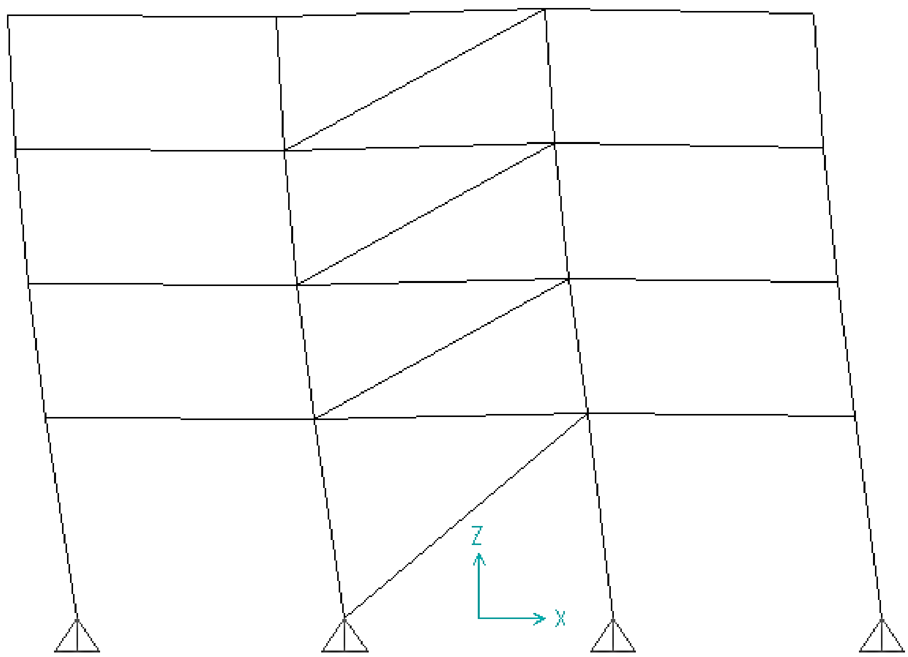


(b) at $T_1 = 0.374$ s

Figure A1 Deformed shape of four stories flexible unbraced and braced system at different period of vibration

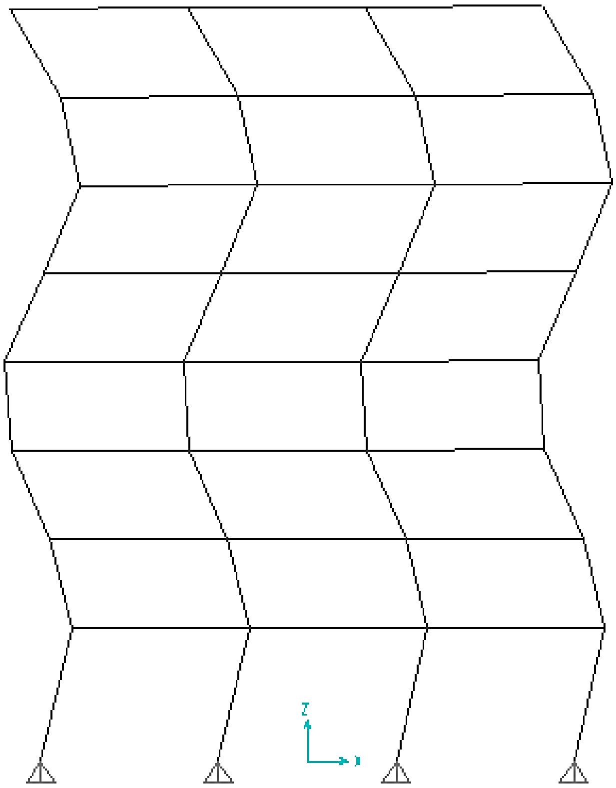


(a) at $T_2 = 0.1926$ s

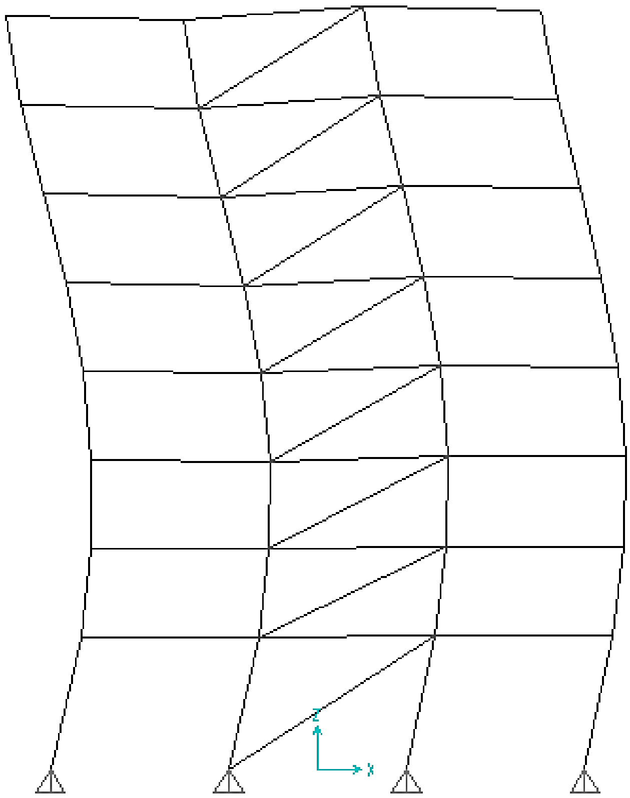


(b) at $T_1 = 0.269$ s

Figure A2 Deformed shape of four stories rigid unbraced and braced system at different period of vibration

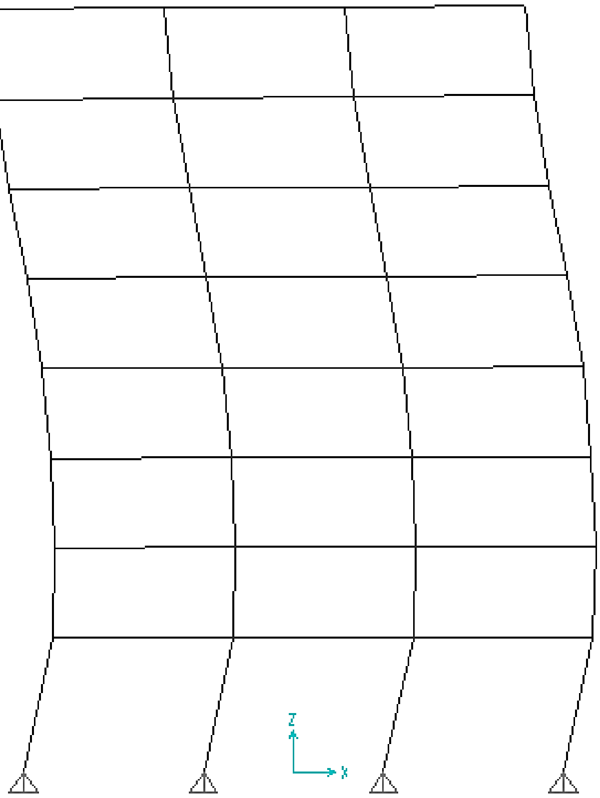


(a) at $T_4 = 0.213$ s

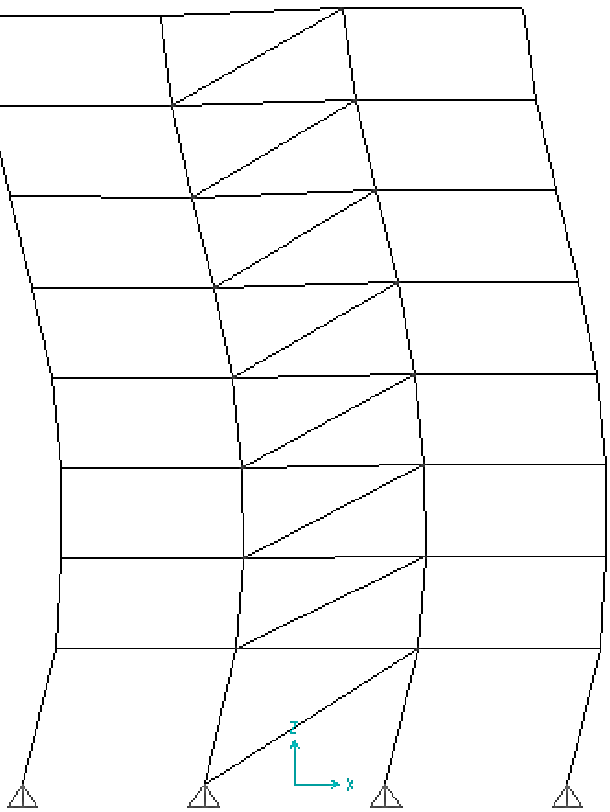


(b) at $T_2 = 0.327$ s

Figure A3 Deformed shape of eight stories flexible unbraced and braced system at different period of vibration

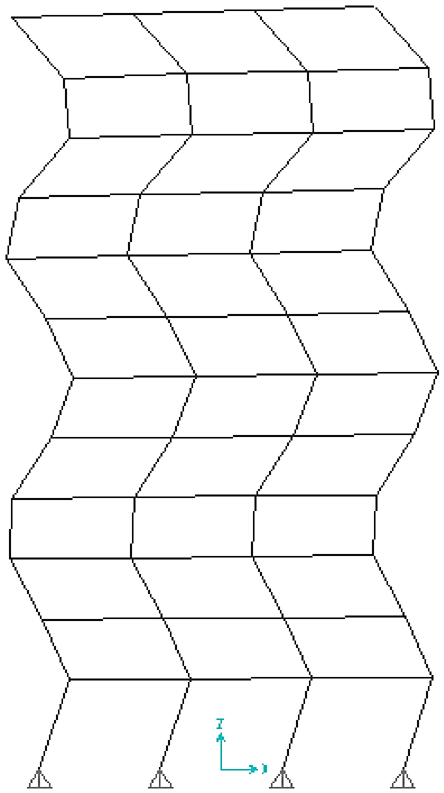


(a)at $T_2 = 0.349$ s

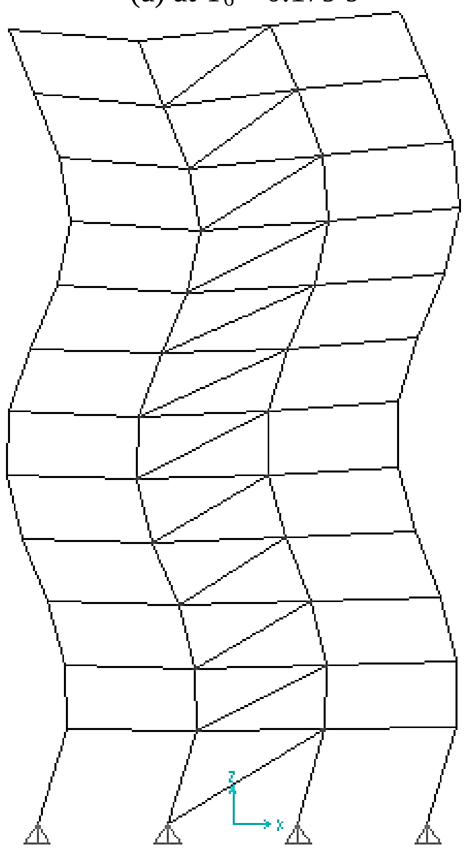


(b)at $T_2 = 0.235$ s

Figure A4 Deformed shape of eight stories rigid unbraced and braced system at different period of vibration

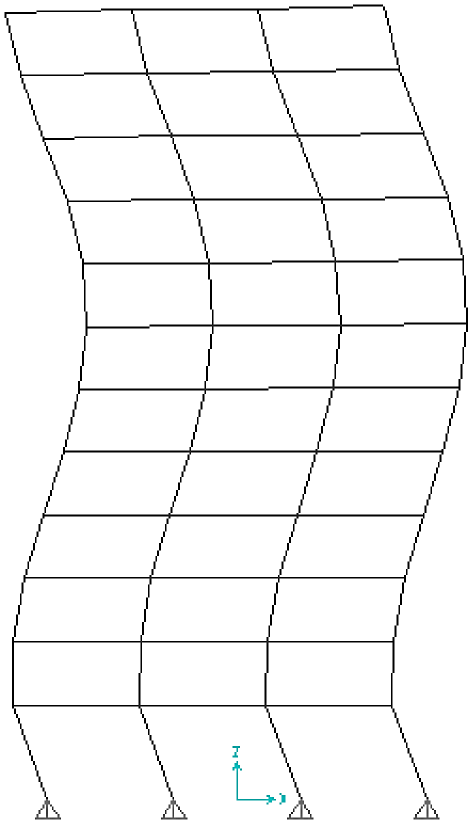


(a) at $T_6 = 0.179$ s

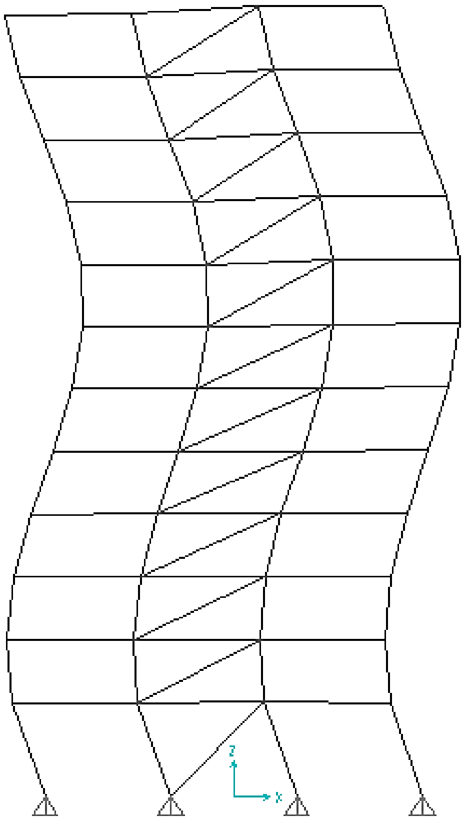


(b) at $T_4 = 0.173$ s

Figure A5 Deformed shape of twelve stories flexible unbraced and braced system at different period of vibration

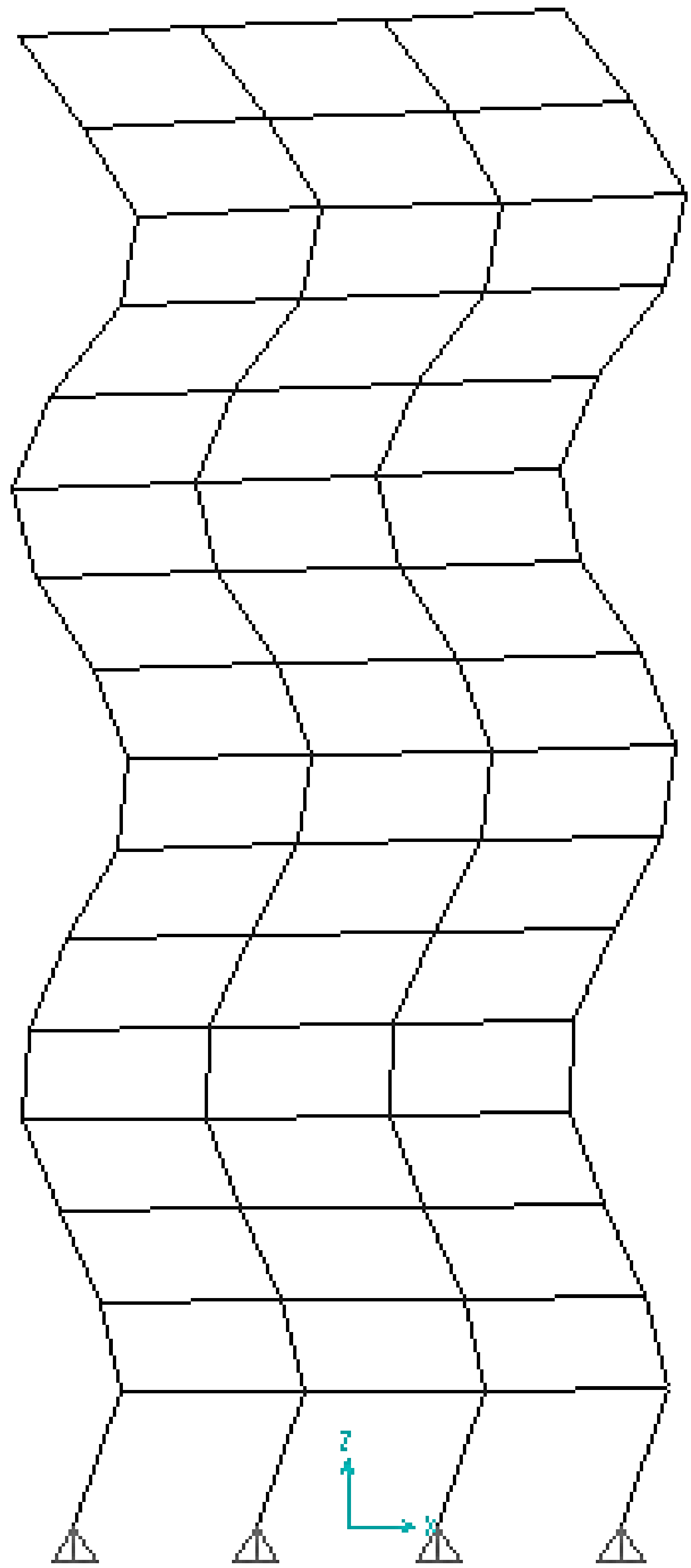


(a) at T_3 = 0.237 s

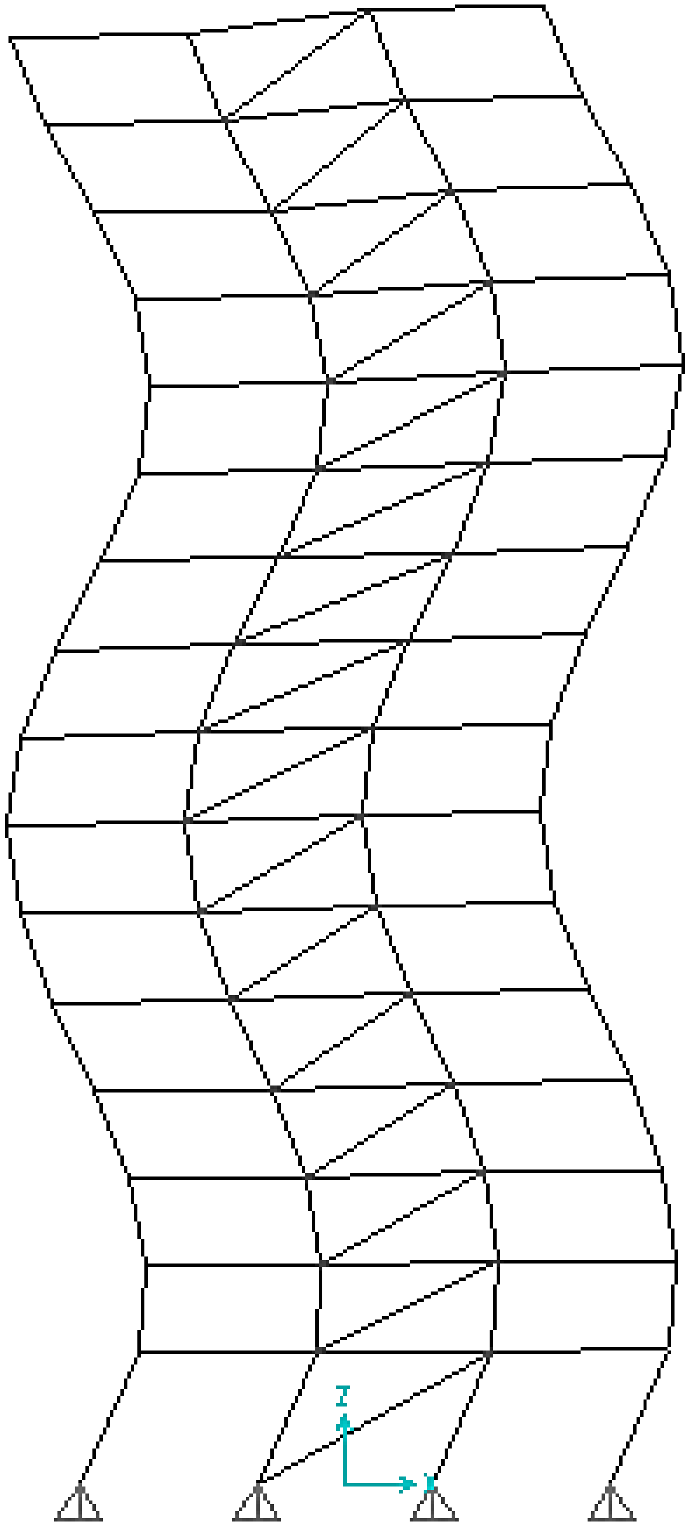


(b) at T_3 = 0.169 s

Figure A6 Deformed shape of twelve stories rigid unbraced and braced system at different period of vibration

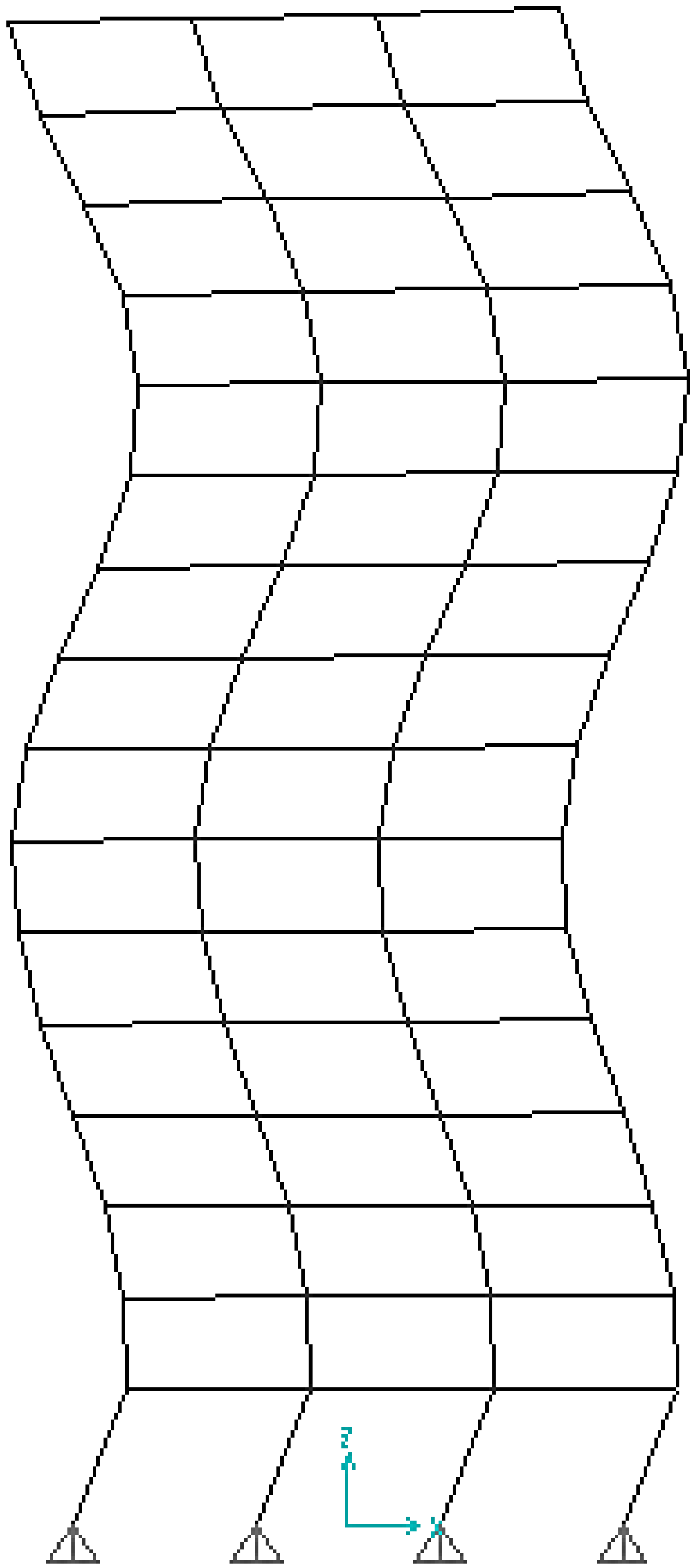


(a) at T_6 = 0.223 s

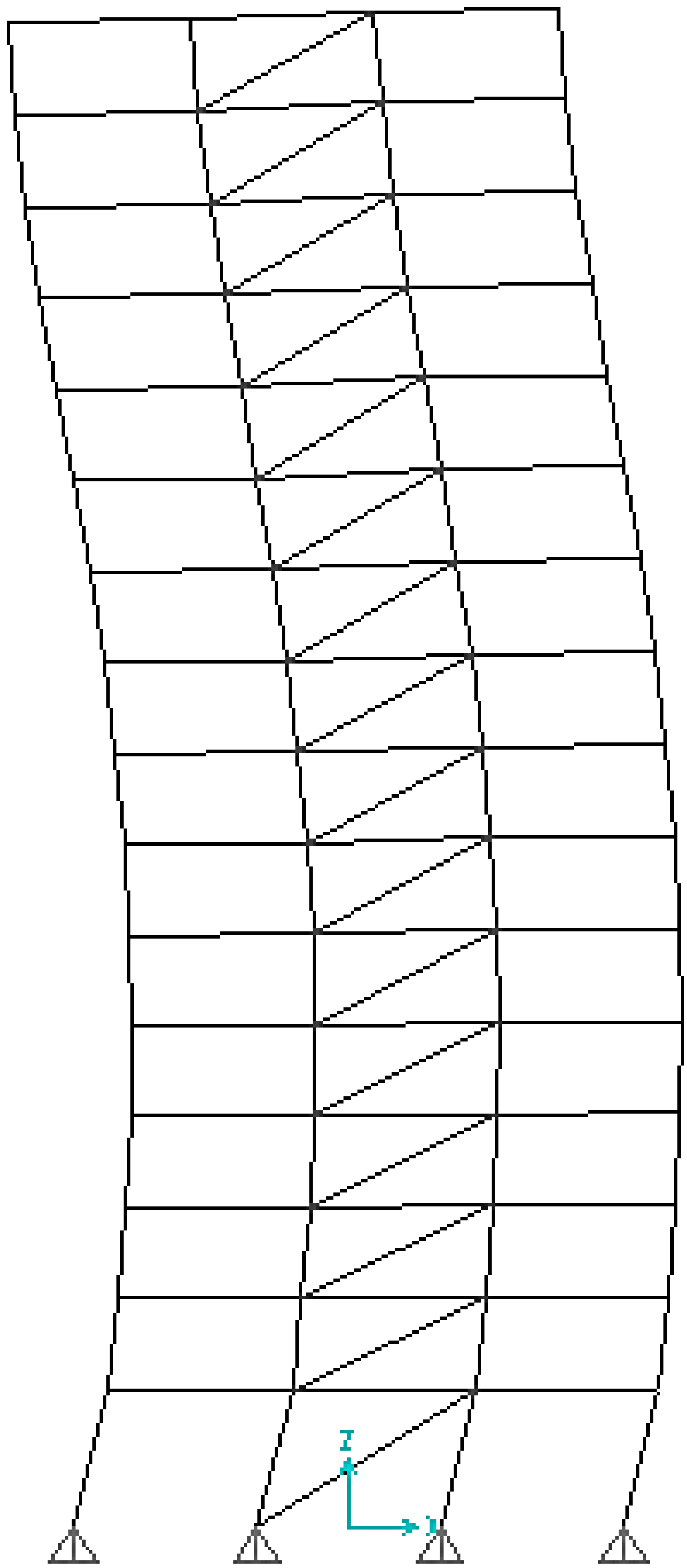


(b) at T_4= 0.242 s

Figure A7 Deformed shape of sixteen stories flexible unbraced and braced system at different period of vibration



(a) at $T_4 = 0.218$ s



(b) at $T_2 = 0.442$ s

Figure A8 Deformed shape of sixteen stories rigid unbraced and braced system at different period of vibration