UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

NONLINEAR PUSHOVER ANALYSIS OF MEDIUM AND HIGH RISE BUILDINGS WITH X-BRACES

M. Sc. THESIS
IN
CIVIL ENGINEERING

BY
RUSHDI ESA
APRIL 2013

Nonlinear Pushover Analysis of Medium and High Rise Buildings with X-Braces

M.Sc. Thesis
In
Civil Engineering
University of Gaziantep

Supervisor
Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By
Rushdi Esa
April 2013

T.C.
UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Nonlinear Pushover Analysis of Medium and High Rise Buildings with X-Braces

Name of the student: Rushdi ESA
Exam date: 30. April. 2013

Approval of the Graduate School of Natural and Applied Sciences

Assoc. Prof. Dr. Metin BEDIR
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Mustafa GÜNAL
Head of Department

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

Assoc. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members	Signature
Assoc. Prof. Dr. Esra METE GÜNEYİSİ	
Assoc. Prof. Dr. Mehmet GESOĞLU	
Prof. Dr. Mehmet TOPALBEKİROĞLU	

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Rushdi ESA

ABSTRACT

Nonlinear pushover analysis of medium and high rise buildings with X-braces

ESA, Rushdi Khalis
M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Esra METE GÜNEYİSİ

March 2013, 102 pages

This study aims to investigate the structural behavior of medium and high rise frame buildings with and without X-brace. The performance of the concentrically X-braced frame structures were examined using pushover analysis. For this purpose, steel moment resisting frame buildings having four to sixteen stories were considered. All buildings have the same plan and contain three bays on each direction. The existing steel frames had two different design properties and designated as flexible moment-resisting frames and rigid moment-resisting frames. Then, X-braces were introduced into the middle bay of each frame. The nonlinear analyses were performed using the finite element analysis program of SAP 2000. The parameters such as capacity curve, inter-storey drift index, global damage index, and base shear were evaluated for the frame systems. It was observed that the incorporation of concentric X-braces into the frame resulted in a significant enhancement in the structural behavior of the existing frames.

Keywords: Bracing elements; Moment-resisting frame; Pushover analysis; Steel structures; Structural performance.

ÖZET

Merkezi X-çaprazlı orta ve yüksek katlı yapıların doğrusal olmayan itme analizi

Esa, Rushdi Khalis
İnşaat Mühendisliği Yüksek Lisans
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
Nisan 2013, 102 sayfa

Bu çalışmada, X-çaprazlı ve çaprazsız orta ve yüksek katlı binaların yapısal davranışlarının araştırılması amaçlanmıştır. Mekezi X-çaprazlı çerçeve yapıların performansı statik itme analizi kullanılarak incelenmiştir. Bu amaçla, yükseklikleri dört ila onaltı kat arasında değişen, moment aktaran çelik yapılar araştırmada kullanılmıştır. Bütün binalar aynı plana sahip olup, her yönde üç açıklıktan oluşmaktadır. Mevcut çelik çerçeveler, esnek ve rigit moment aktaran çerçeveler olmak üzere iki farklı tasarım özelliğine sahiptir. Sonrasında, X-çaprazlar her bir çerçevenin orta aksına yerleştirilmiştir. SAP2000 programı kullanılarak, doğrusal olmayan analizler gerçekleştirilmiştir. Çerçeve sistemleri için kapasite eğrisi, katlar arası ötelenme, hasar indeksi ve taban kesme kuvveti parametreleri değerlendirilmiştir. X-çaprazların mevcut çerçevelerin yapısal davranışını önemli ölçüde iyileştirdiği gözlenmiştir.

Anahtar kelimeler: Çapraz elemanlar; Moment aktaran çerçeve; İtme analizi; Çelik yapılar; Yapısal performans.

ACKNOWLEDGEMENTS

This study was conducted under the supervision of supervisor Assoc. Prof. Dr. Esra METE GÜNEYİSİ. I would like to express my sincere appreciation for the support, guidance, and insights that she has provided me throughout the study. Firstly, began thanks to Allah for everything.

My special thanks to the committee members of Assoc. Prof. Dr. Mehmet GESOĞLU and Prof. Dr. Mehmet TOPALBEKİROĞLU.

I would like to express my deepest appreciation to my wife which she is very much generosity, help and support me and thank my relatives for their helping especially my father, brothers and sister, without their support, this research project would not have been possible.

And thanks to my cognate Dr. Nahro Karizchi when he helped and supported me in a difficult situation and to others.

And thanks to my manager Dr. Dilshad Kaka sor jafe to give me this chance to finish my master degree.

To my two flowers Naz and Ban which they are my light of the eye.

My sincere appreciation also extends to my friends and to those gave me the delight of smile. Finally, I hope to deeper thanks to everyone help me at this level and arrived me to this statement especially my father and deceased mother which they suffered every think to us. I hope from my Allah to be them in his mercy and shelter.

TABLE OF CONTENT

Page

LIST OF FIGURES

Page

LIST OF TABLES

Page

LIST OF SYMBOLS/ABBREVIATIONS

BF-BF	Braced frames in both horizontal direction
CP	Collapse prevention level
[C]	Damping matrix
D	Roof displacement
ESDOF	Equivalent single degree of freedom system
FF	Flexible frame
H	Total height of the building
h	Storey height
IO	Immediate occupancy level
K_e	Elastic stiffness
K^*	Elastic stiffness of the equivalent SDOF system
LS	Life safety level
MRF	Moment resisting frame
MDOF	Multi degree of freedom system
[M]	Mass matrix
RF	Rigid frame
SDOF	Single degree of freedom system
SMRF	Special moment resisting frame

T_{eq}	Initial period
UF-UF	Unbraced frames in two horizontal direction
UF-BF	Unbraced frames in one horizontal direction and braced frames in the other direction
U	Relative displacement vector
u^*	Reference displacement
$\ddot{u}_g$	Ground acceleration history
u_t	Roof Displacement
u_y	Yield displacement
V_b	Base Shear
δmax	Maximum inter-storey drift
α	Strain-hardening ratio
$\{\Phi\}$	Shape vector

CHAPTER 1

INTRODUCTION

1.1 Generals

The performance based design led to the development and use of methods based on nonlinear analysis, namely pushover analysis. This analysis is a nonlinear static analysis in which the magnitude of the structural load is increased according to a certain predefined pattern. In the process of increasing the magnitude of the loading, the weak connections, sections or collapse modes of the structure can be found (Braz-César and Barros, 2009).

The nonlinear static pushover analysis during the last decade has been gaining interest among the structural engineering society as an alternative mean of analysis. The assessment is based on the estimation of important structural parameters such as global and inter-storey drift, element deformations, and internal forces. To assess the structural performance by estimating the strength and deformation capacities using nonlinear analysis and comparing these capacities with the demands at the corresponding performance levels is the purpose of the pushover analysis. The geometrical nonlinearity and material inelasticity as well as the redistribution of internal forces accounts in the analysis (López, 2004).

Antoniou and Pinho (2004a, 2004b) proposed an appealing adaptive pushover method. In this adaptive pushover technique, the lateral load distribution was continuously updated and not kept constant during the process. Modal shapes and participation factors were derived by eigenvalue analysis carried out at each analysis step. Two variants of the method currently exist: Force-based adaptive pushover (FAP) and Displacement-based adaptive pushover (DAP). The method was multimodal and accounted for softening of the structure, its period elongation, and the modification of the inertial forces due to spectral amplification. Moreover, of the two variants, the latter proved to provide results very similar to those obtained from the time history analysis when the response of seismically design buildings was analyzed (Antoniou and Pinho, 2004b).

In the study of Kadid and Boumrkik (2008), a series of incremental static analysis basically, a pushover analysis was carried out to develop a capacity curve for the building. Based on the target displacement, capacity curve, which was an estimation of the displacement that the design earthquake produced in the building was determined. Target displacement was the extent of damage experienced by the structure was considered when subjected to design level ground shaking. The application of pushover analysis was also illustrated in the other studies (Moghadam and Tso, 1998, 2000; Saiidi and Sözen, 1981).

Modal pushover analysis (MPA) proposed by Chopra and Goel (2001) improved pushover procedure by including the contributions of higher modes. To demonstrate the accuracy of seismic demand estimation in taller moment-frame buildings, the MPA procedure was applied to 9- and 12-stories buildings and compared with the conventional pushover analysis. In spite of including the contribution of higher modes, the MPA was conceptually no more difficult than standard procedures since higher modes for pushover analyses were similar to the first mode of pushover analysis. Moreover, for the first few (two or three) modes which considering the MPA procedure contribution were typically sufficient (Chopra and Goel, 2001). Another study conducted by Moghaddam and Hajirasouliha (2006) investigated the accuracy of pushover analysis when seismic demands were needed to be estimated for braced steel frames. Three steel braced frames of 5-, 10-, and 15- storeys were considered. Three different load patterns were used; the first mode distribution, the uniform distribution, and the inverted triangular distribution. The results showed significant sensitivity to the choice of the load patterns for all the structures.

1.2 Objective and scope

In this study, the structural response of different types of moment resisting frame buildings with and without concentric X-braces subjected to lateral loading were investigated through nonlinear static analysis. For this, a total of 16 different cases (covering unbraced and braced frames) were taken into consideration. As a result, their structural performances were determined in terms of inter-storey index, global damage index, total base shear, inter-storey drift ratio, and variation in storey displacement.

1.3 Outline of the thesis

Chapter 1-Introduction: is devoted to the presentation of the research topic and the identification of the general scope and specific objectives. Knowledge and objectives of the thesis were introduced.

Chapter 2-Literature review: This chapter traces the background on practical application and previous studies on pushover analysis, types of frames, and braced frames.

Chapter 3-Methodology: In this chapter, type of the frame systems and rehabilitation strategies that is used in this study is presented and the type of analysis procedures that has been carried out with the assumptions for modelling is discussed.

Chapter 4-Results and discussion: This chapter presents and discusses the results obtained from nonlinear static analysis (or pushover analysis) for assessing the structural performance of each frame system and the effect of concentrically X- brace on each type of frames.

Chapter 5-Conclusions: General conclusions are drawn regarding the overall results from all chapters.

CHAPTER 2

LITERATURE REVIEW

2.1 Backgrounds of pushover analysis

After many earthquakes happened in several places of the world such as the 1994 Northridge (California), 1995 Kobe (Japan), and 1999 Chi-Chi (Taiwan), the damage observed in structures (e.g. see in Figures 2.1 and 2.2) led several researchers to improve the procedure of analysis and retrofit the structures which have not got sufficient seismic performance. The nonlinear static (pushover) analysis as a new methodology, has become a popular tool during the last decade for the assessment of such buildings (Pecker, 2007).

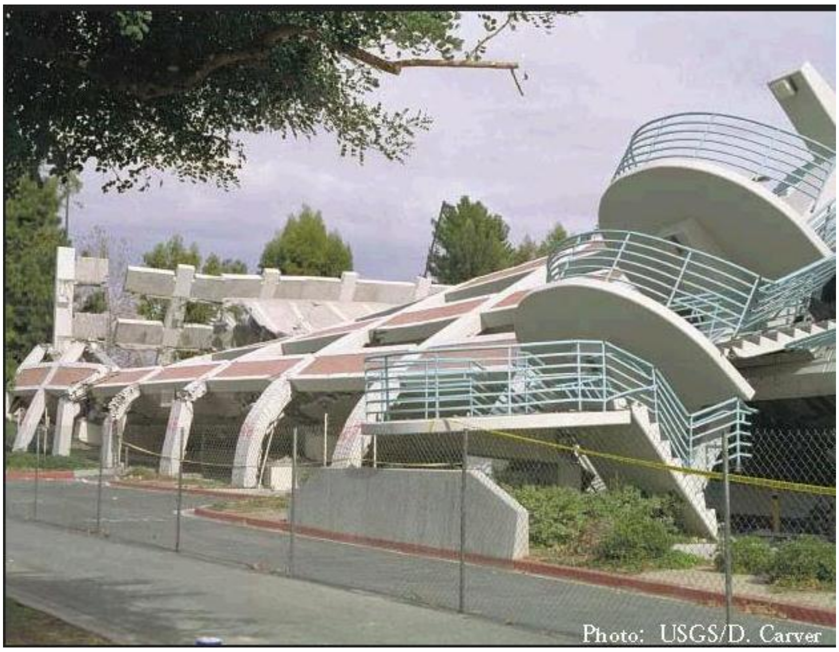


Figure 2.1 1994 Northridge parking structure (California)
(www.vibrationdata.com/earthquakes/northridge.htm)

Figure 2.2 Scenes of damaged structures, Kobe (Japan) 1995
(www.slideshare.net/IBGeogIST/2004-indian-ocean-tsunami-2109248)

Earthquake engineering is a profession of civil engineering that deals with the mitigation of earthquake induced damage on structures and the minimization of loss of life. During the last forty years, this field has advanced considerably due to the rapid developments of computers and computing, the improved experimental facilities, and the development of new methods of seismic design and assessment of structures. This advancement has not been enough to resist the catastrophic consequences that earthquakes impose. However, it has led to some improvement of design and assessment procedures with a shift from traditional force-based procedures to displacement-based procedures by Antoniou (2002), as inelastic displacements have been deemed to be more representative of different structural performance levels. However, it is still difficult to physically divide these procedures from each other, since the forces and displacements are strongly related to each other. Nevertheless, the characterization of the various performance levels has led to performance-based earthquake engineering, the most recent path of seismic design and assessment. The procedures recommended for seismic design and assessment purposes were briefly described, and their shortcomings were addressed. The theoretical background of the nonlinear static ‘pushover’ analysis method was then described together with the various pushover analysis procedures (Themelis, 2008).

Several attempts at adapting the force distribution within the state of inelasticity were done. The first attempt to improve the conventional static pushover procedure was proposed in the study of Sasaki et al. (1998). In this study, several pushover analyses were performed for the same structure with force patterns representing the various modes of the structure. A combination of these separate solutions was then studied to assess the overall structural response, as seen in Figure 2.3.

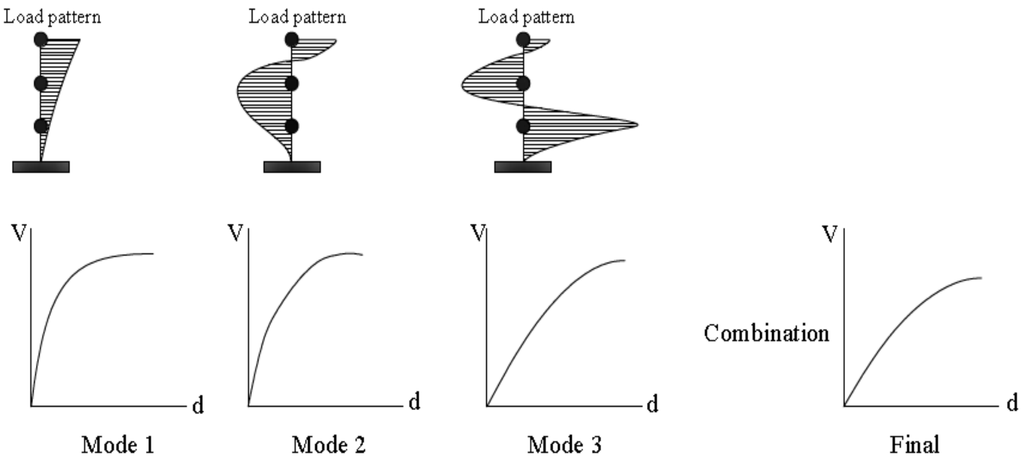


Figure 2.3 Load pattern (Sasaki et al., 1998)

Newmark and Hall (1982) and Miranda (2000) estimated applying certain displacement modification factors in which procedures were expressed on the basis of displacement modification factors where the maximum inelastic displacement demand of multi-degree of freedom system (MDOF) was defined considering maximum deformation of the equivalent elastic single degree of freedom system (SDOF) having the same lateral stiffness and damping coefficient as that of MDOF system. The procedures of Newmark and Hall (1982) proposed displacement modification factor varies depending on the spectral region based on the estimation of inelastic response spectra from elastic response spectra. Moreover, a statistical analysis of ratio of maximum inelastic to maximum elastic displacements conducted by Miranda (2000) and computed that from ground motions recorded on firm soils and proposed a simplified expression, which depended on ductility and initial vibration period.

Similarly, FEMA-356 (2000) described displacement coefficient method. This is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of the nonlinear MDOF system was obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients.

In the study of Teran-Gilmore and Ruiz-Garcia (2011), they presented an analytical study for evaluating the feasibility of using buckling-restrained braces as a retrofit scheme. As existing structures, multi-bay multi-story steel building with 4 and 8 stories were taken into consideration. They were first designed by Santa-Ana and Miranda (2000). For that purpose, the seismic response of four two-dimensional frame models representative of typical steel buildings designed in a region of high seismicity was analyzed prior to and after including buckling-restrained braces as a retrofit strategy. The four- and eight-story flexible frame exhibited much larger inter-storey drifts and also developed soft stories. They observed that a rigid building performance was better than flexible building and it was better to resist lateral load by using buckling restrained brace in each building.

Moghadam and Tso (2000) discussed about different types of buildings and they investigated the efficiency of pushover analysis, three systems were studied. The first model was a ductile moment resisting frame building and the second model was a set-back building and the last one was a wall-frame structure. Ten spectrum compatible time history records as ground motion excitations at the base were subjected to each building. The procedure used an elastic spectrum analysis of the building to obtain the target displacements and load distributions for pushover analyses. The means of the maximum responses of these buildings were computed using three-dimensional inelastic dynamic analyses and the proposed procedure. A comparison of the two sets of results demonstrated the capabilities and limitations of the proposed procedure.

Papanikolaou (2000) studied about an adaptive pushover analysis which took into account mode interaction and spectral amplifications. According to his study, the lateral load pattern was not kept constant during the analysis but it was continuously updated based on a combination of the instantaneous mode shapes and/or spectral amplifications corresponding to the inelastic periods of the structure. Figure 2.4 shows the model combination used in such pushover method.

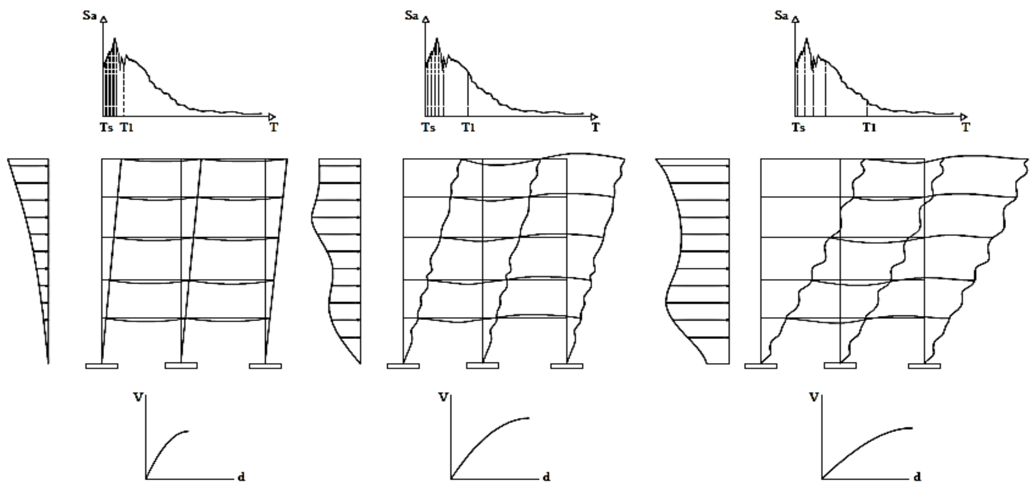


Figure 2.4 Adaptive pushover using modal combinations (Papanikolaou, 2000)

These mode shapes were determined by performing eigenvalue analysis (using a Lanczos eigenvalue solver) operating on the current tangent stiffness matrix ($\mathbf{K}_T$) and the spectral amplifications were calculated by numerical integration of the strong-motion record. A flow chart of the procedure is shown in Figure 2.5 (Papanikolaou, 2000).

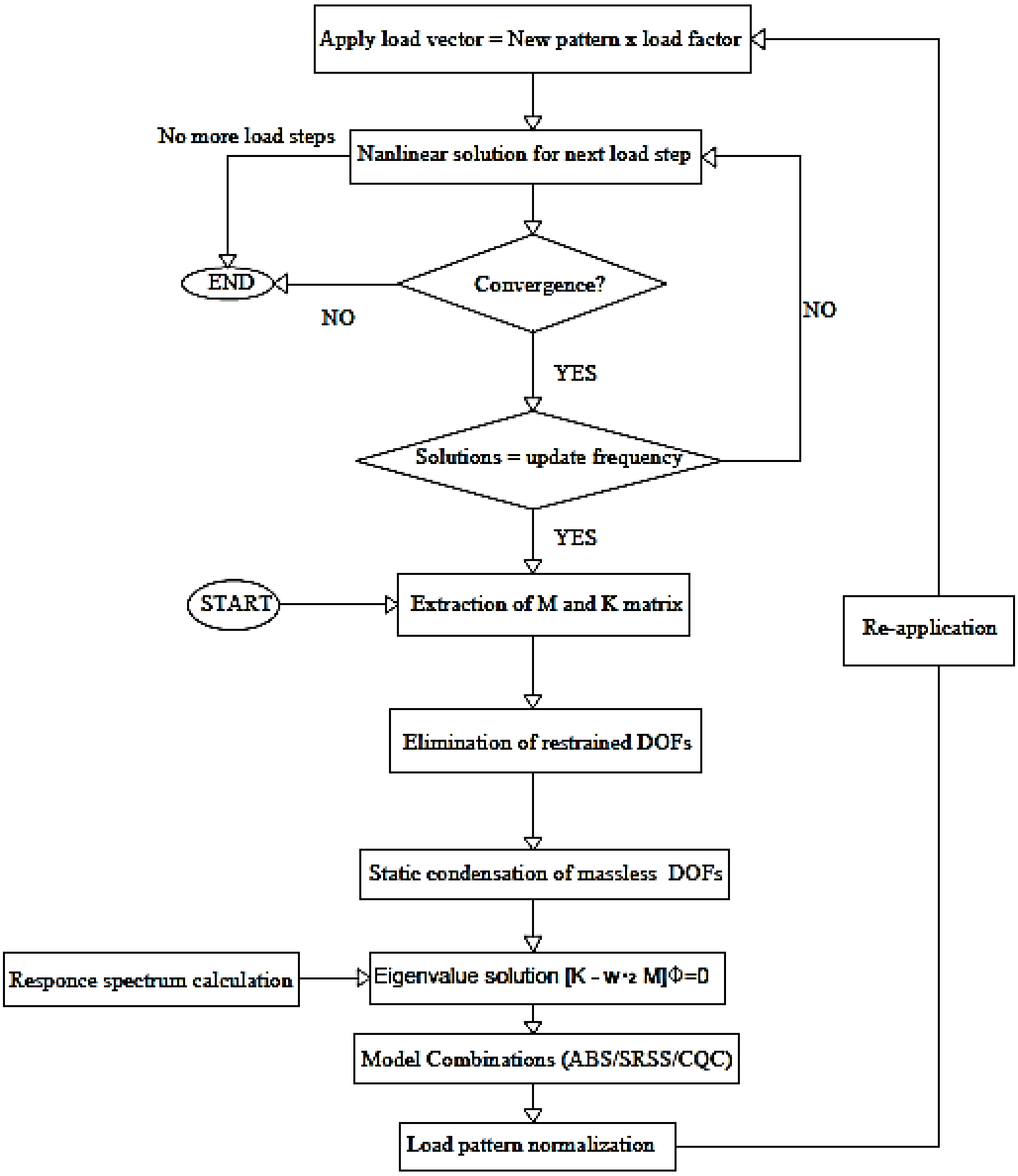


Figure 2.5 Flow chart of the adaptive pushover method (Papanikolaou, 2000)

Braz-César and Barros (2009) presented a pushover analysis on the seismic performance of metallic braced frames equipped with K-bracing and diagonal X-bracing systems. MIDAS/Civil finite element software was used in the analyses of nonlinear static methods. The frames were modelled to obtain the pushover curve. Three steel frames corresponding to 3, 6 and 10 stories buildings were considered. The principal objective was to compare the evaluation of the structural performance of these buildings with respect to the proposed N2-method. Consequently, the study show the convenience of using pushover methodology for the seismic analysis of structures.

Fajfar and Fischinger (1988) set out the N2 method as a variation of pushover The convenience of this method was checked with the experimented results obtained from a seven-storey reinforced concrete (RC) frame-wall building tested in Tsukuba, Japan as part of U.S. Japan joint research project. In addition, the other researchers used the uniform and inverted triangular load distributions to perform nonlinear static analyses of the structure. Considerable differences in the shapes of pushover curves were seen the analytical results were compared with experimental results. It was worthy noting that when the inverted triangular distribution was un-conservative in estimating base shear demands due to the effect of higher modes. Moreover, in general, non-conservative shear forces in the nonlinear dynamic analysis of the equivalent SDOF system from the comparison of theoretical and experimental results were observed. However, the target displacement at the ultimate limit state and the rotations of the floors approximated satisfactorily when compared with those results. was also found that the uniform distribution seemed more rational when shear demand was to be assessed (Kabeyasawa et al., 1983; Okamoto et al., 1984; Bertero al., 1984).

A study conducted by İnel et al. (2003) showed the accuracy of various lateral load patterns used in the pushover analysis procedures. Inverted triangular, rectangular, "code" were the first mode; adaptive lateral load patterns and multimode pushover analysis studied. Lateral load patterns were performed on four buildings designed as part of the SAC joint venture (FEMA-355C, 2000) and modified versions of these buildings with a weak first story consisting of 3- and 9-story regular steel moment resisting frames. The results obtained from pushover analyses were peak values of story displacement, story shear, inter story drift, and overturning moment at different values of peak roof drifts representing elastic and various degrees of nonlinear response. Those results were compared with those obtained from nonlinear dynamic analysis. A total of 11 ground motion records was selected from a Pacific Earthquake Research Center (PEER) strong motion database, and those were used in nonlinear dynamic analyses. The trends in the accuracy of load patterns were approximate bounds of error for each lateral load pattern with respect to mean dynamic response. To provide very good estimates of peak displacement response, both regular and weak-story buildings were analyzed using the simplified inelastic procedures. However, the use of the multiple modes generally improved the estimation of inter story drift, story shear, and overturning moment. The results also indicated that in the first mode lateral load pattern could be made without an appreciable loss of accuracy.

Ghiasvand (2012) investigated the inelastic behavior of structures with steel concentric braced frames (CBFs) for two, four, and eight story models by pushover analysis using the SAP 2000 software. The results indicated that with special plans like changing the dimensions of sections and modifying the process of forming plastic hinges in structure, the inelastic behavior of the structure could be improved significantly. The main purpose of the nonlinear static procedure was focused on effective nonlinear parameters in analysis and following many advantages that there were not in linear analysis such as: exact evaluating of force, deformability in structural and non-structural elements, deterioration of strength, critical areas of structure, and route of force. The third degree indeterminate frame under the effect of loading is also illustrated in Figure 2.6.

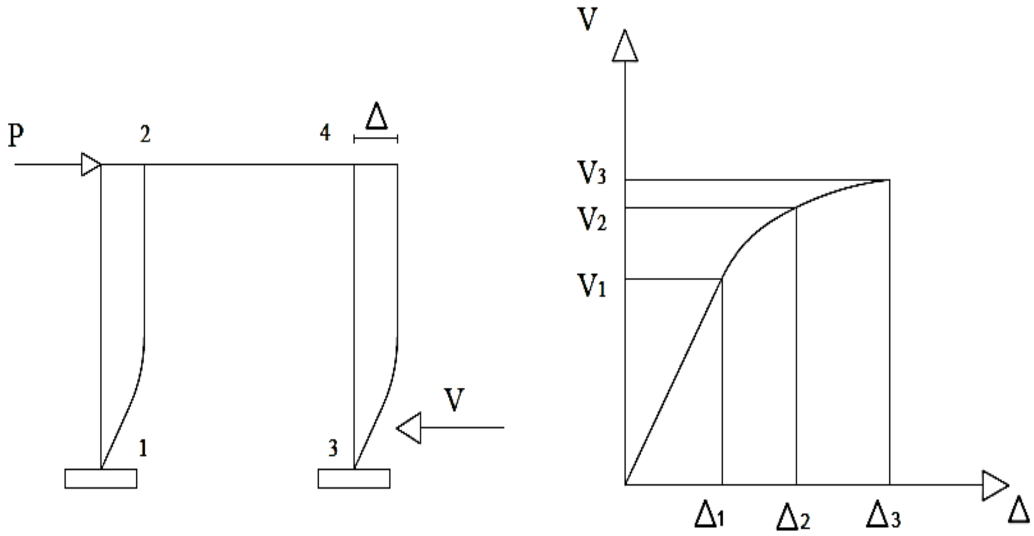


Figure 2.6 Three-degree indeterminate frame subjected to an increasing load (Ghiasvand, 2012)

Based on the structural dynamic's theory named as modal pushover analysis (MPA), an improved pushover analysis procedure was developed Chopra and Goel (2001). Firstly, the procedure was applied to linearly elastic buildings, and it was shown that the procedure was equivalent to the well-known response spectrum analysis. Then, to estimate the seismic demands of inelastic systems, the procedure was extended by describing the assumptions and approximations involved. Earthquake induced for a 9-story SAC building was determined by the MPA, pushover analysis, and nonlinear dynamic analysis using uniform, "code", and multi-modal load patterns. comparison of results demonstrated that pushover analysis for all load patterns underestimated the story drift demands and led to large errors in the plastic hinge rotations. The MPA results were also shown to be weakly dependent on ground intensity based upon the results obtained from El Centro ground motion scaled by factors varying from 0.25 to 3.0. It was including the contributions of a sufficient number of modes (two or three); the height-wise distribution of responses estimated the MPA was generally similar to the exact results for nonlinear dynamic analysis. MPA was more accurate than all pushover analyses in estimating floor story drifts, plastic hinge rotations, and plastic hinge locations.

Chintanapakdee and Chopra (2003) also evaluated the accuracy of the MPA for a wide range of buildings and ground motions. Five strength levels corresponding to SDOF-system with ductility factors of 1, 1.5, 2, 4 and 6 were utilized for generic one-bay frames of 3, 6, 9, 12, 15, and 18 stories. Considering California earthquakes, each frame was analyzed with a set of 20 large-magnitude smaller-distance records. The median values of story drift demands from the MPA and nonlinear dynamic analyses were calculated and compared. It was shown that with two or three modes included, the MPA predictions were in good correlation with the nonlinear dynamic analyses, and the MPA predicted the changing height wise variation of demand with building height and ductility factor accurately. The bias and dispersion in the MPA estimates of seismic demands were found to increase for long-period frames and ductility factors although no perfect trends were observed. It was also illustrated that the bias and dispersion in the MPA estimates of seismic demand for inelastic frames were larger than those for elastic systems due to additional approximations involved the MPA procedure. Finally, the MPA procedure was an extended to estimate the seismic demand of inelastic systems with seismic demand being as defined by an elastic design spectrum.

In the study of Providakis (2008), the pushover analysis procedure for seismic analysis was employed. Characteristic of steel-concrete composite structures under the near-fault earthquake excitations were numerically explored. Three-dimensional (3-D) buildings contained two and five-storey with steel–concrete composite slabs, beams, and steel columns. The offering 3-D building models were supposed by placing in a near-fault area so that they had the influence of robust ground motion of the isolation apparatus. It approved the confirmation of the suitability of the isolation system. The analogy to the performance of the seismic-protected building to the unprotected buildings were determined. The limitations and characteristics of their performances were resulted from this study.

Another study about adaptive pushover methods for seismic assessment was presented by Pinho et al. (2006). The latest establishment and evolution of the alleged adaptive pushover methods was improved after every analysis step of the modal guidelines considering the property of the structural response at expanding loading levels. Within such adaptive framework, the implementation of a displacement, contrary to force, incremental loading vector was evaluated in every step of the analysis allowing to the widespread dynamic characteristics of the structure. Additionally, it was very attractive since that was in line with the offered approach for the evolution and code execution of displacement or further widely deformation-based design and evaluation methods as illustrated in Figure 2.7 (Pinho et al., 2006).

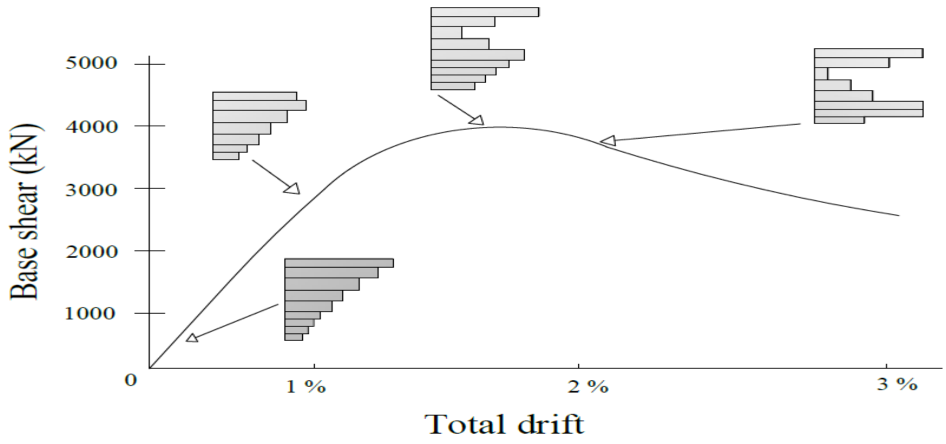


Figure 2.7 Adaptive pushover: shape of loading vector updated at each analysis step (Pinho et al., 2006)

It was noted that in an adaptive pushover method to the response to the structure was calculated in increased mode by piecewise linearization, as schematically shown in Figure 2.8. Therefore, it was attainable for italicization the tangent rigidity at the beginning of every increment with each other through the mass surrounded by the system, to estimate modal response characteristics of every increased bogus system through elastic eigenvalue analysis, and italicization like as modal measures to be coincident renovate (i.e. increment) the pushover loading vector (Pinho et al., 2006).

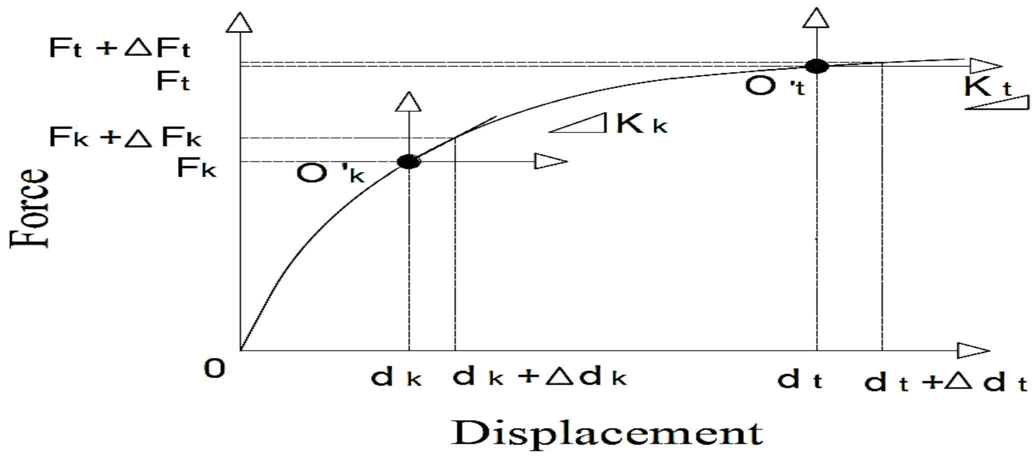


Figure 2.8 The use of tangent stiffness in updating (i.e. incrementing) the loading vector (Pinho et al., 2006)

Moghaddam and Hajirasouliha (2006) investigated the nonlinear dynamic behaviour of 5, 10, and 15 storey bare frames using 15 synthetic earthquake record suitable to a design spectrum. The predestined lateral load was taken into consideration. The pattern supplied suspicious estimation of inter-story drift. In this approximation, a multi-storey frame was analized in view of an equivalent shear-building model. To vanquish this, a clarified analytical model for the seismic response of the concentrically braced frames was recommended. A conventional shear-building model was also adjusted through presenting additional explanations for displacements in additive shear displacements. It was shown that the modified shear-building models had a better estimation of the nonlinear dynamic response of the framed structures as compared to the nonlinear static method.

Saiidi and Sözen (1981) produced a ‘low-cost’ analytical model which was named as the Q-model for calculating displacement histories of multi-storey reinforced concrete structures subjected to various ground motions. The Q-model, which was based on the idea of Gülkan and Sözen (1974), involved two simplifications on the reduction of a MDOF model of a structure to a SDOF oscillator and the approximation of the variation of stiffness properties for the entire structure by a single spring to take account of the nonlinear force-displacement relationships. That characterized its properties. Earthquake simulation experiments of eight small-scale structures were performed, and the displacement histories were compared to the results from the nonlinear static analyses based on the Q-model. It was revealed that the performance of the Q- model in the simulation of high- and low- amplitude responses was satisfactory for most of the test structures. It was reported that the model would need to be further validated by more experimental and theoretical analyzes.

A simplified analysis model for the seismic response prediction of the steel frame was proposed by Baik et al. (1988). It was based on the pushover analysis concept but included cumulative damage parameters using the Park-Ang damage model (Park, 1985). These parameters accounted for the effects of all inelastic excursions and not only for the maximum excursion. The model was tested on 10- and 20- storey single bay steel structures and was considered acceptable for preliminary design purposes. It was noted that the prediction of damage using the equivalent SDOF model deteriorated with increasing structure height and in the presence of irregularities. The authors suggested that the SDOF nonlinear model could provide better estimates of damage parameters than an elastic multi-storey model.

In the study of Deierlein and Hsieh (1990), they utilized the capacity spectrum method to compare the experimental and theoretical results for the seismic response of a single storey single bay steel frame with the analytical results of a 2D pushover analysis. The frame was modelled with semi-rigid connections. The results showed differences approximately 10% to 20% between the comparative quantities such as the period of vibration, maximum displacement, and maximum acceleration. It was concluded that the capacity spectrum method could provide reasonably accurate lower and upper bounds on the inelastic response to a structure subjected to strong ground motion.

A seismic response prediction method for building structures presented by Hasegawa and Kamura (2008) predicted the maximum inter-storey drift and damage of each member of the buildings. In the procedure of seismic response prediction for steel moment-resisting frame was illustrated, which was utilized inelastic strain energy obtained from the pushover analysis. In order to further examine the validity of the proposed method for practical application, thirteen kinds of sample moment-resisting steel frame were selected. A series of earthquake response analyses of these example frames was carried out and compared to the results of the proposed method. From the results of the earthquake response analysis, it was found that the maximum inter-story drift and the cumulative ductility demands of members obtained from the proposed method approximately catched the tendency of the results of the earthquake response analysis, as seen in Figure 2.9.

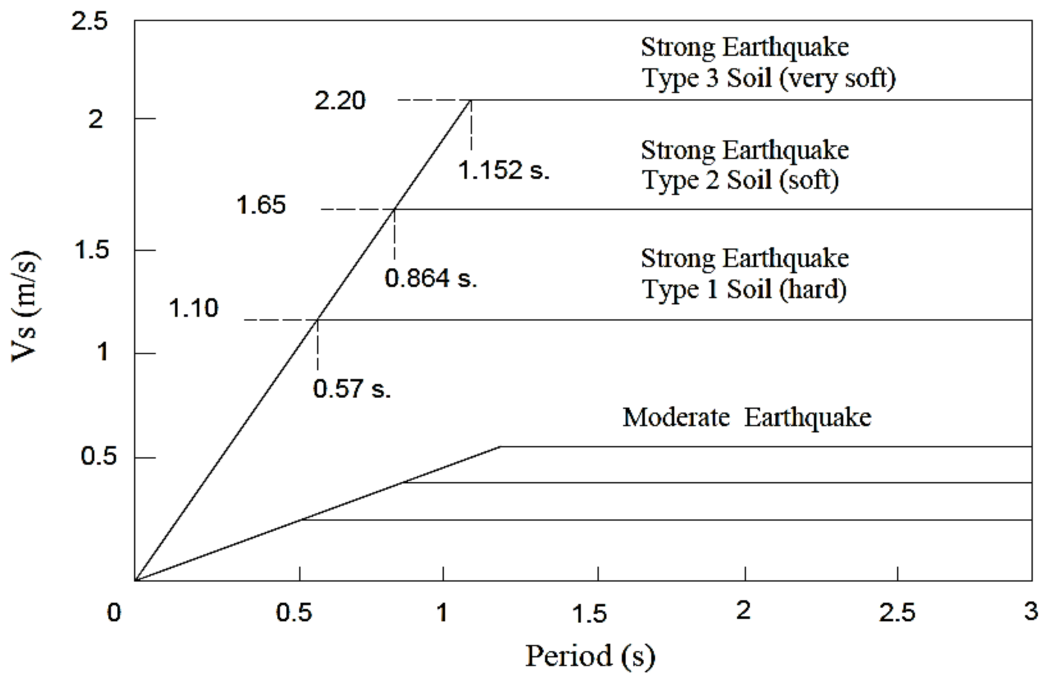


Figure 2.9 Design pseudo-velocity spectrum prescribed in Japanese building code (Hasegawa and Kamura, 2008)

The energy input into a structure was calculated based on the damage causing an energy, which was equal to the sum of the vibration strain energy and the cumulative plastic energy of the structure. Vs is the design pseudo-velocity spectrum as shown in Figure 2.9, which was stipulated in the Notification No.631 (Seismic calculation method for the building structures based on the balance of energy) of Japanese building code and it was used in the calculation of dissipated energy of a structure

(Hasegawa and Kamura, 2008).

Extended N2 method by attempting to include cumulative damage was also studied in the technical literature. The test structure applied to the seven-storey reinforced concrete building tested in the U.S, Japan research project. The seismic demands for each element were computed in terms of the dissipated hysteretic energy using the Park-Ang model (Park, 1985). The conclusions were that the dissipated hysteretic energy increased with increasing the duration of ground motion, and it was significantly affected by the reduction of strength of the structural elements. It was also concluded that when the fundamental period of the structure was much larger than the dominant period of the ground motion, the higher mode effects became an important issue. In this case, the input energy and dissipated hysteretic energy to a MDOF system were generally larger than the corresponding quantities in the equivalent SDOF system. The researchers suggested that the N2 method was likely to underestimate the quantities, which governed damage in the upper part of a structure (Gaspersic et al., 1992).

2.2 Bracing systems in structures

One of the most widely used lateral load resisting systems is the bracing. When bracing is added, the structure can not undergo the deformation under the lateral load and its increase the stiffness of the frames. Worldwide damage experienced demonstrates during past earthquakes that steel multi-storey building structures generally exhibit adequate seismic response (Di Sarno and Elnashai, 2002). The enhanced energy absorption of structural ductile systems employed because of the favorable mass to stiffness ratio of base metal. However, comparatively the latest earthquakes, e.g. the 1994 Northridge (California), 1995 Kobe (Japan), and 1999

Chi-Chi (Taiwan) have exhibited that inferior specifying of relations (e.g. the beam-to-column, brace-to-beam, brace-to-column, and column-to-base) and buckling of diagonal braces be able to weaken within the structure (Broderick et al., 1994; Elnashai et al., 1995; Nakashima et al., 1998; Watanabe et al., 1998; Naeim et al., 2000).

The distribution of the damage level, destruction of structural members, and junctions by appreciation to a structure are exhibited in Figure 2.10. Similarly, it was observed in the consequences of the 1995 Hyogoken-Nanbu (Kobe) earthquake, damaged buildings were categorized as having unbraced (UFs) or braced (BFs) frames. So considering the two essential framing tendencies of a building, the evaluated structures contained the subsequent 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).

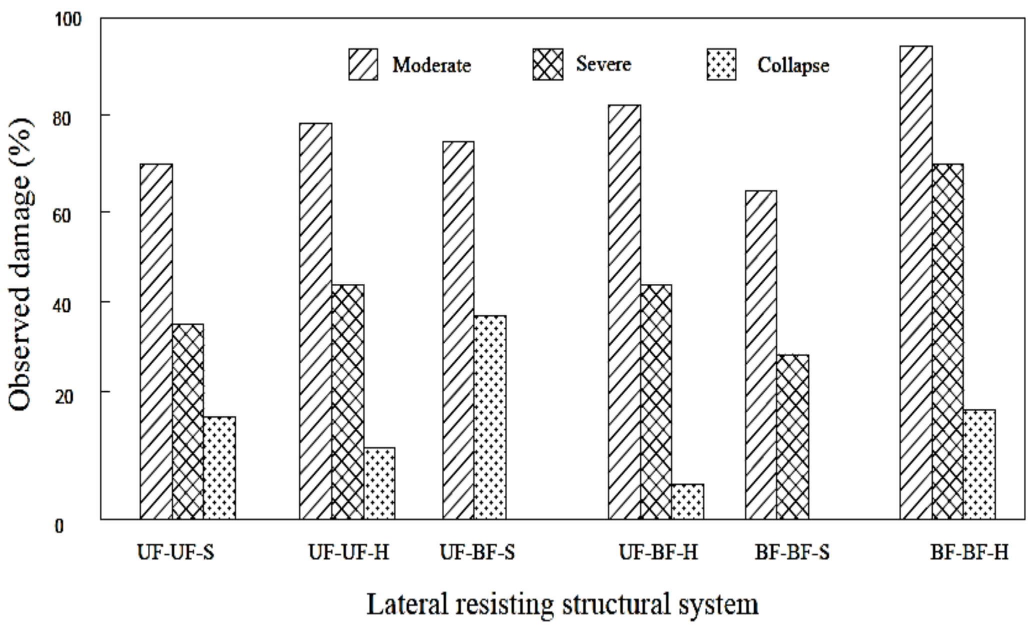


Figure 2.10 Distribution of damage level, damage to structural members, and connections (Di Sarno and Elnashai, 2009)

The beams consisted of wide-flange sections, also revolved or built-up. For the columns, wide-flange (H) sections were utilized extensively; square-tube (S) sections were additionally used in several structural systems. In view of the 988 damaged steel building, 432 (43.7%) were UF-UF, 134 (13.6%) were UF-BF, and 34 (3.4%) were BF-BF, with 388 (39.3%) having unknown framing systems. It was reported that greater part of damaged buildings were named as the unbraced moment resisting frames (MRFs). Figure 2.10 also presents the position of damage, namely columns, beams, beam to column connections, braces and column bases as a part of frame variety of main attentions from the composed data. The following observations were highlighted (FEMA 355E, 2000):

- Columns in UFs sustained the most damage proportionate to another frame element (in terms of the number of buildings), at the same time, braces in BFs were the most frequently damaged structural element,
- Damage beam-to-column connections were most significant for UFs employing hollow section (square-tube) columns, and
- Damage beam-to-column connections and column bases were also significant in UFs.

Considering the damage to columns, it was mainly important for UFs accompanied by wide flange members. The argument of the more than observed information was illustrative of the usual structural response of steel buildings damaged through moderate-to-severe earthquake ground motions as shown in Figure 2.11 (Di Sarno and Elnashai, 2009).

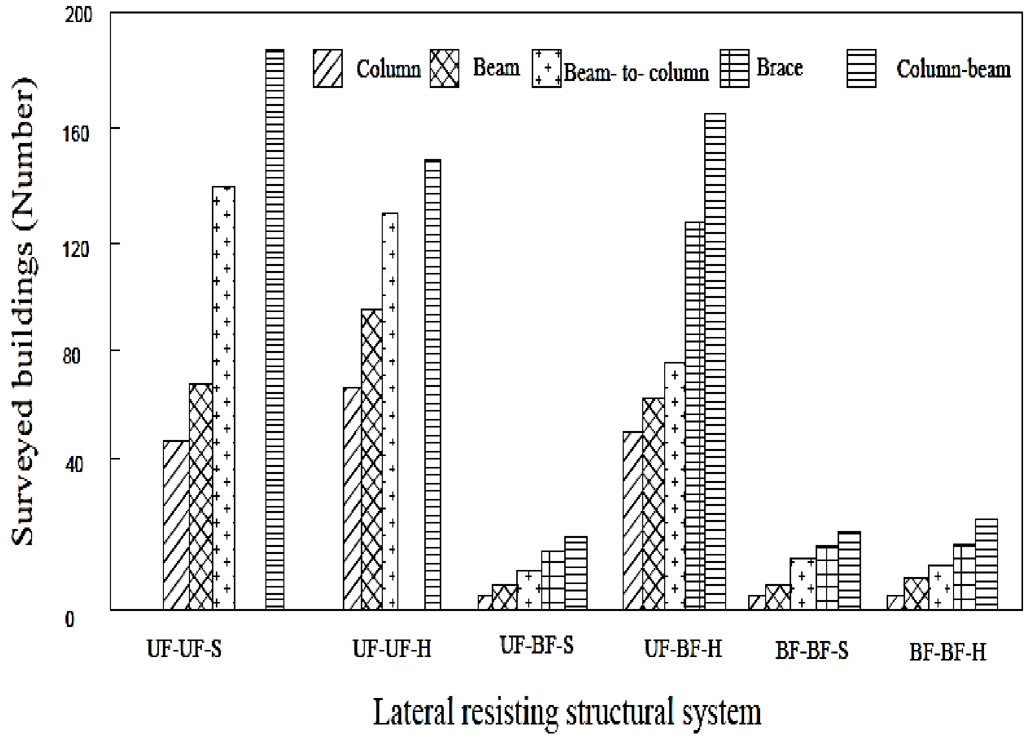


Figure 2.11 Distribution of damage level with respect to structural type (Di Sarno and Elnashai, 2009)

Frequently, in the plastic range of multi-storey buildings, the event of buckling resulted in the loss of the capacity of the structure, and so unexpected alterations in the dynamic qualities of the lateral resisting structure system. Figure 2.12 depicts such brittle fractures for the beam-to-column and brace–column. The beam-to-column together with braces as an outcome might be incompetent in ductile MRFs or concentrically braced frames (CBFs) if they were unsuitable capacity designed (Bruneau et al., 1998; Nakashima et al., 2000; Tremblay, 2002; Broderick et al., 2005).

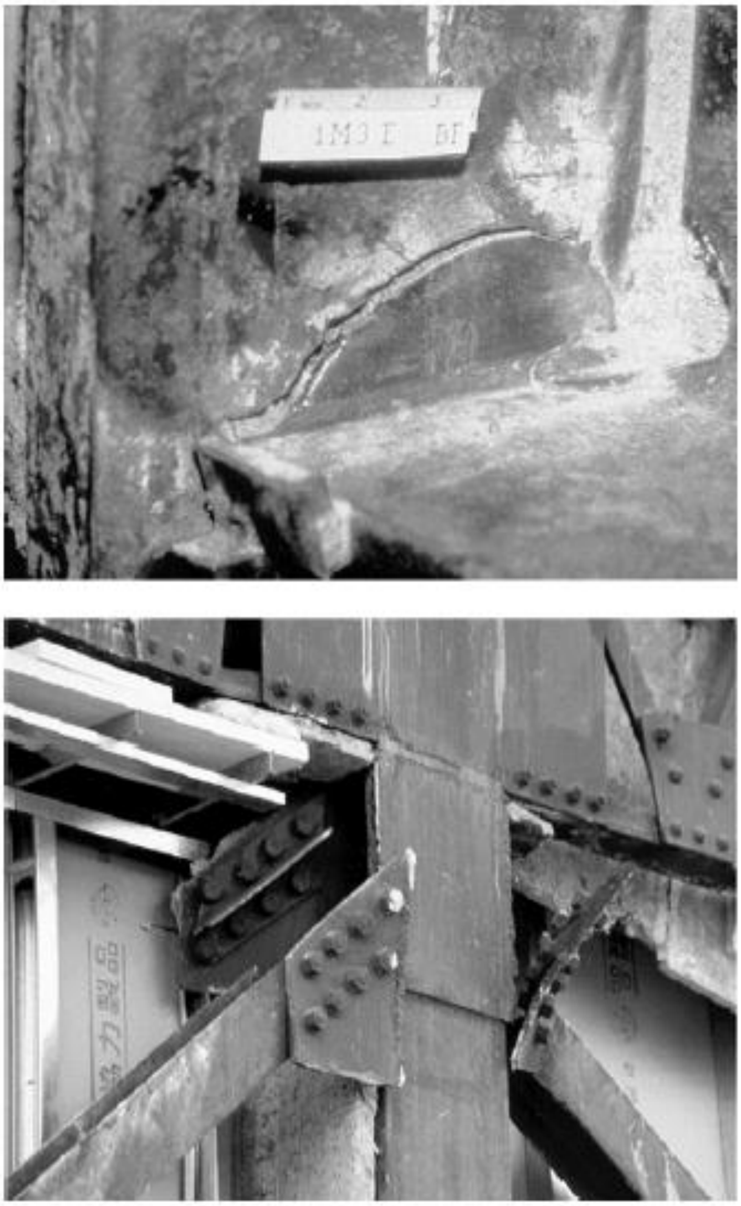


Figure 2.12 Fracture in beam-to-column connections in the Northridge earthquake (top) and web tear-out in bolted brace-to-column connections during the 1995 Kobe earthquake (bottom) (Di Sarno and Elnashai, 2009)

Bracing is an extremely operative global upgrading strategy to increase the global stiffness and strength of steel UFs. Earthquake loads whenever hysteretic dampers were utilized increased the energy assimilation of structures and/or reduced the demand required structures with enhanced energy excess may securely oppose forces, and deformations resulted in firm ground motions. Usually global adjustments to the structural system were envisaged such that the design required on the existing structural and non-structural components were reduced than their abilities as seen in Figure 2.13. Lower demands might decrease the danger of brittle malfunction in the structure and/or avert the obstruction of its functionality and in turn, the downtime caused by the retrofitting, those were key characteristics in the earthquake defeat estimation (Bozorgnia and Betero, 2004; Deierlein, 2004).

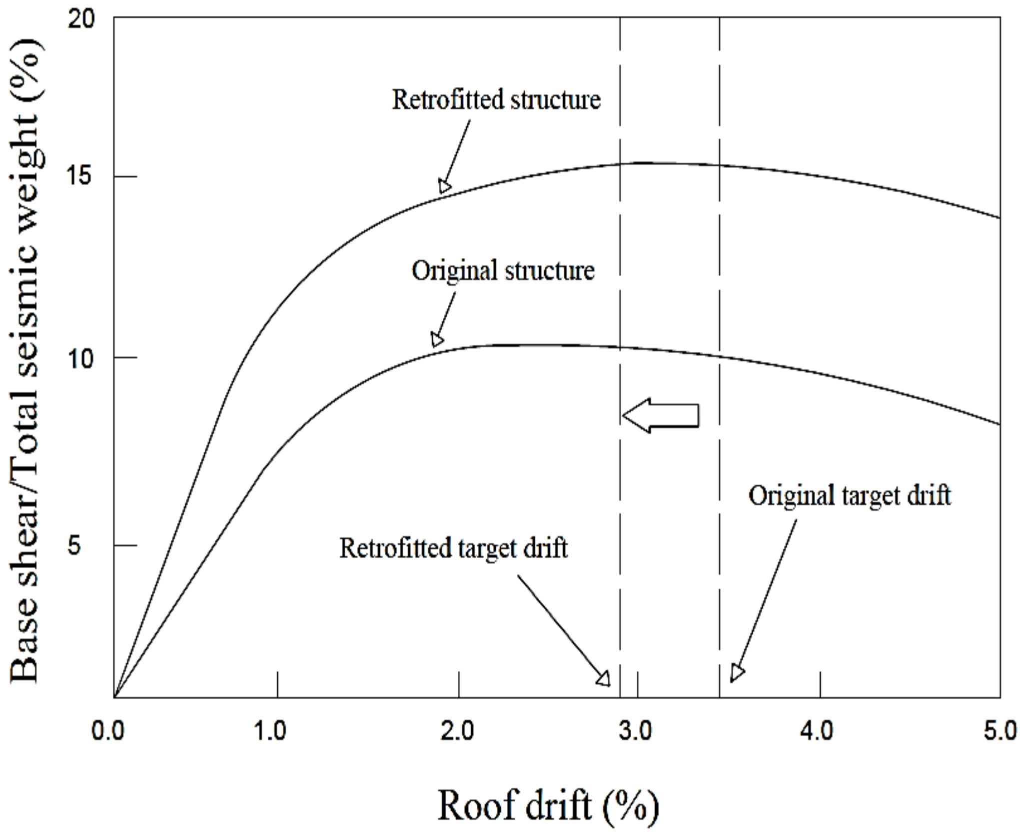


Figure 2.13 Characteristics of global intervention approaches in seismic retrofitting of structures (Di Sarno and Elnashai, 2009)

The capacity design outline by forcing inelasticity was considered and surrounded by the dissipative localities (plastic hinges in MRFs and braces in CBFs) and certifying that all other members and connections performed linearly attained by the achievement of global structural ductility. The diagonal braces were able to be aesthetically disagreeable where they altered the primary architectural characteristics of the building (Tena-Colunga and Vergara, 1997; Wolfe et al., 2001).

2.3 Previous researches on concentrically braced frames

Concentrically braced frames resist lateral seismic accelerations primarily through axial forces and cause deformations of braces, beams, and columns. In the elastic range, they behave as vertical trusses. Their post elastic behavior may involve the flexure of frame members, but the inelastic drift is expected to be mainly a result of brace axial deformation (except in the case of chevron bracing). Figure 2.14 shows a few common concentrically braced frame configurations: diagonal bracing, cross bracing (X braces), and chevron bracing V- or inverted-V bracing (Sabelli, 2001).

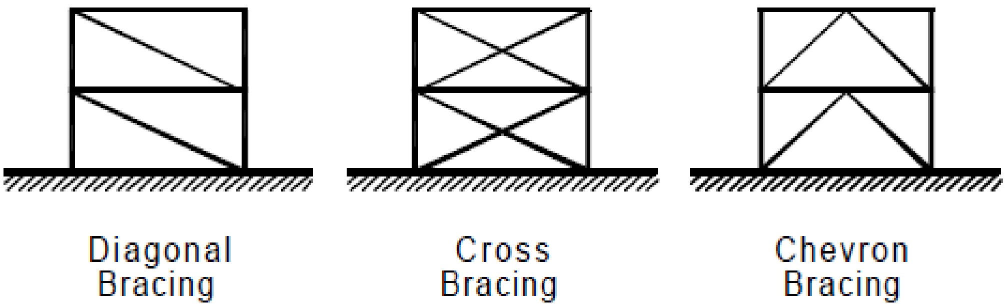


Figure 2.14 Common bracing configurations (Sabelli, 2001)

Structural engineers have frequently reached concentrically braced steel frames as a thrifty means for confronting earthquake loads. Previous to now this system, popularly employed, has viewed an extension utilitization in the latest years as a result of the raised scrutiny disposed to steel moment-resisting frames, which contain indicated to be responsive to huge lateral displacements through strict earthquake ground motions and to need special care to require abstain from troubles related with the fracture of beam to column connections (FEMA-355F, 2000). However, damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico (Osteraas and Krawinkler, 1989), 1989 Loma Prieta (Kim and Goel, 1992), 1994 Northridge (Tremblay, 1995; Krawinkler, 1996), and 1995 Hyogo-ken Nanbu (AIJ, 1995; Hisatoku, 1995; Tremblay, 1996) earthquakes, enhances interests through the ultimate deformation capacity of this section of structure. Distinctive braces frequently occupy the only limited ductility capacity less than cyclic loading (Tang and Goel, 1989 a, 1989 b).

Typical brace hysteresis performance is irregular between tension and compression as well as the illustrations significant strength decline when loaded monotonically or cyclically in compression. Considering this complex performance, the real disseminations of internal forces and deformations in less than a structure frequently oppose much from those prophesied utilizing traditional design methods (Jain and Goel, 1978 a, 1978b; Jain et al., 1978; Khatib and Mahin, 1987). Design simplifications and applicable respectfulness regularly manufacture designs in which the braces chosen for some stories include distant larger capacity than needed though those chosen for different stories have capacities extremely close to their computed demand. This alteration on storey demand-to-capacity ratios, together with actual post-buckling strength defeats in those braces, tends to concentrate earthquake damage in several stories. Such concentrations of damage place largely burdens on the limited ductility capacities of braces and their connections. The seismic design demands for braced frames have been adapted significantly through the 1990, and the idea of special concentric braced frames has been commenced (AISC, 1997; UBC, 1994). A large investigation has in addition started to develop the appearance of concentrically braced frames by the institution in recent structural configurations (Khatib and Mahin, 1987) or the utilitization of special braces by Liu and Goel (1987), metallic yielding

(Watanabe, 1996; Kamura et al., 2000), high-performance materials (Ohi et al., 2001), and friction and viscous damping (Aiken, 1992). Through the past decade, an evaluation of the extensive seismic performance qualities of concentrically braced frames designed to prevalent standards was timely and there have been parallel improvements in investigation connected to qualifying the seismic risk at a site, appearing the seismic response, and theories of qualifying seismic performance in probabilistic terms (Sabelli, 2001).

The arrange and allocation of plasticity in braced frames were influenced through the brace sizes, slenderness ratio, frame shape, and analysis kind in ductile concentrically braced frames of regular buildings. The global ultimate strength was operated by the organization of structural mechanism in one story, considering that the redistribution of internal forces in one story was contained particular in that story. The allocation of shear in other stories was not affected. Thus, a yielding in the tension braces in one story would outcome in the formation as a mechanism in that story and the structure, therefore, ranged over its ultimate capacity. This indicated that the support in strength of the critical story was further the global stand in strength to the frame (Annan et al., 2009).

The seismic resistance of steel braced frames was a mainly obtained from the axial force capacity of their constituents. Steel braced frames were effective as vertical secures where the columns were the chords and the beams and braces were the web members. Braced frames might act alone or in concurrence with concrete or masonry walls, or steel moment frames, to form a compined system. A nonlinear load-deformation behavior for braces was determined by experimentation or analysis supported by the experiment (FEMA 273, 1997).

The Federal Emergency Management Agency FEMA/SAC steel project resulted in the development of guidelines for the design of steel moment frames, which limited interstory displacement to give performance levels (FEMA 350, 2000). However, 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 was through the use of innovative materials in the braces system (McCormick et al., 2007).

Prior to the 1993 Load and Resistance Factor Design (AISC, 1993), in the 1994 Uniform Building Code (UBC, 1994), concentrically braced frames had been treated by building codes as elastic truss systems. Post elastic behavior was only considered in prescribing calculated brace strengths (based on strength degradation under cyclic buckling as a function of brace slenderness); this effectively increased the elastic force capacity of these systems but did not effectively address post elastic modes of behavior. Concentrically braced frames designed according to such requirements have not been observed to achieve ductile post elastic behavior. One of several non- ductile behaviors has usually led to brittle failure, typically of a brace or its connections. For this reason, braced frames have been required to be designed to higher forces, raising the threshold hazard level for which ductility demands on the system could be expected, but not increasing ductility or greatly reducing the response to more severe ground motions (Sabelli, 2001).

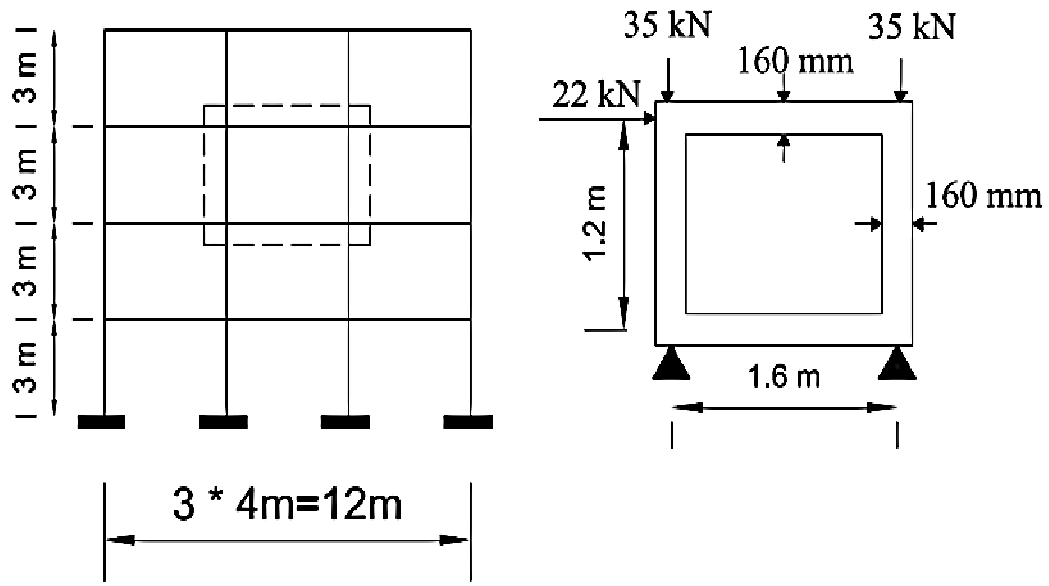
The creation of the category of Special Concentrically Braced Frames (SCBFs) in the 1994 Uniform Building Code (UBC, 1994) acknowledged research carried out at the

University of Michigan which showed that these systems, with careful proportioning of members and detailing of connections, could perform in a ductile manner (Astaneh et al., 1985; Hassan and Goel, 1991; Goel, 1992). Recent design guidelines have focused on proper proportioning and detailing of SCBFs so that they can achieve trilinear hysteretic behavior; the three ranges of behavior were the elastic range, the post buckling range, and the tensile yielding range (AISC, 1997; Bruneau et al., 1998; SEAOC, 1999). The 1994 Uniform Building Code and Seismic Provisions for Structural Steel Buildings (AISC, 1997) category of Special Concentrically Braced Frames is intended to lead to systems in which braces can undergo several large cycles of buckling and yielding. Such systems were expected to be ductile and to perform well in earthquakes (SEAOC, 1999).

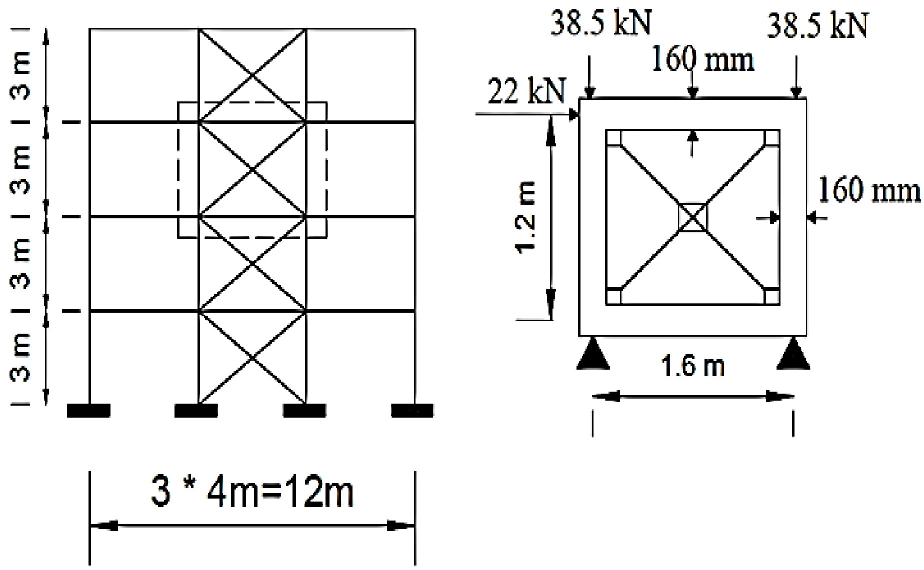
In current United States practice, concentrically braced frames were typically designed using elastic analyses. In such analyses, the behavior of these frames follows that of a vertical truss for any concentrically braced frame configuration: chevron, cross-braced, or zipper frames. However, the post elastic behavior of braced frames was integrally linked to the configuration of braces. This was affected either directly, such as in the post-buckling change in deformation mode of chevron-braced frames (Khatib et al., 1988), or indirectly, by the selection of brace size and its influence on brace fracture life (Lee, 1987). Current United States building codes treat these various configurations as one system (AISC, 1997; UBC, 1994). These codes recognize issues specific to one configuration, chevron bracing, and provisions exist to minimize the deviation of its post elastic performance from that of other configurations; chevron-braced frames designed to those provisions were treated by the codes as equivalent to other configurations.

These building codes made significant progress over the last decade in addressing unfavorable yield modes of braces and their connections. Since premature connection failures are now less likely to limit demands on braces in new construction, brace fracture life could become a much more important issue in the performance of these buildings. Several studies on the behavior of braces subjected to cyclic loading and on fracture criteria for steel braces were conducted at University of Michigan (Lee, 1987). Sophisticated nonlinear element models were performed to match these criteria (Jain and Goel, 1978 a, 1978 b; Jain et al., 1978; Rai et al., 1996).

The use of internal steel bracing for seismic performance of reinforced concrete (RC) frames were also investigated by Youssef et al. (2007). They used concentric internal steel bracing for new construction. Two specimens representing a reinforced concrete moment frame with moderate ductility and a braced reinforced concrete frame were designed. A four-storey building and 2/5 scaled models are shown in Figure 2.15.



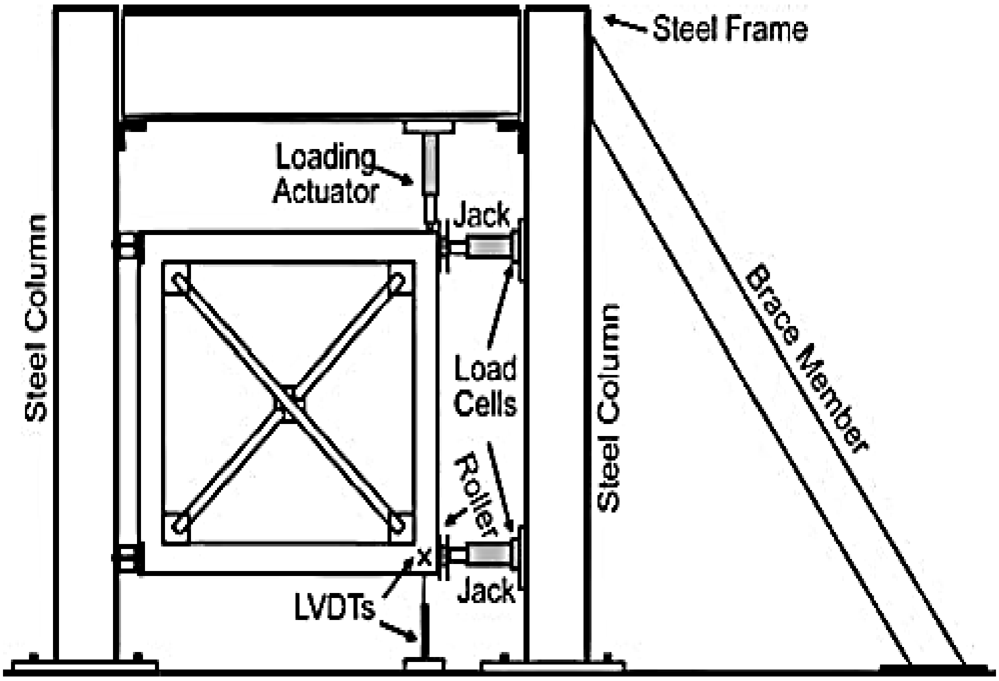
a) Moment frames



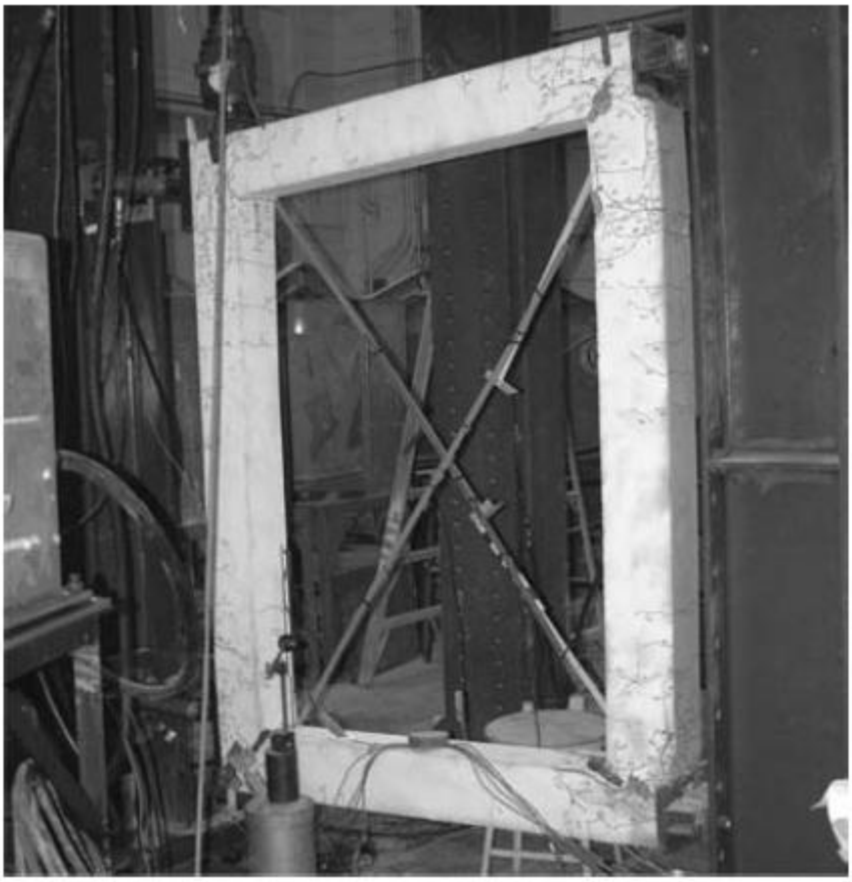
b) Braced frames

Figure 2.15 View of a) ductile RC moment frame and scaled ductile RC moment frame and b) braced RC frame and scaled braced RC frame (Youssef et al., 2007)

Test results illustrated that the concentric braced frame had higher lateral load carrying capacity than the moment frame and supplied suitable ductility. The immoderate load capacity and the initial stiffness of the concentrically braced reinforced concrete frame was about 2.5 times that of the reinforced concrete moment frame. Due to formerly buckling of the compression brace, the lateral stiffness of the braced frame was greater than that of the moment frame. It was observed that at low drift levels, the energy dispensed through the braced frame was decreased than that by the moment frame. The test setup used in this investigation is shown in Figure 2.16 (Youssef et al., 2007).



a)



b)

Figure 2.16 Test set up: a) schematic and b) photo views (Youssef et al., 2007)

The reason for the results described in the study of Youssef et al. (2007) was mainly due to the fundamental high stiffness of the braced frame. At higher levels of drift, the energy spread by the braced frame was very higher than that by the moment frame. This exhibited that the seismic behavior of the braced frame was expected to be better than that of the moment frame. Since the frame of the braced frame system was newly constructed, this study additionally implied that the using of the steel bracing instead of shear walls for recent construction has various advantages:

- Reducing the weight of the structure, thus reducing the seismic loads and
- Increasing the ductility of the structure.

Maheri and Sahebi (1997) proposed the use of internal concentric brace members over internal steel trusses. The research included a sequence of tests acting on a number of model frames whose details are exhibited in Figure 2.17.

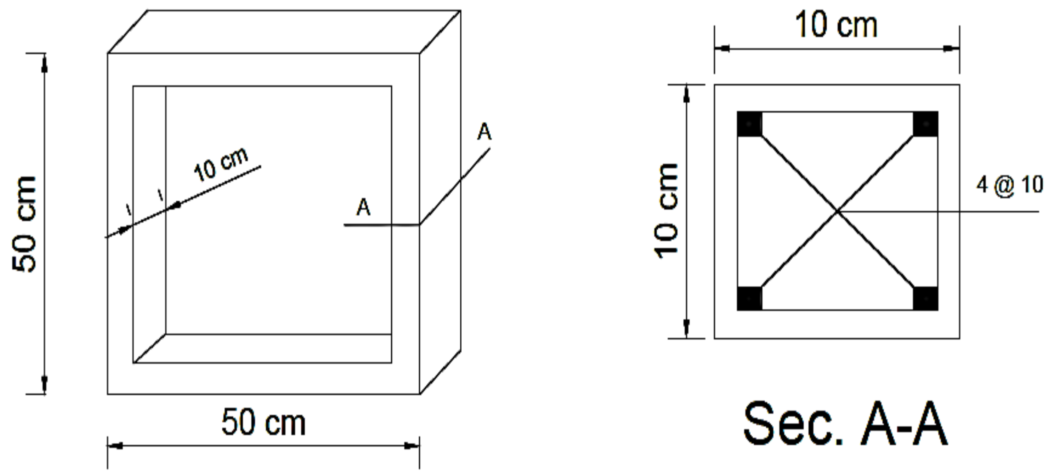
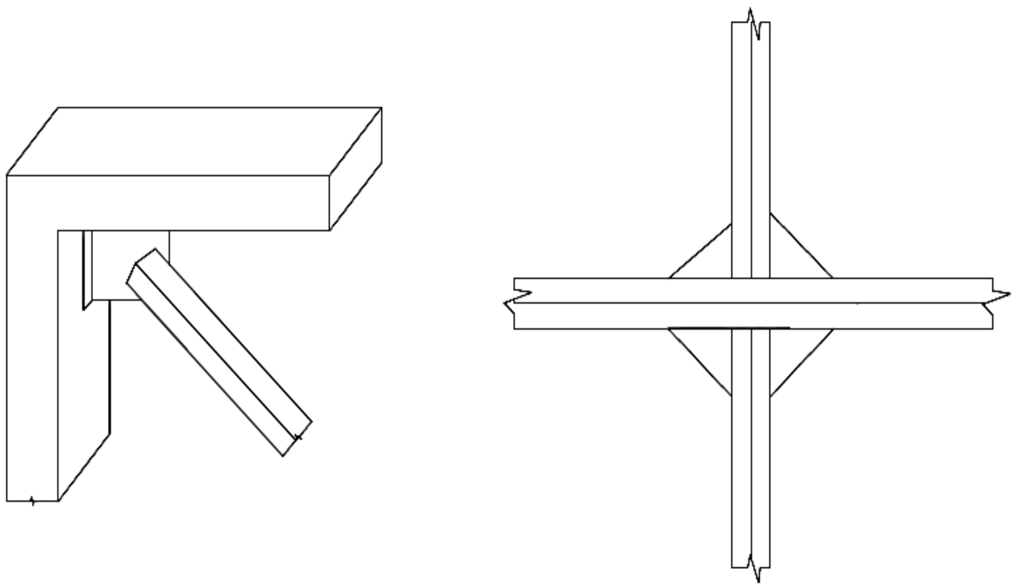


Figure 2.17 Detail of typical test model (Maheri and Sahebi, 1997)

The objective of the testing program conducted by Maheri and Sahebi (1997) was to determine the degree of effectiveness of different diagonal bracing arrangements to increase the lateral load capacity of the existing concrete frames and to observe the relative behavior of tension and compression braces. For these investigations, the common X-bracing system was chosen. Four model frames were selected namely: a concrete frame without bracing, a concrete frame braced with a diagonal tension brace, a concrete frame braced with a diagonal compression brace, and a concrete frame braced with X-bracing. In order to reduce the buckling tendency of the compression brace in the X-brace system, the two diagonal braces were also connected to each other at their cross-point by a steel plate. The connection to the frame was done by welding the braces to the sides of a steel plate which was welded to an equal angle positioned and pre-cast at the corners of the frame. The connection details are shown in Figure 2.18.



a) b)

Figure 2.18 Connection detail of a) steel brace to concrete frame and b) steel cross braces to each other (Maheri and Sahebi, 1997)

An important point of observation in their study was that while testing of a cross-braced frame, the rate at which the two braces carried the load was not equal. The more dominant behavior of the tension braced was observed and compared with the compression brace initially. The tension brace carried higher load than the compression brace. However, at higher loads, the system showed a nonlinear behavior and the dominance of the tension brace started to reduce. The failure in the tension brace was noticed to be occurred in its welded connection of the mid-span plate. After the tension brace failed, the compression brace was buckled under the increased load. In testing of the compression braced frame, it appeared that in the elastic range, the load was transferred directly to the concrete frame and the share of load bearing of the compression brace was almost zero. Only at higher loads, the response of the frame was transferred into a nonlinear range. So, the compression brace started to participate in load. The failure was realized later than the compression brace buckled. It was observed that a huge increment in the shear strength of a concrete frame caused by only one diagonal brace acting also in tension or compression was attained. For the model frames tested, the increment in shear strength caused by the one brace was 2.5 times that of the frame itself. The strength of the concentric X-braced model frame was calculated at four times that of the unbraced frame (Maheri and Sahebi, 1997).

`

CHAPTER 3

METHODOLOGY

3.1 Analytical models

In this study, the existing steel structures having four different numbers of stories (4, 8, 12, and 16) were taken into consideration as a case study. Santa-Ana and Miranda (2000) were first designed these structures. All buildings have the same plan and they contain three bays on each direction as shown in Figure 3.1.

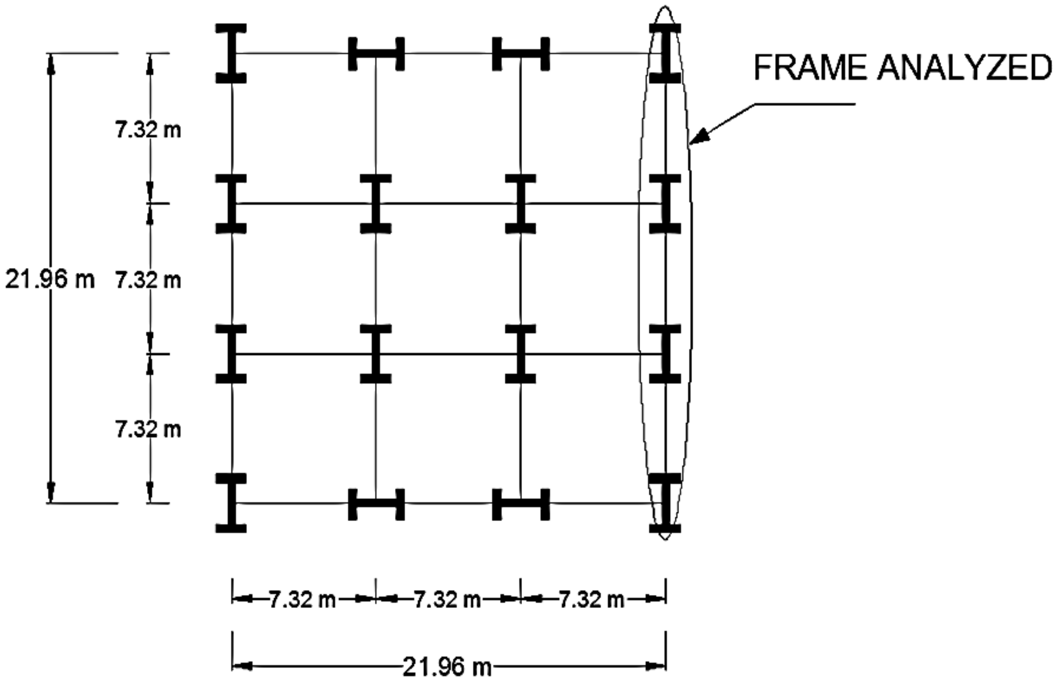


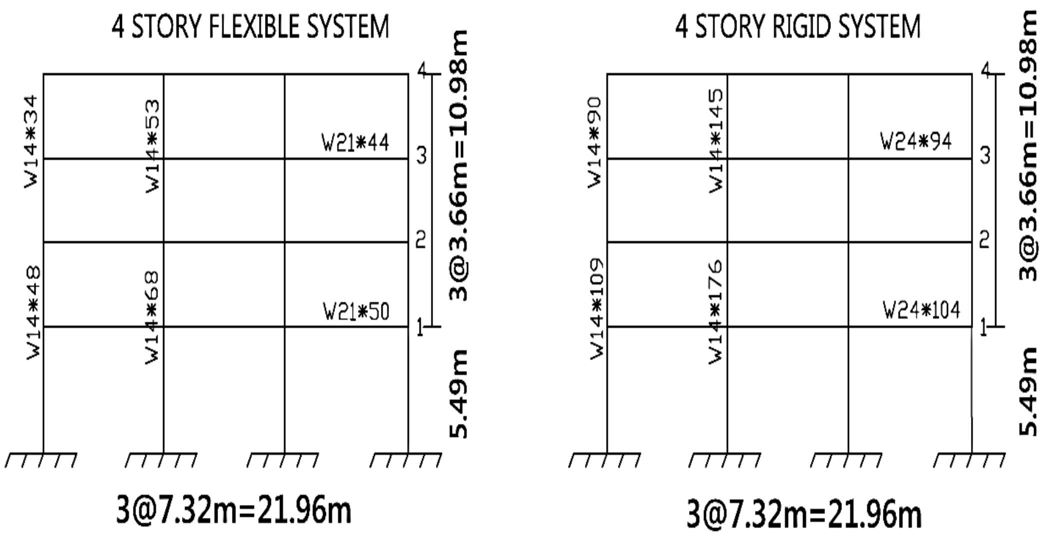
Figure 3.1 Plan view of the existing buildings (Santa-Ana and Miranda, 2000)

The buildings were assumed to have a uniform mass distribution over their height and a non-uniform lateral stiffness distribution. Steel members in the buildings were designed using the lateral load distribution specified in the 1994 Uniform Building

`

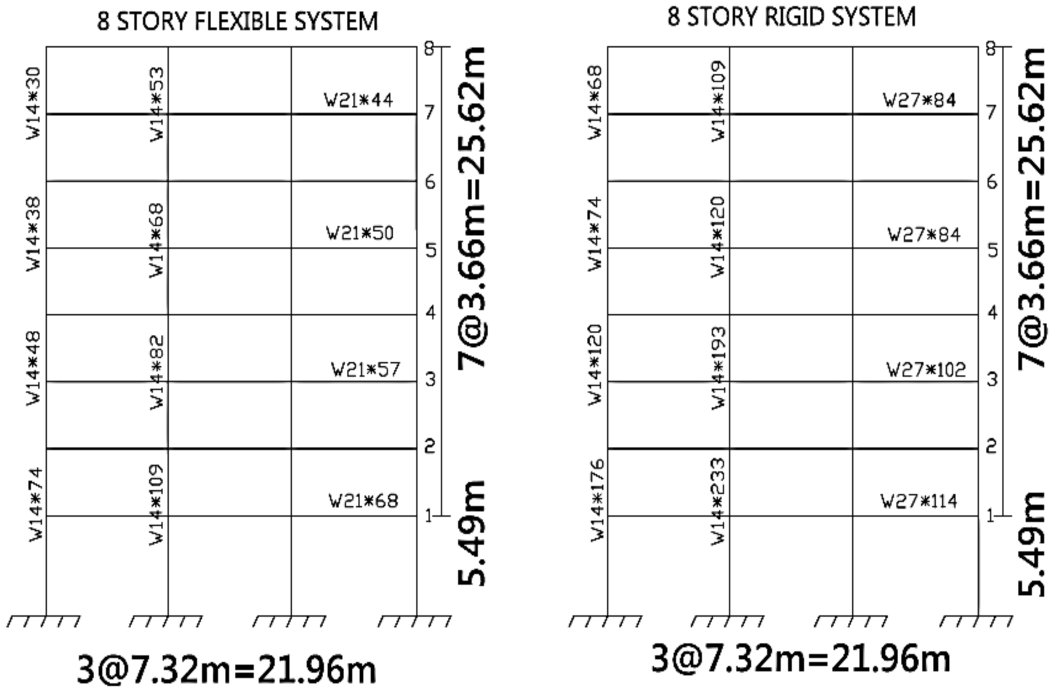
Code (UBC,1994). The member stiffness was tuned to accomplish the fundamental periods of vibration for each structure representative of those obtained from actual earthquake records. As a result, two frames with different dynamic properties, namely flexible and rigid frames were considered. In addition, as reported in the study of Santa-Ana and Miranda (2000), with the exception of the beam-to-column connections in the top floor, the steel sections of structural members were chosen such that the sum of plastic section modulus of the columns framing into each beam-column joint was greater than that of plastic section modulus of the beams framing into the same joint.

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. The beams and columns were constructed with W profiles. The section profiles of the unbraced frames having various number stories are shown in Figures 3.2 and 3.3 and the dynamic characteristics of moment resisting frames (MRFs) are exhibited in Table 3.1.



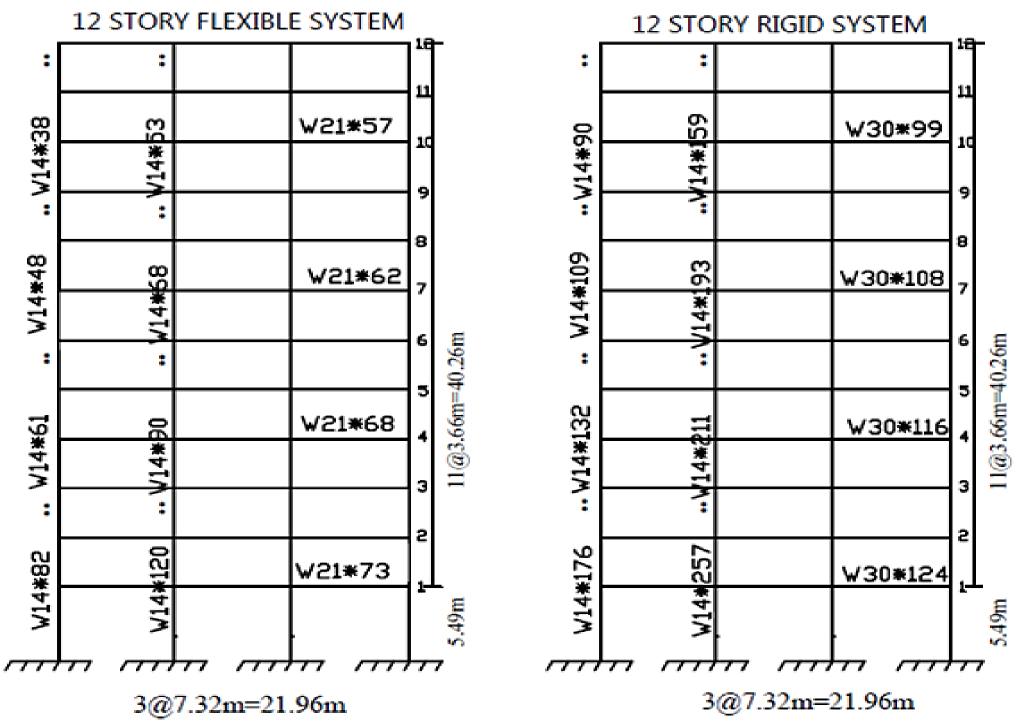
a)

`

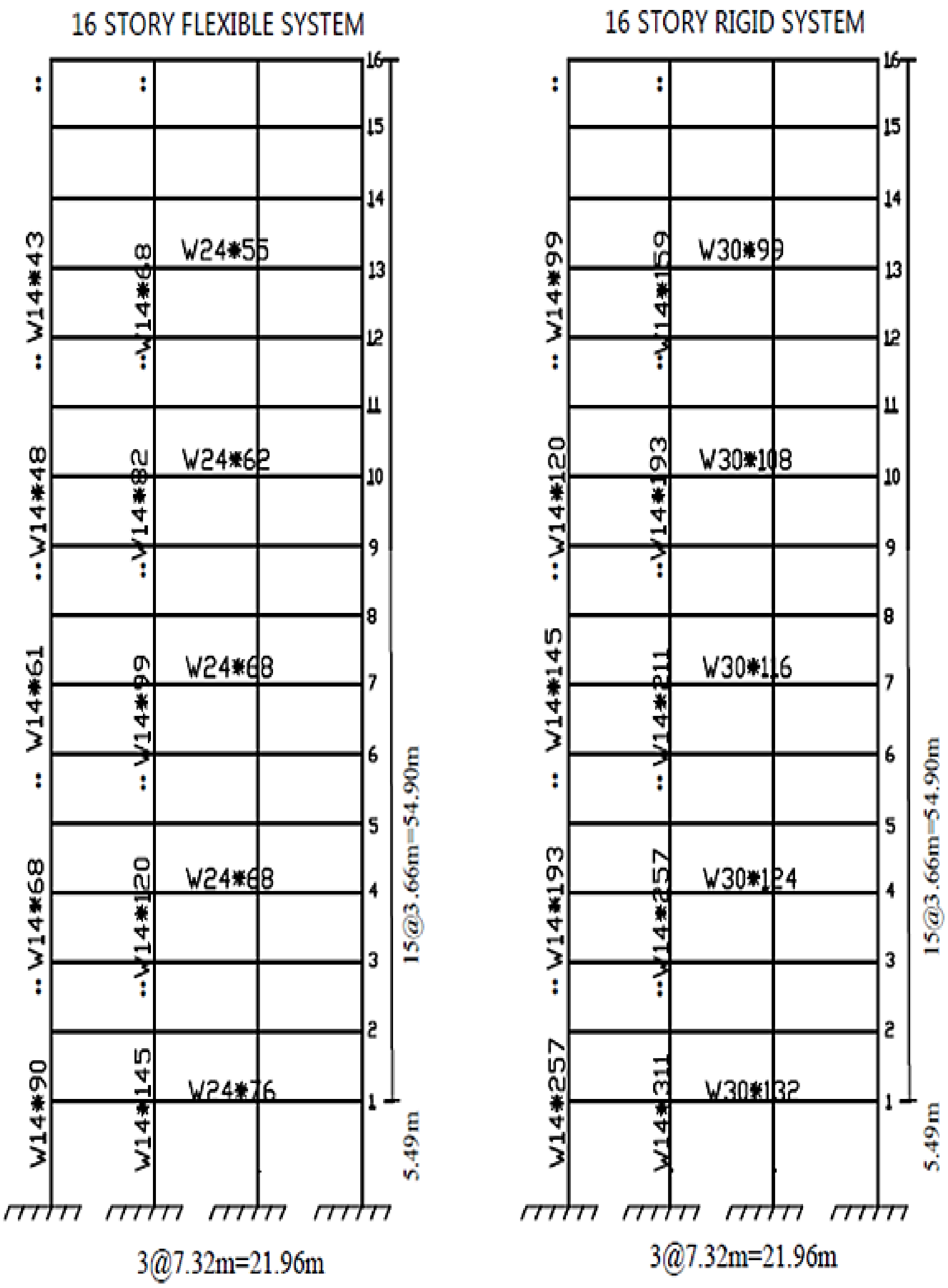


b)

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



a)



b)

Figure 3.3 12 and 16 stories flexible and rigid frames under consideration (Santa-Ana and Miranda, 2000)

`

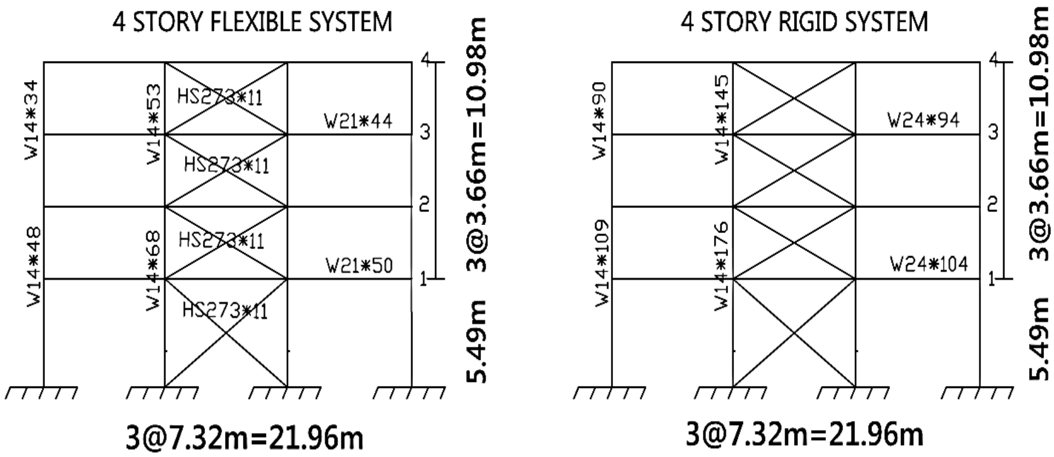
Table 3.1 Dynamic characteristics of the existing frame systems

Type of buildings	Natural periods (s)		
	T_1	T_2	T_3
4 story flexible unbraced	1.540	0.344	0.170
4 story rigid unbraced	0.847	0.190	0.092
8 story flexible unbraced	1.970	0.616	0.326
8 story rigid unbraced	1.080	0.349	0.170
12 story flexible unbraced	2.530	0.839	0.450
12 story rigid unbraced	1.350	0.440	0.237
16 story flexible unbraced	2.910	1.010	0.555
16 story rigid unbraced	1.659	0.580	0.322

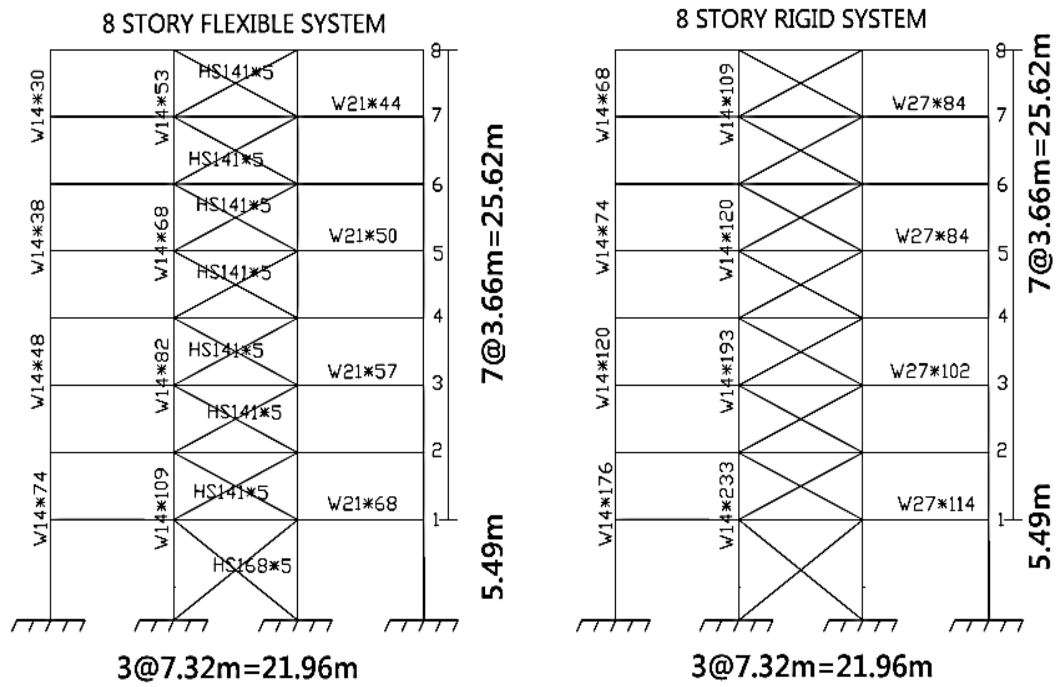
Considering two sets of target fundamental periods, the moment resisting frames with different number of stories were designed by Santa-Ana and Miranda (2000). These sets were selected to approximately supply upper and lower bounds for those recently obtained from earthquake records (Goel and Chopra, 1997). It is important to notice that even the rigid MRFs considered herein possess fundamental periods greater than those obtained using the formulation suggested by UBC (1994) for MRF buildings. Moreover, it was observed that the first two modes captured most of the response of the structure which was about 97%.

Then, X-braces were inserted into the middle bays of these frames as seen in Figures 3.4 and 3.5. A total of 16 cases were analyzed in this study (8 unbraced cases and 8 braced frames with X- braces). The performance of the existing frames and frames with X-braces was evaluated by using the finite element program of SAP 2000 non-linear version 14.

The dynamic properties of the braced frames are given in Table 3.2. As seen from Tables 3.1 and 3.2, the fundamental periods of the braced frames were considerably shorter than unbraced frame, which was also an indication that the braced frames were stiffer than unbraced frames.

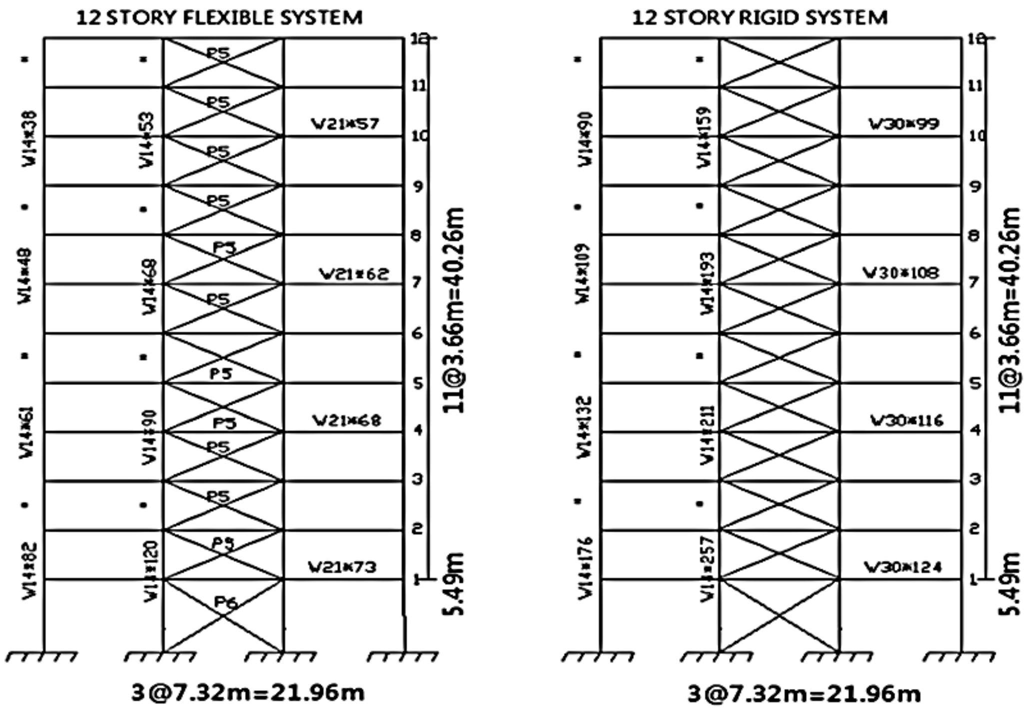


a)

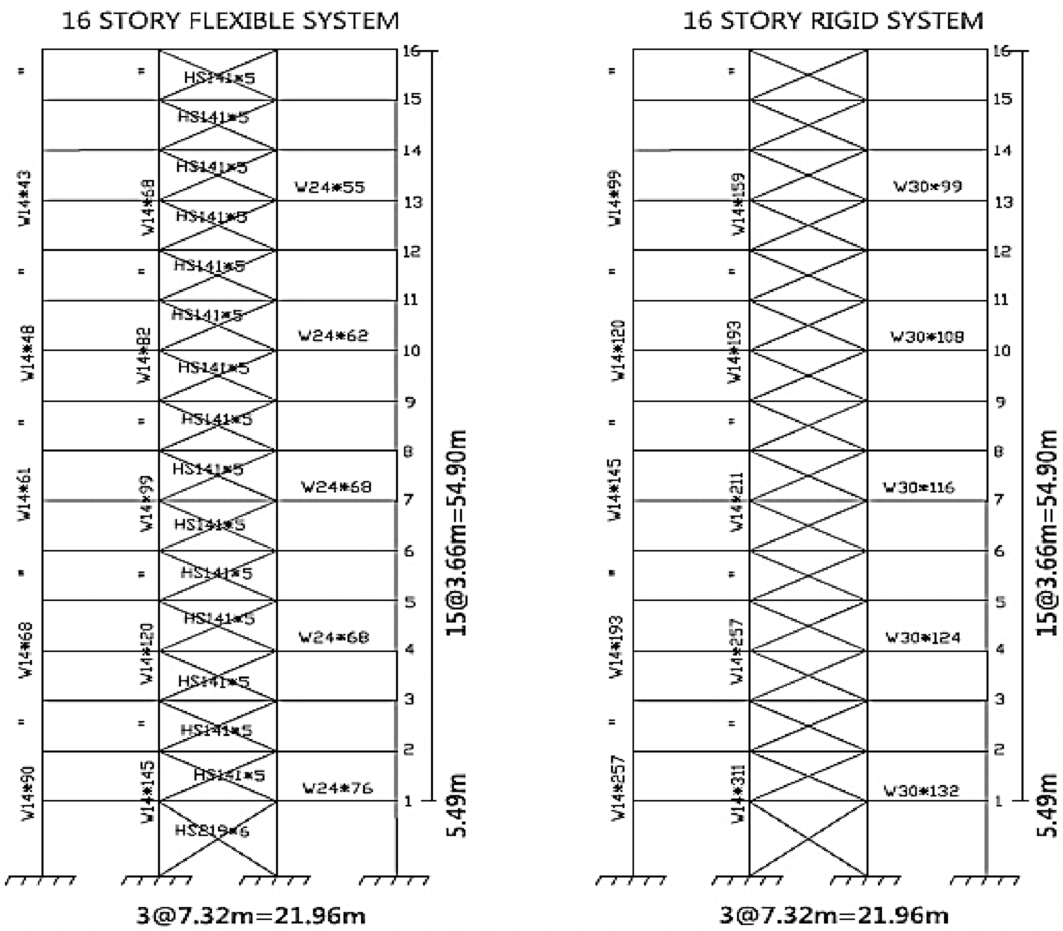


b)

Figure 3.4 4 and 8 stories flexible and rigid frames with X-braces analyzed in this study



a)



b)

Figure 3.5 12 and 16 stories flexible and rigid frames with X-braces analyzed in this study

`

Table 3.2 Dynamic properties of the X-braced frame systems

Type of buildings	Natural periods (s)		
	T_1	T_2	T_3
4 story flexible braced	0.275	0.090	0.070
4 story rigid braced	0.200	0.067	0.040
8 story flexible braced	0.766	0.260	0.143
8 story rigid braced	0.560	0.198	0.110
12 story flexible braced	1.103	0.356	0.189
12 story rigid braced	0.745	0.256	0.142
16 story flexible braced	1.569	0.543	0.300
16 story rigid braced	1.100	0.380	0.214

3.2 Method of the pushover analysis

Nonlinear static analysis or pushover analysis method provides the basis for transforming a dynamic problem to a static problem. It supplies an information about the response of the structure on the assumption that the response of the structure is controlled by the first few modes of vibration, and that this shape remains constant throughout the elastic and inelastic response of the structure. Furthermore, the response of a multi degree of freedom (MDOF) structure is related to the response of an equivalent single degree of freedom system (ESDOF) (Themelis, 2008). This concept is illustrated in Figure 3.6.

`

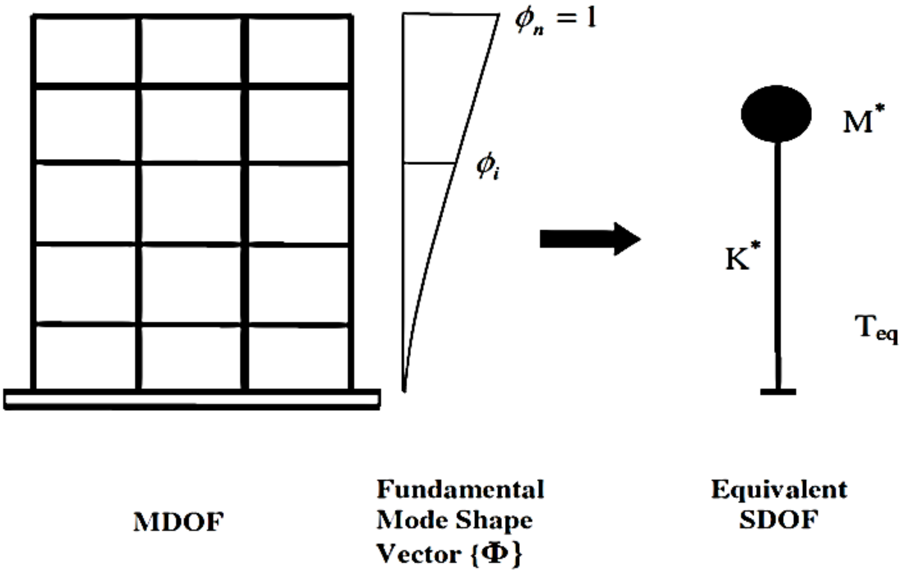


Figure 3.6 Diagram for transformation of MDOF to SDOF system (Themelis, 2008)

Elastic or inelastic MDOF systems which induced from the earthquake motion can derived from its governing differential equation:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+\{F\}=-[M]\{1\}\ddot{u}_g \qquad (3.1)$$

Where [M] is the mass matrix, [C] is the damping matrix, {F} is the storey force vector, {1} is an influence vector characterizing the displacements of the masses when a unit ground displacement is statically applied, and $\ddot{u}_g$ is the ground acceleration history.

By assuming a single shape vector, {Φ}, which is not a function of time and defining a relative displacement vector, U, of the MDOF system as U = $\{\Phi\}u_t$, where u_t denotes the roof or top displacement, the governing differential equation of the MDOF system will be transformed to:

$$[M]\{\Phi\}\ddot{u}_t+[C]\{\Phi\}\dot{u}_t+\{F\}=-[M]\{1\}\ddot{u}_g \qquad (3.2)$$

`

If the reference displacement u^* of the SDOF system is defined as:

$$u^* = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{1\}} u_t \tag{3.3}$$

Pre-multiplying equation (3.2) by $\{\Phi\}^T$ and substituting for u_t and using equation (3.3), the following differential equation describes the response of the ESDOF system:

$$M^*\ddot{u}^* + C^*\dot{u}^* + F^* = -M^*\ddot{u}_g \tag{3.4}$$

Where

$$M^* = \{\Phi\}^T[M]\{1\} \tag{3.5}$$

$$C^* = \{\Phi\}^T[C]\{\Phi\}\frac{\{\Phi\}^T[M]\{1\}}{\{\Phi\}^T[M]\{\Phi\}} \tag{3.6}$$

$$F^* = \{\Phi\}^T\{F\} \tag{3.7}$$

The MDOF structure which a nonlinear incremental static analysis can now be carried out from which it is possible to determine the force-deformation characteristics of the ESDOF system. The outcome of the analysis of the MDOF structure is a base shear, V_b, versus roof displacement, u_t, diagram, namely the global force-displacement curve or capacity curve of the structure, as shown Figure 3.7a. This capacity curve provides valuable information about the response of the structure because it estimates how it will behave and perform after exceeding its

`

elastic limit. About the post-elastic stage of the capacity curve some uncertainty exists, since the results obtained are dependent on the material models used (Pankaj and Lin, 2005) and the modelling assumptions (Dieirlein and Hsieh, 1990; Wight et al., 1997).

For simplicity, the curve is idealized as bilinear from which the yield strength V_y, an effective elastic stiffness $K_e = V_y/u_y$ and a hardening/softening stiffness $K_s = \alpha K_e$ were defined. The idealized curve can then be used together with equations (2.3) and (2.7) to define the properties of the equivalent SDOF system, Figure 3.7b. Thus the initial period T_{eq} of the equivalent SDOF system will be:

$$T_{eq} = 2\pi\sqrt{\frac{M^*}{K^*}} \tag{3.8}$$

Where K^*defines the elastic stiffness of the equivalent SDOF system and is given by:

$$K^* = \frac{{F_y}^*}{{u_y}^*} \tag{3.9}$$

The strain-hardening ratio, α, of the base shear–roof displacement relationship of the ESDOF system is taken as the same as for the MDOF structure.

`

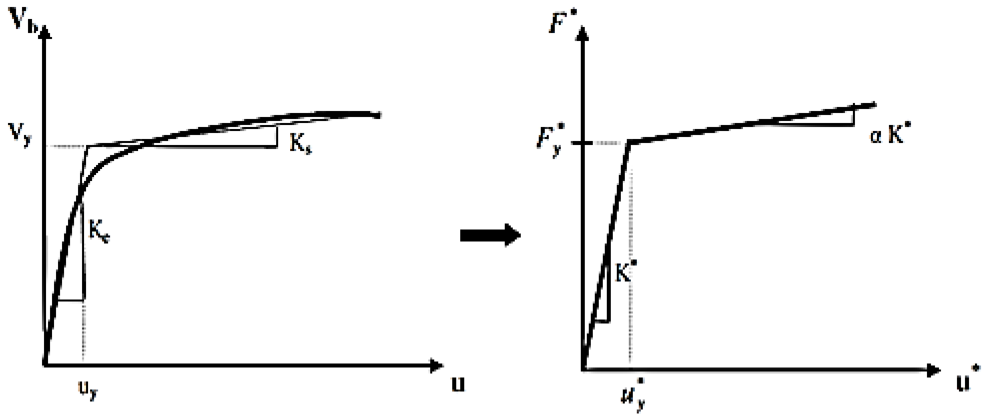


Figure 3.7 Views of a) capacity curve for MDOF structure and b) bilinear idealization for the equivalent SDOF system (Themelis, 2008)

Then, the corresponding displacement of the MDOF system can be estimated by re-arranging eq. (3.3) as follows:

$$u_t = \frac{\{\Phi\}^{\mathrm{T}}[\mathrm{M}]\{1\}}{\{\Phi\}^{\mathrm{T}}[\mathrm{M}]\{\Phi\}} u^* \tag{3.10}$$

u_t which is the target displacement is dependent on the choice of the mode shape vector $\{\Phi\}$.

The first mode-shape can provide accurate predictions of the target displacement if the response of the structure is dominated by its fundamental mode as shown in previous studies (Lawson et al., 1994; Fajfar and Gaspersic, 1996; Krawinkler and Seneviratna, 1998; Antoniou, 2002).

Pushover analysis is an approximate analysis method in that the structure was submitted to monotonically increasing lateral forces by a constant height-wise apportionment until a target displacement is reached. Pushover analysis can be performed as force-controlled or displacement controlled. The full load combination is

`

applied as specified, i.e., when the load is known (such as gravity loading) it called the force-controlled pushover procedure. Also, the lateral force can be applied as force-controlled but some numerical problems that affect the accuracy of results may occur since target displacement can be related with a particular small positive or negative lateral stiffness considering the evolution of mechanisms and P-delta effects. At the target displacement computed the internal forces and deformations are used to assess the inelastic strength and deformation requests for a performance check (Oğuz, 2005).

Generally, the pushover analysis is performed as a displacement-controlled proposed by (Allahabadi, 1987) to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. Increased or decreased the magnitude of load combined as necessary until the control displacement reaches a specified value. Generally, the control displacement is the roof displacement at the center of mass of the structure.

The definition of the material model is another very important aspect in the pushover analysis which was used to simulate the ductility of the structural members. Figure 3.9 shows the simplified force-deformation relationship used to model the beam elements or columns whose actions are controlled by deformation (FEMA-273/274, 1997; Bento et al., 2004).

For a structural number, the first line AB of the load deformation curve shows a linear response with a yield point at B. The inclination of the second line BC is usually low (0 to 10% of the value of the inclination of the elastic regime AB) and it represents some hardening. The third line CD represents the degradation of the resistive capacity while

`

the line DE corresponds to the plastification of the structural element. The criteria of acceptable deformation is also included by appropriate deformation ratios for primary elements (P) and secondary elements (S), which are also presented qualitatively in Figure 3.9 for three safety levels: Collapse Prevention (CP), Life Safety (LS) for the human life and Immediate Occupation (IO) for usefulness or serviceability of the structure. The values attributed to each point of the curve vary in function of the type of structural element, and they still depend on other parameters as specified in the ATC-40 (1996) and in the FEMA-356 (2000). In simple framed structures, the non-linear behavior occurs in sections or nodes that can be previously identified and introduced in the calculation model through hinges with non-linear behavior defined as given in Figure 3.9.

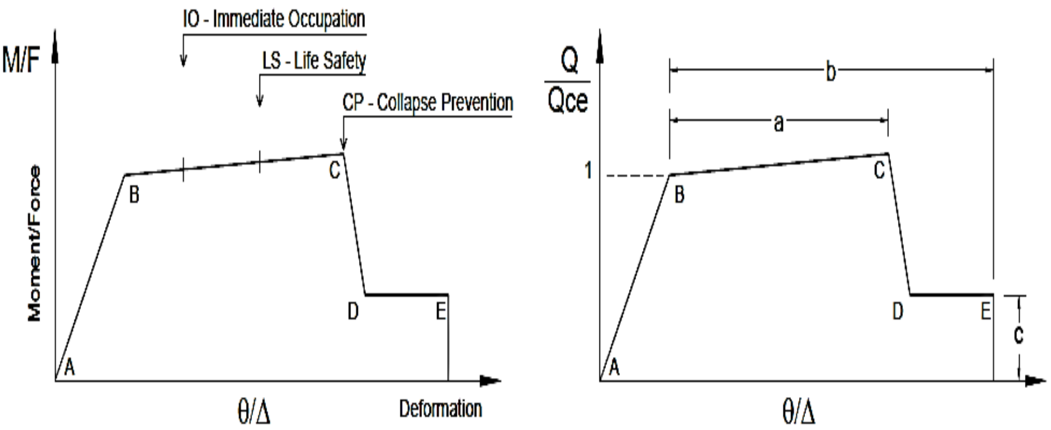
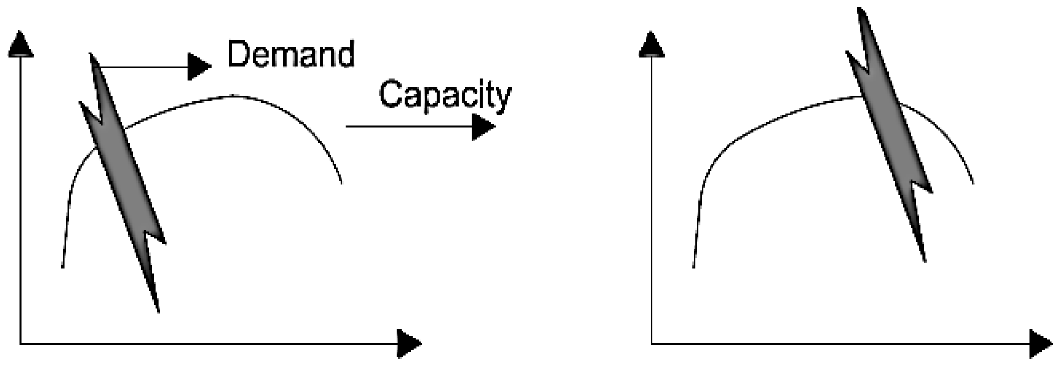


Figure 3.9 Constitutive relationship for pushover analyses (FEMA 356, 2000)

The main output of a pushover analysis is the capacity curve. If the demand curve intersects the capacity curve near the elastic range, Figure 3.10 a, it can be said that the structure have an excellent seismic resistance. If the demand curve crosses the capacity curve as shown in Figure 3.10 b, it can be said that the structure have little reserve of strength and deformation capacity (Kadid and Boumrkik, 2008).

`



a) b)

Figure 3.10 Typical seismic demand versus capacity: a) safe design and b) unsafe design (Kadid and Boumrkik, 2008)

Identification of plastic hinges formation is an important feature of pushover analysis as it can give the designer an insight into which parts of the structure need special considerations in the design process. However it has been shown that this ability of pushover methods in capturing local quantities such as the formation of plastic hinges diminishes as the structure grows taller (Rahnama and Krawinkler, 1994).

In the pushover analysis, various kinds of plastic hinges must be assigned to the beam elements and the column elements individually. For beam elements, plastic hinges are mainly produced by uniaxial bending moments (M3) while for column elements, plastic hinges are mainly produced by axial loads and biaxial bending moments (PMM) (Jianguo et al., 2006).

In order to perform a pushover analysis for a MDOF system, a pattern of increasing lateral forces needs to be applied to the mass points of the system. The purpose of this is to represent all forces which are produced when the system is subjected to earthquake excitation. By incrementally applying this pattern up to and into the inelastic stage, progressive yielding of the structural elements can be monitored.

`

During the inelastic stage the system will experience a loss of stiffness and a change in its vibration period. This can be seen in the force-deformation relationship of the system (Themelis, 2008).

The choice of the load pattern to capture a dynamic phenomenon through a static analysis is of much importance because it has been recognized that it can affect the results significantly (Lawson et al., 1994; Naeim and Lobo, 1998; Gupta and Kunnath, 1999; Mwafy and Elnashai, 2001; Lew and Kunnath, 2001; İnel et al., 2003).

Since the application of a single load pattern sometimes would not be able to capture the dynamic response of any system due to a seismic event, the standards of FEMA 356 and EC8 recommend to use at least two loads patterns in order to envelope the responses as shown in Table 3.3.

`

Table 3.3 Loading pattern: FEMA 356 and EC 8 (Braz-César and Barros, 2009)

FEMA 356	**EC (2003) – Method N2**
• Model*(fundamental mode) $F_i = \frac{m_i h_i^k}{\sum_{j=1}^{n} m_i h_i^k} F_b$ F_b- basal shear F_i- inertia forces at floor level i m_i- mass of floor level i h_i-coefficient associated with fundamental mode (height of floor level i) K=1.0 for T< 0.5s K=0.75+T/2 for 0.5≤T<2.5s K=2.0 for T≥2.5s *Can be multimodal (association of 3 first modes, as proposed by Chopra and Goel) • Uniform (see detail in EC8)	• Model $F_i = \frac{m_i \emptyset_i}{\sum_{j=1}^{n} m_i \emptyset_i} F_b$ F_b- basal shear F_i- inertia forces at floor level i m_i- mass of floor level i $\emptyset_i$- model coefficient at floor level i • Uniform $F_i = \frac{m_i}{\sum_{j=1}^{n} m_i} F_b$

`

CHAPTER 4

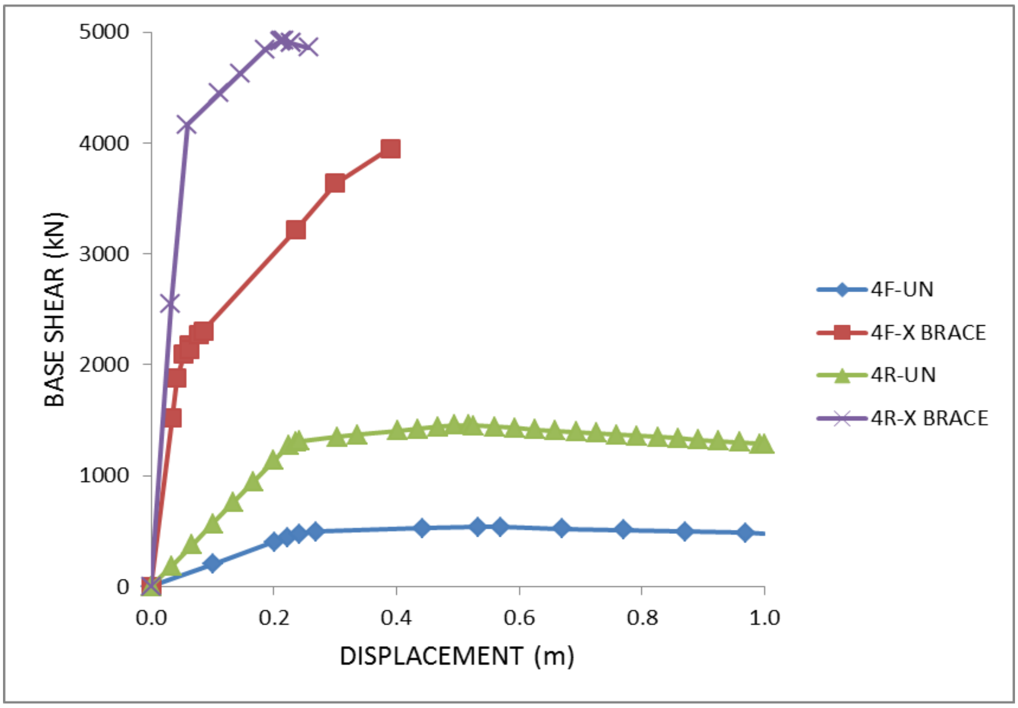
RESULTS AND DISCUSSION

4.1 Capacity curves

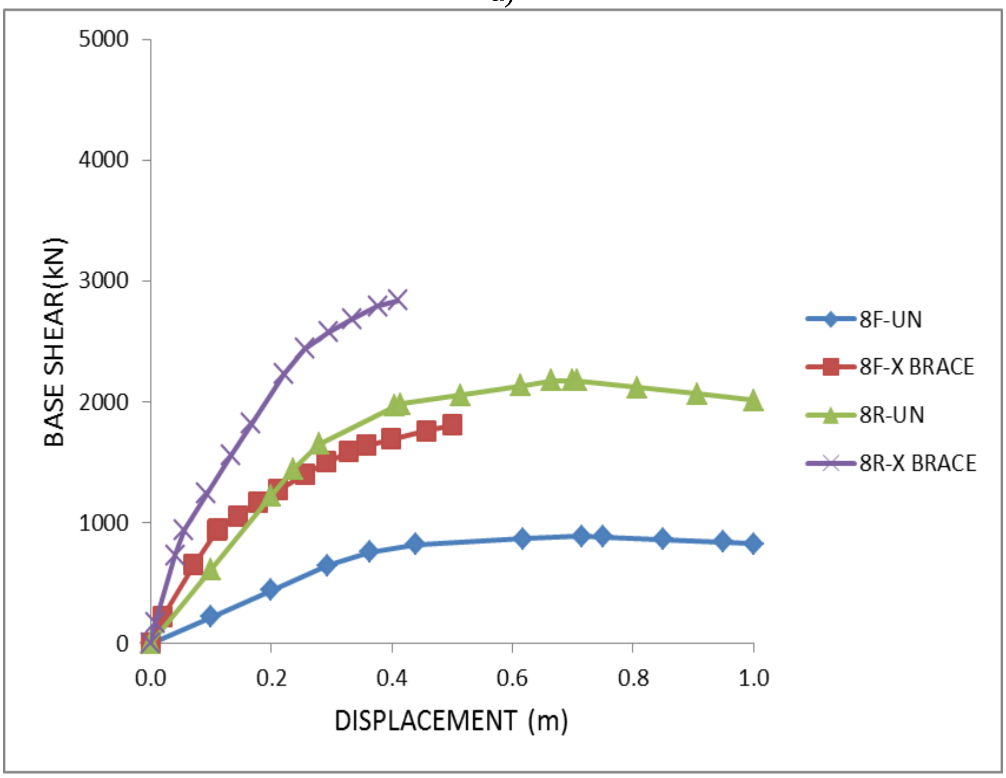
Figure 4.1 shows the comparison of the capacity curves of flexible and rigid frames with and without X-braces. It was certain and obtained that for both of frame systems, the X-braced frames were much stiffer and showed a better performance compared to the unbraced frames.

Rigid frames were much stiffer and exhibited a better work and performance compared to the flexible ones with brace and without brace under lateral loads, also putting the stiffness of the X-brace in perspective with unbraced frames, it was observed that in general the former was much stiffer than later. When the buildings were pushed well into the inelastic range, the curves became linear again but with a smaller slope. In the fact, the capacity curves in general for unbraced frames were bilinear since at the bases beginning the structure was globally in the elastic range and given a linear elastic slope, and 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 first change in the elastic slope was due to the yielding of the braces and the second change was due to the yielding of the structural members. Therefore, the length of the second slope was the delay between the yielding of the brace and the structural members. Also, it was observed that commonly the yielding of the brace members occurred at constant roof displacement.

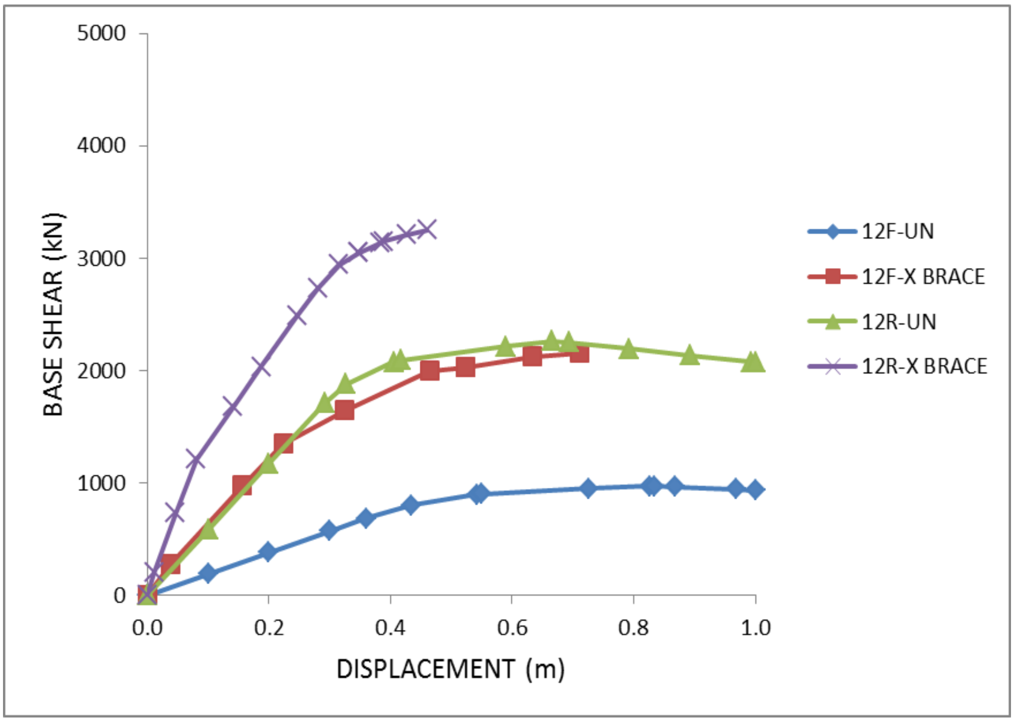

5000
4000
3000
2000
1000
0
BASE SHEAR (kN)
0.0
0.2
0.4
0.6
0.8
1.0
DISPLACEMENT (m)
4F-UN
4F-X BRACE
4R-UN
4R-X BRACE

a)

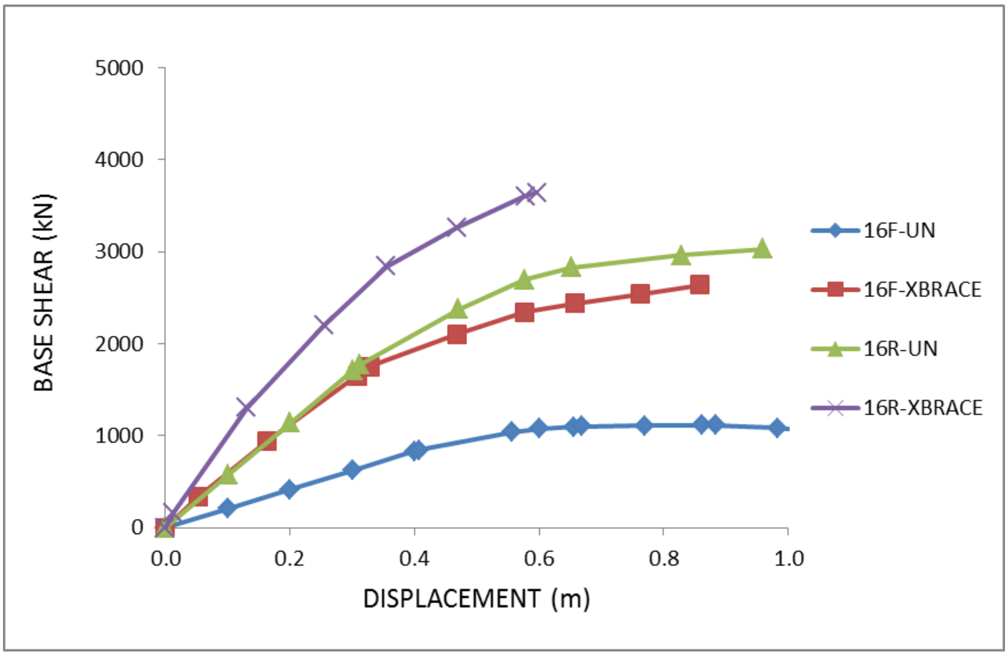

5000
4000
3000
2000
1000
0
BASE SHEAR(kN)
0.0
0.2
0.4
0.6
0.8
1.0
DISPLACEMENT (m)
8F-UN
8F-X BRACE
8R-UN
8R-X BRACE

b)

`



c)



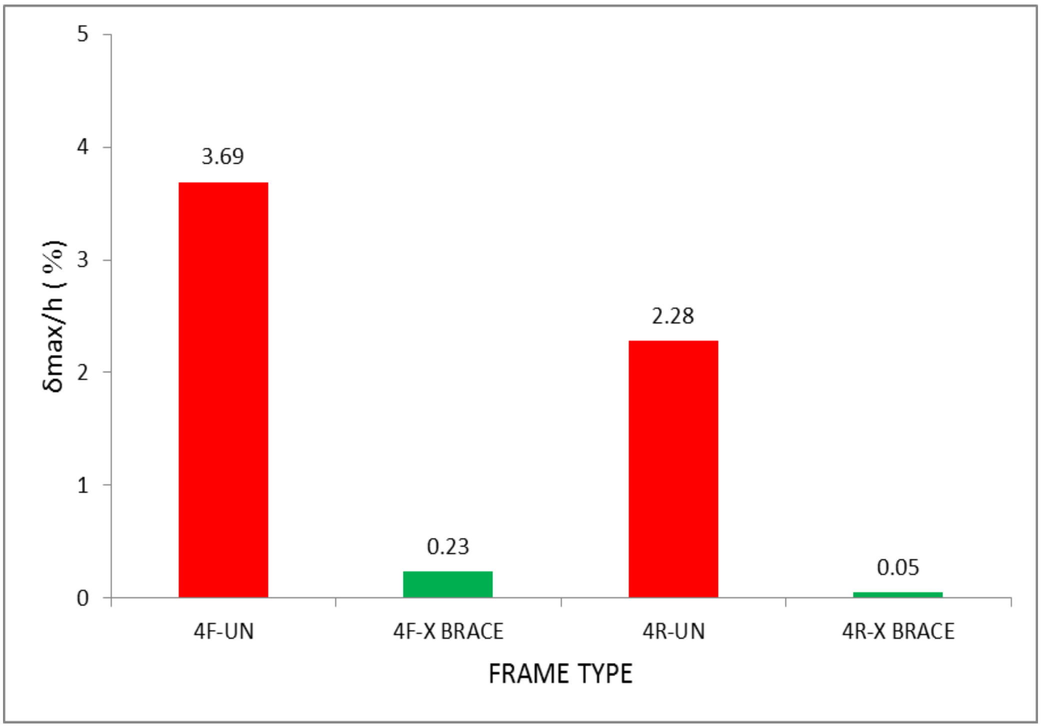
d)

Figure 4.1 Capacity curves for the unbraced and braced flexible and rigid frames of a) 4-storey, b) 8-storey, c) 12-storey, and d) 16-storey

`

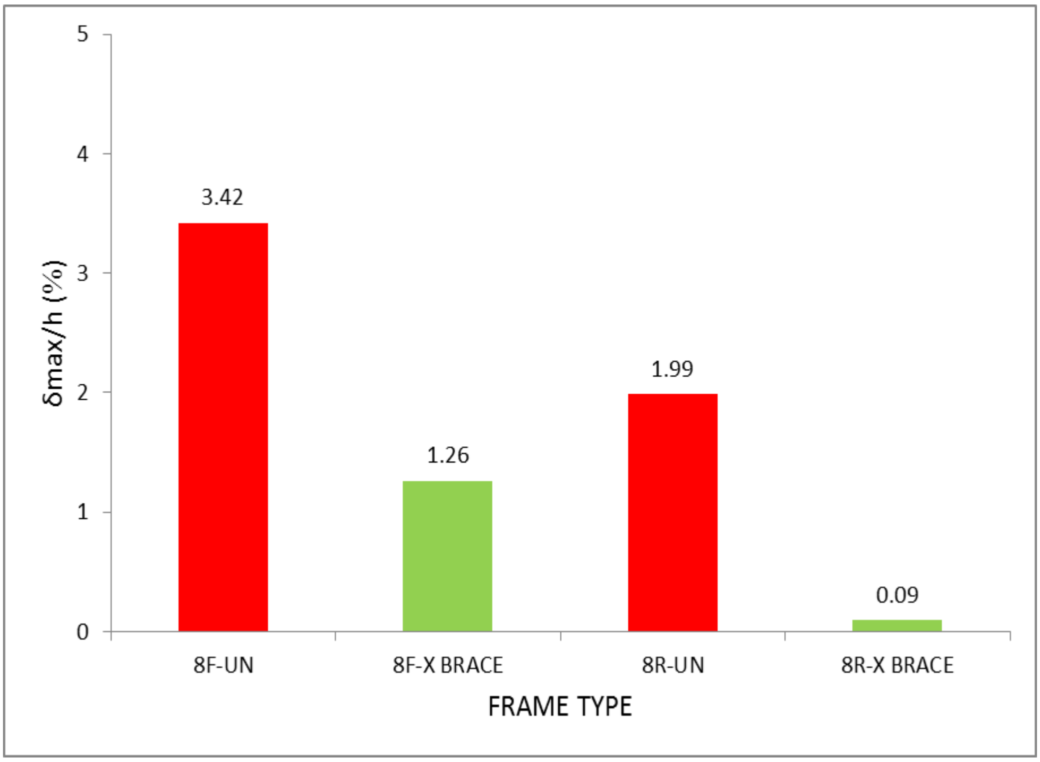
4.2 Inter-storey index

The defined maximum inter-storey index is the maximum interstorey drift (δmax) divided by the storey height (h). The index is related to a significant of the damage level experienced by the structural members. Figure 4.2 compares maximum inter-storey index for rigid and flexible frames with and without X-braces. The comparison of maximum inter-storey index indicated that this index for flexible frames was so much greater than for rigid frames. In both cases of braced and unbraced, it was pointed out from the response plots that a good performance could be observed in rigid frames and especially rigid with concentrated brace frames if compared to unbraced frames.

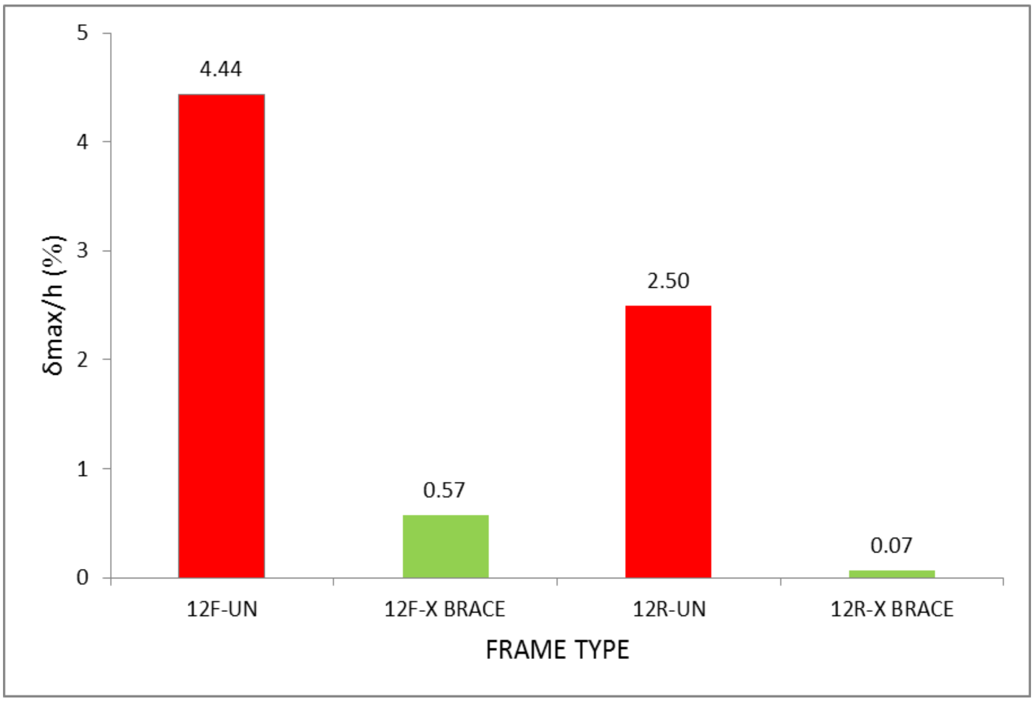


a)

`

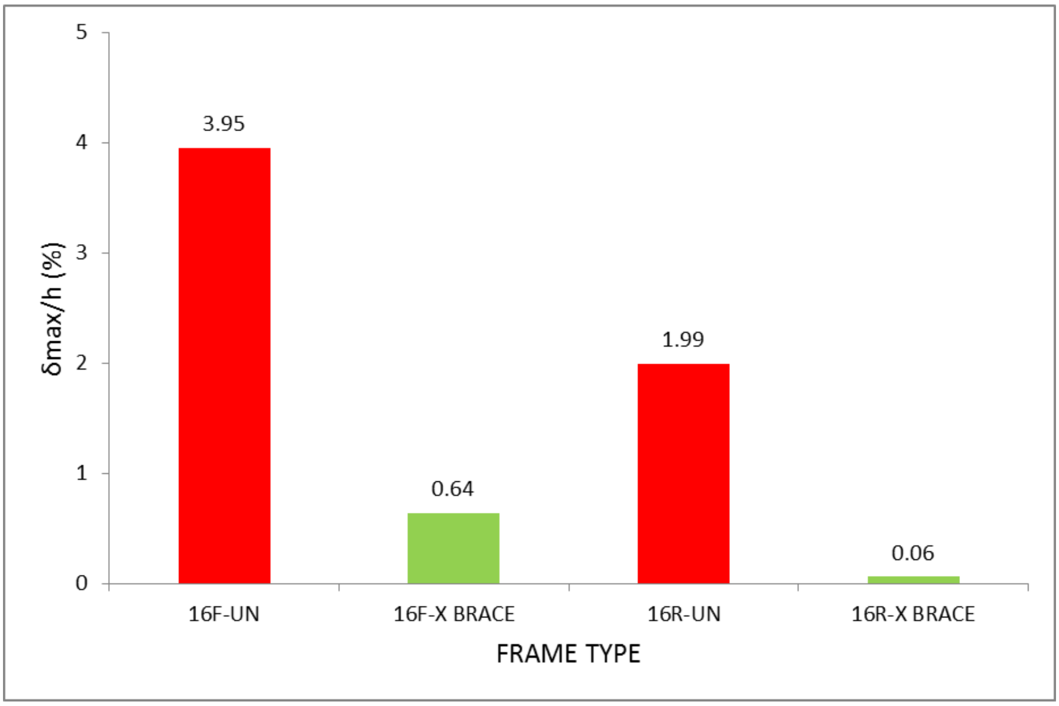

5
4
3
2
1
0
δmax/h (%)
3.42
1.26
1.99
0.09
8F-UN
8F-X BRACE
8R-UN
8R-X BRACE
FRAME TYPE

b)


5
4
3
2
1
0
δmax/h (%)
4.44
0.57
2.50
0.07
12F-UN
12F-X BRACE
12R-UN
12R-X BRACE
FRAME TYPE

c)

`



d)

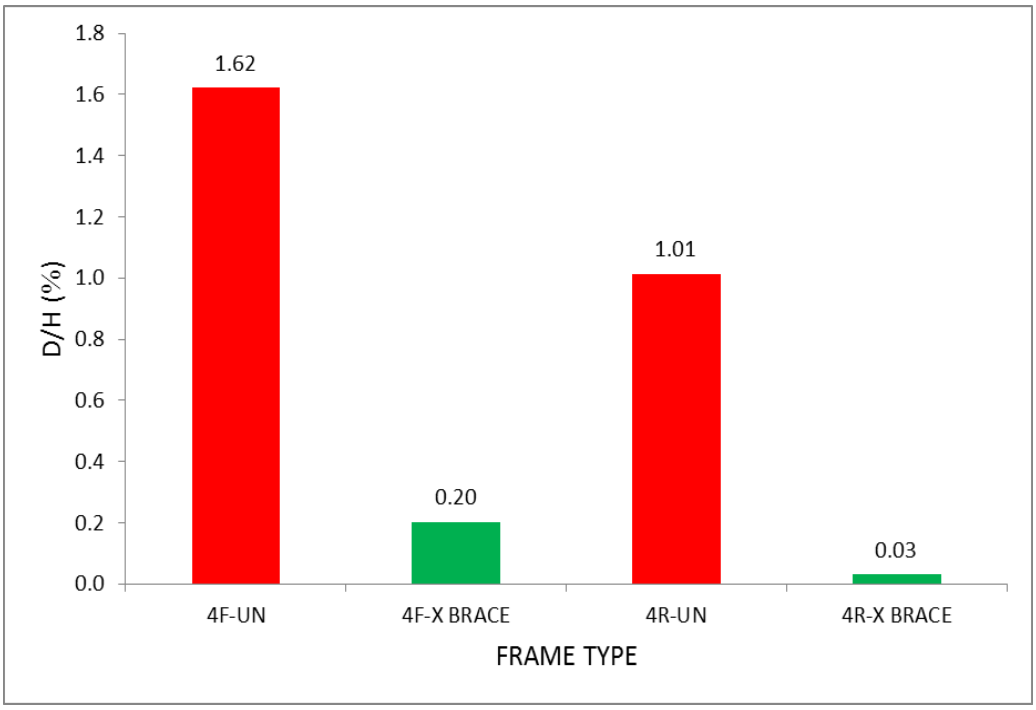
Figure 4.2 Maximum inter-storey indexes for the frames of a) 4-storey, b) 8-storey, c) 12-storey, and d) 16-storey

4.3 Global damage index

The roof displacement (D) ratio over the total height of the building (H) was defined as the global damage index. Figure 4.3 demonstrates the global damage index for rigid and flexible frames with and without concentrating braces. The global damage index was assessed for both types of frames. The computation or estimation of a global damage index of the frames (rigid and flexible) showed the flexible frames it was so much greater than rigid frames, and concentric brace frames showed a better performance in comparison to unbraced frames. The use of concentric brace system resulted in reduction of 10-70%. The magnitude of these global deformations depends mainly upon number of stories and especially characteristics of the frame (flexible or rigid systems).

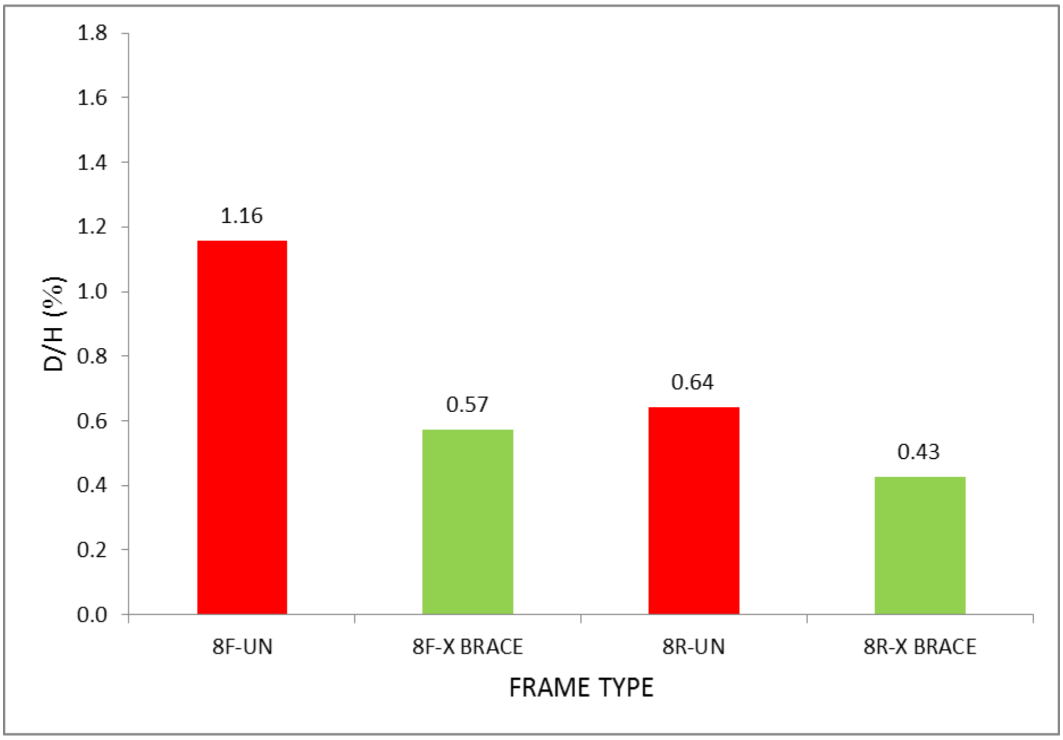
`

The global damage index is based on the evolution of local damage combined with an analysis of the probable collapse mechanism of the structure. The result of this index indicated that the various configurations of frames (number of stories, type of frames and frames with or without concentrating brace) had a greater effect on global performance of the structures. For example, in the case of four-storey buildings, the global index was higher than the other structures. Moreover, the inclusion of concentric brace frames 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.



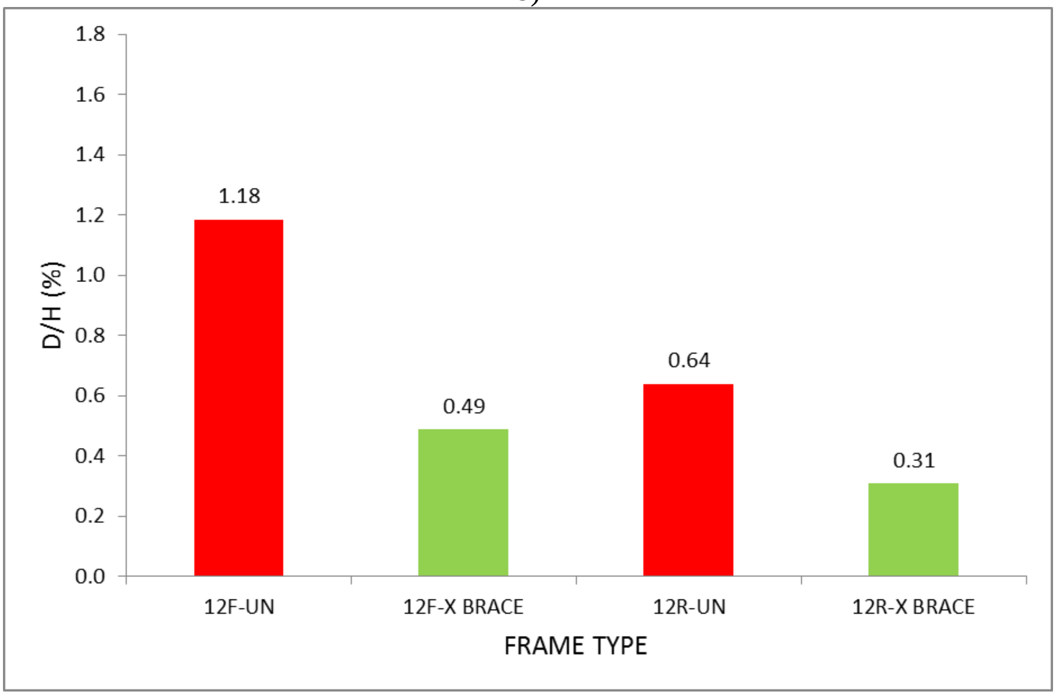
a)

`



1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
D/H (%)
1.16
0.57
0.64
0.43
8F-UN
8F-X BRACE
8R-UN
8R-X BRACE
FRAME TYPE

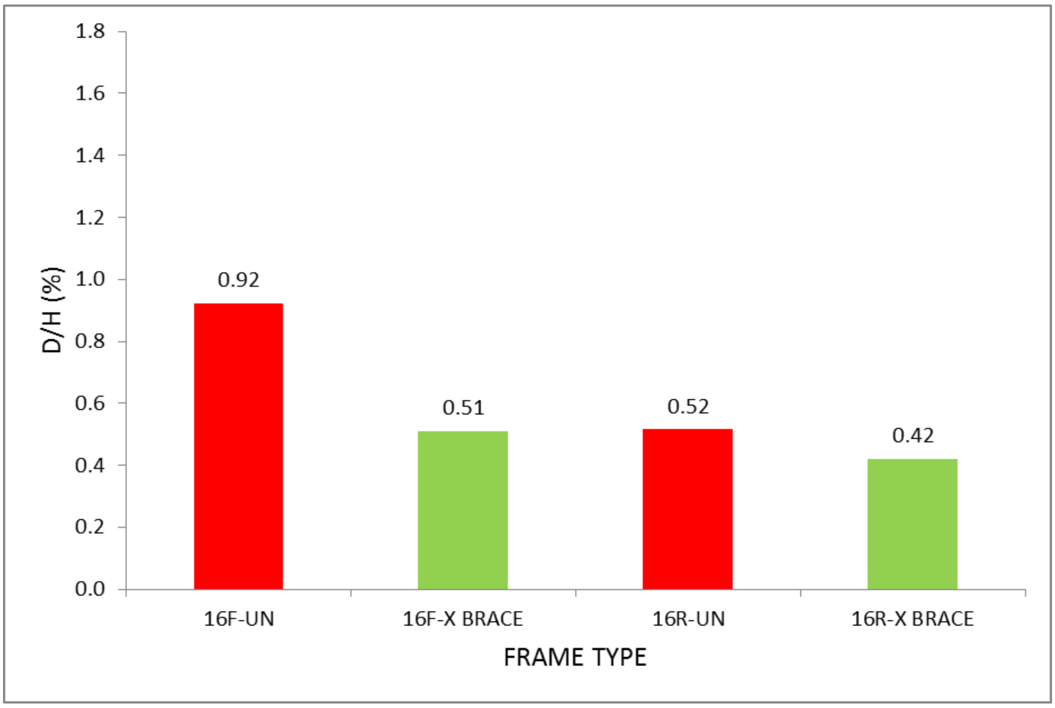
b)



1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
D/H (%)
1.18
0.49
0.64
0.31
12F-UN
12F-X BRACE
12R-UN
12R-X BRACE
FRAME TYPE

c)

`



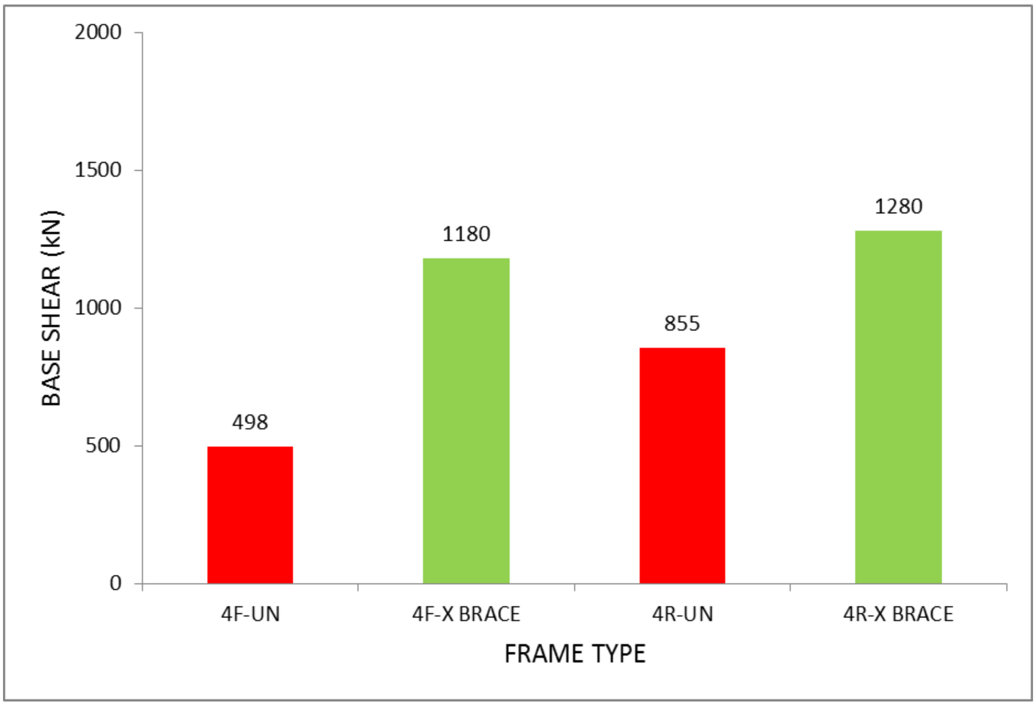
d)

Figure 4.3 Global damage indexes for the frames of a) 4-storey, b) 8-storey, c) 12-storey, and d) 16-storey

4.4 Total base shear

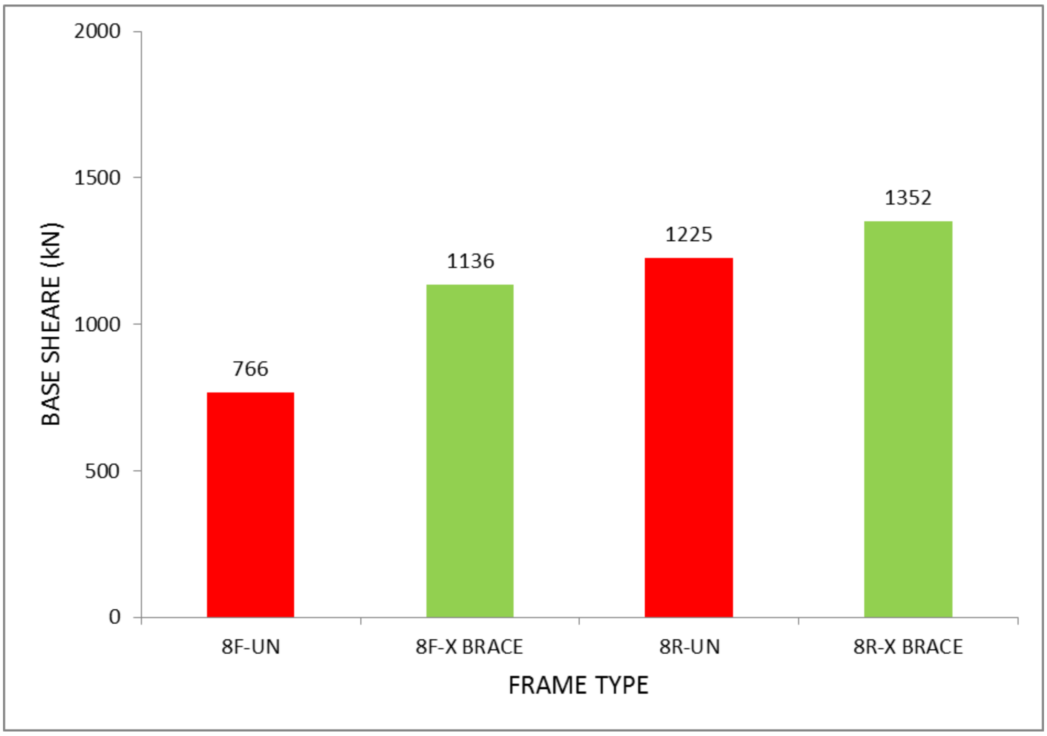
The base shear obtained from the pushover analysis is presented in Figure 4.4. Type of frames and lateral loads influenced the variety of base shear forces. The base shear was increased in the cases of brace frames, but the columns were not influenced so much by this increment, because most of the shear force was supported by the braces, and that increased by rigidity frames. The base shear was also affected by type of frame and number of stories of the structure. For example, in the case of stiff concentrated brace frames, the base shear was greater than flexible concentrated brace frames and by the increasing number of stories the base shear was also increased. Thus, the largest base shear was observed in the case of the sixteen stories stiff concentrated brace frames.

`



2000
1500
1000
500
0
BASE SHEAR (kN)
498
1180
855
1280
4F-UN
4F-X BRACE
4R-UN
4R-X BRACE
FRAME TYPE

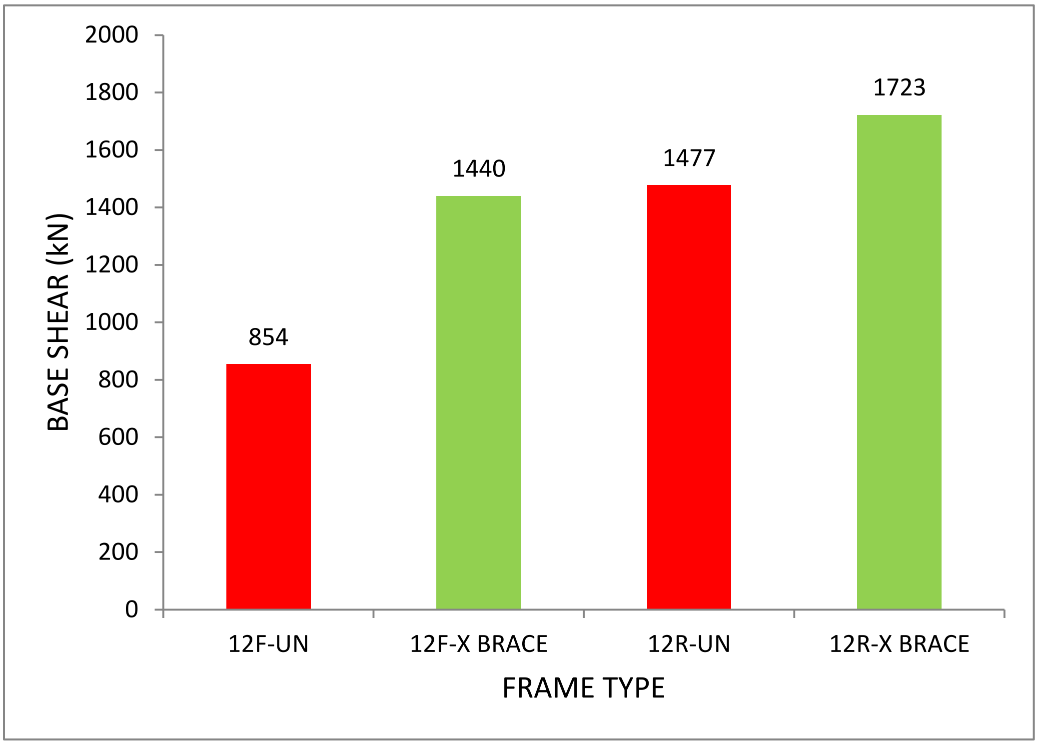
a)



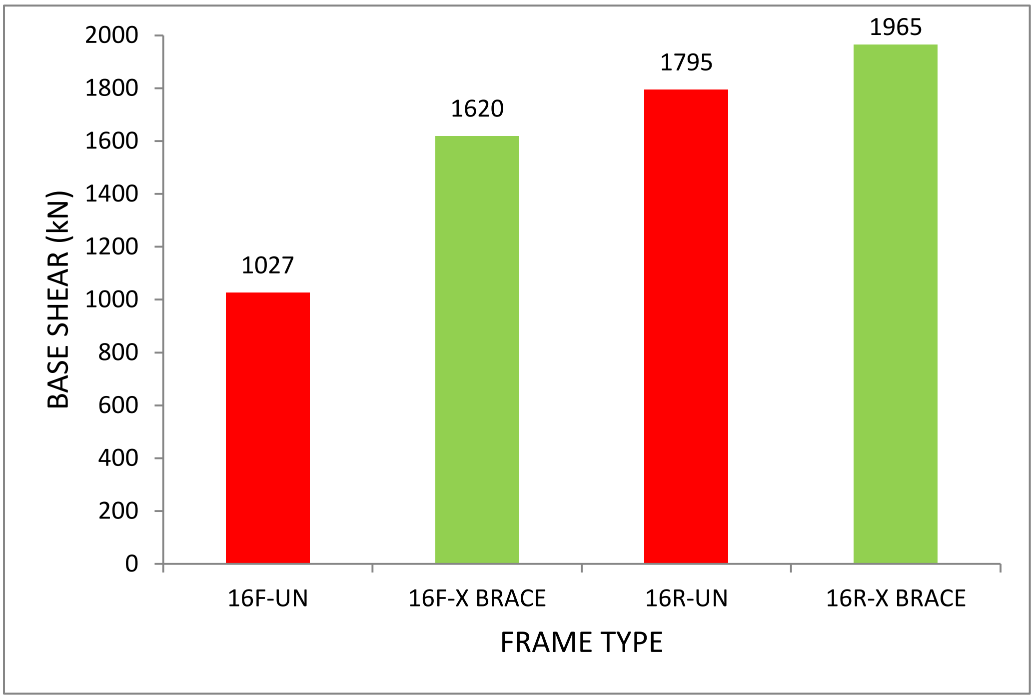
2000
1500
1000
500
0
BASE SHEARE (kN)
766
1136
1225
1352
8F-UN
8F-X BRACE
8R-UN
8R-X BRACE
FRAME TYPE

b)

`



c)



d)

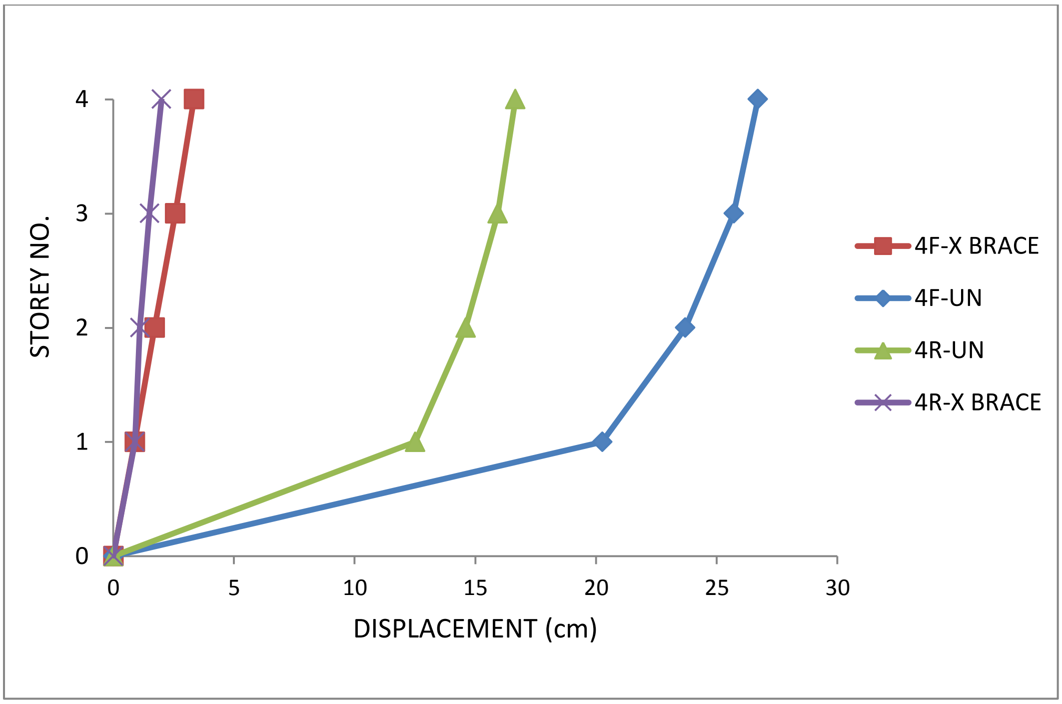
Figure 4.4 Total base shears for the frames of a) 4-storey, b) 8-storey, c) 12-storey, and d) 16-storey

`

4.5 Variation of storey displacement

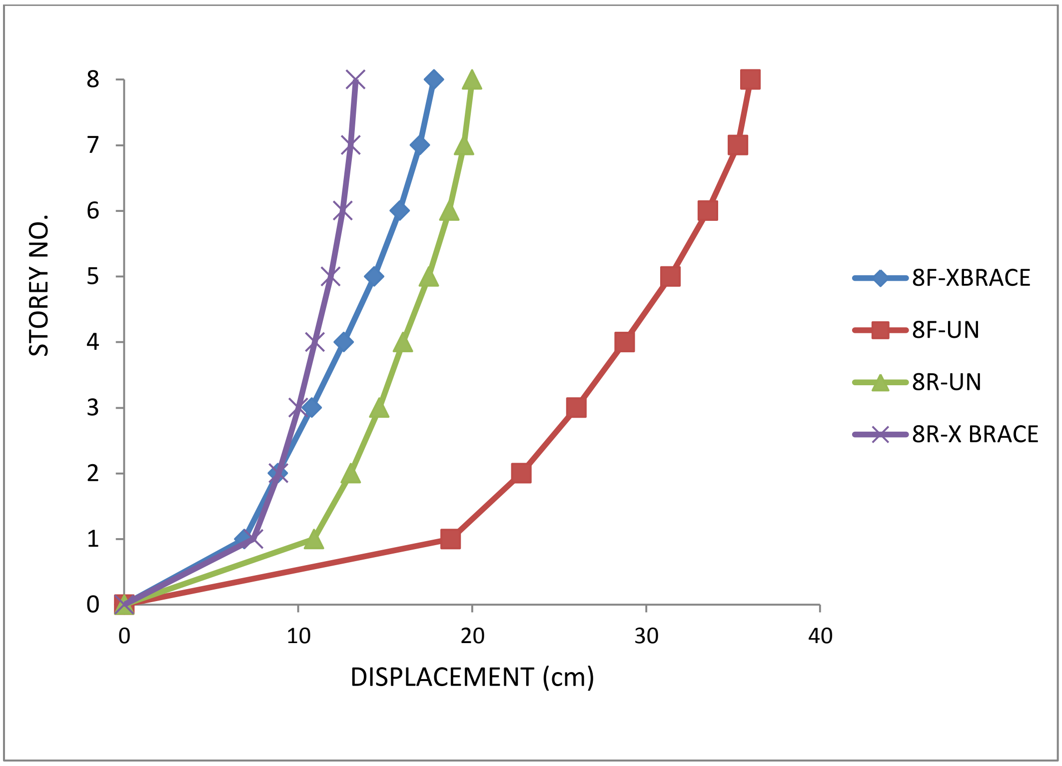
The storey displacement variation of both types of frames (flexible and rigid) with and without X-braces is shown in Figure 4.5. The rigid and flexible frames with the inclusion of concentric brace considerably decreased the value of maximum roof displacement and corresponding storey displacement compared to unbraced frames, especially in the case of rigid frames. Frames with concentric brace yielded very uniform response along the height of the structure and there was not a 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 was also influenced by the number of stories and frame type. For example, in the case of four stories stiff X-brace frames, the maximum storey displacement was smaller than other frames, by an increasing number of stores the maximum storey displacements were also raised.



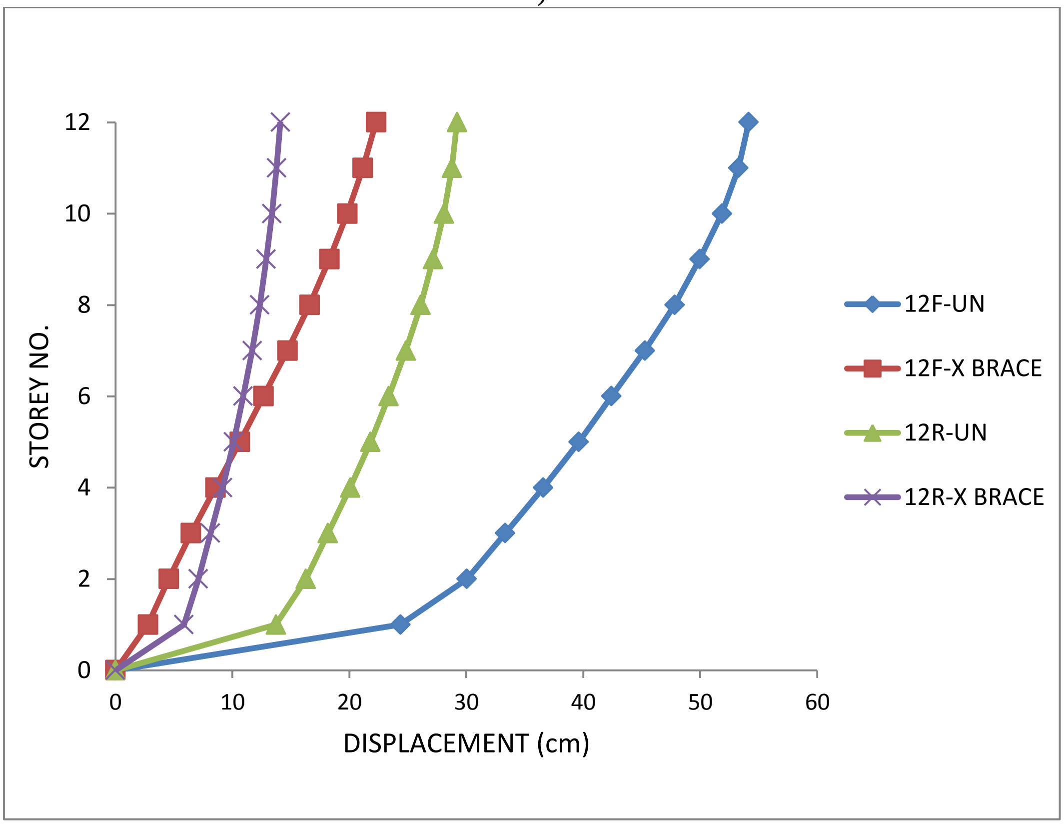
a)

`



STOREY NO.
8
7
6
5
4
3
2
1
0
0
10
20
30
40
DISPLACEMENT (cm)
8F-XBRACE
8F-UN
8R-UN
8R-X BRACE

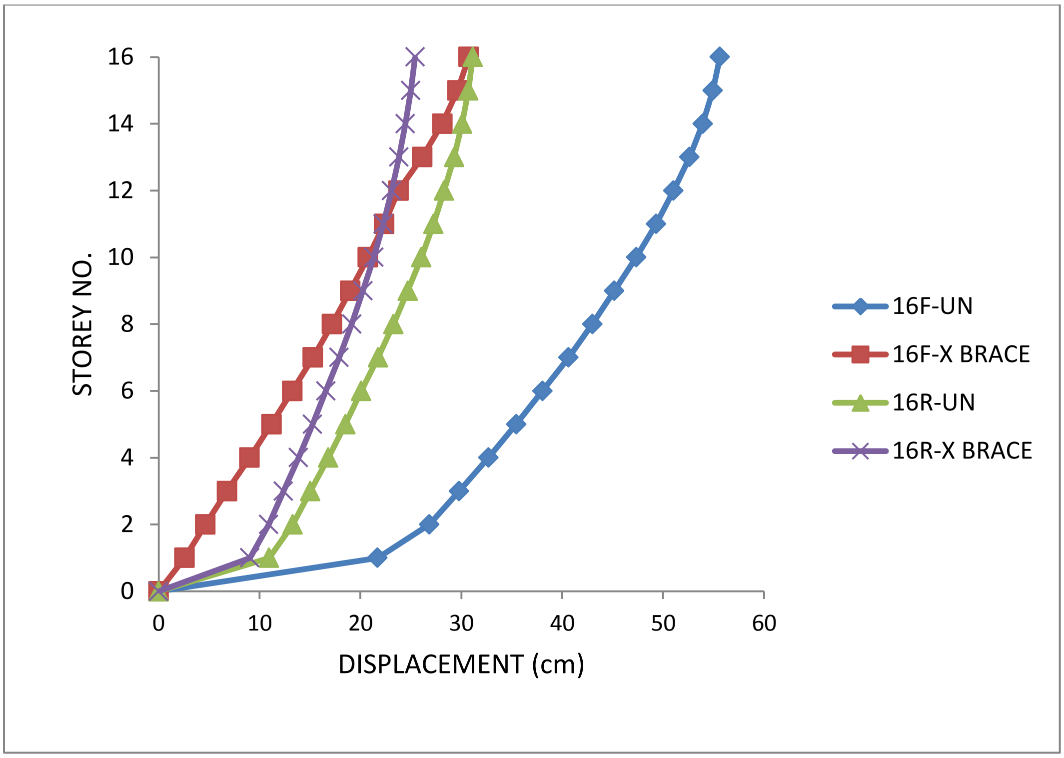
b)



STOREY NO.
12
10
8
6
4
2
0
0
10
20
30
40
50
60
DISPLACEMENT (cm)
12F-UN
12F-X BRACE
12R-UN
12R-X BRACE

c)

`



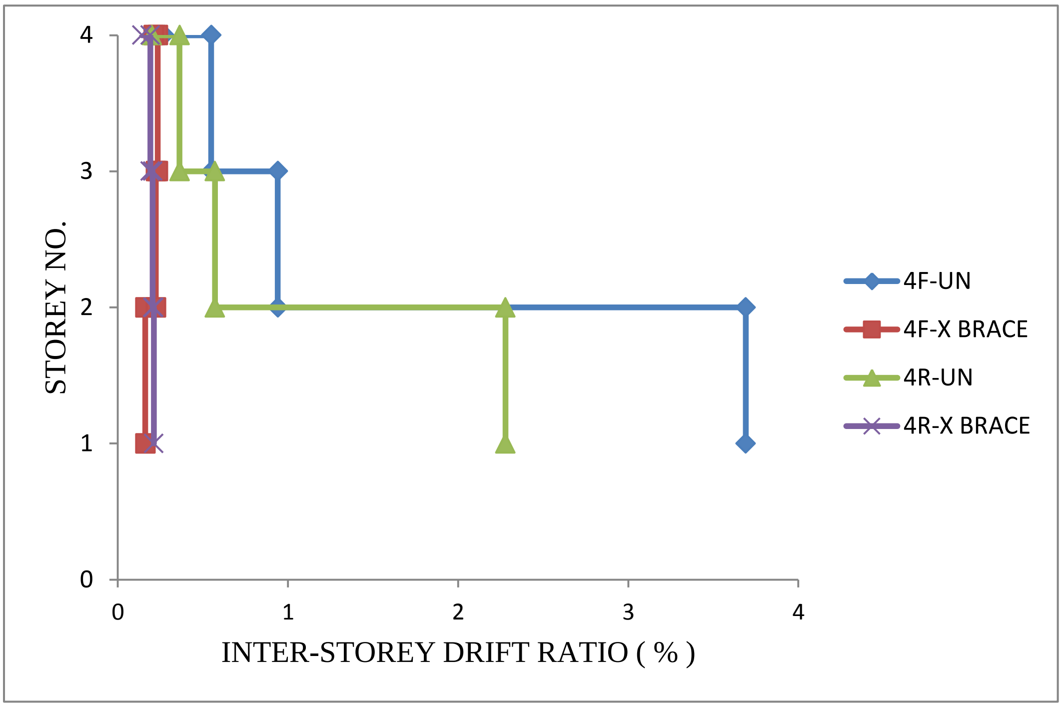
d)

Figure 4.5 Storey displacement versus storey no. for the different frames

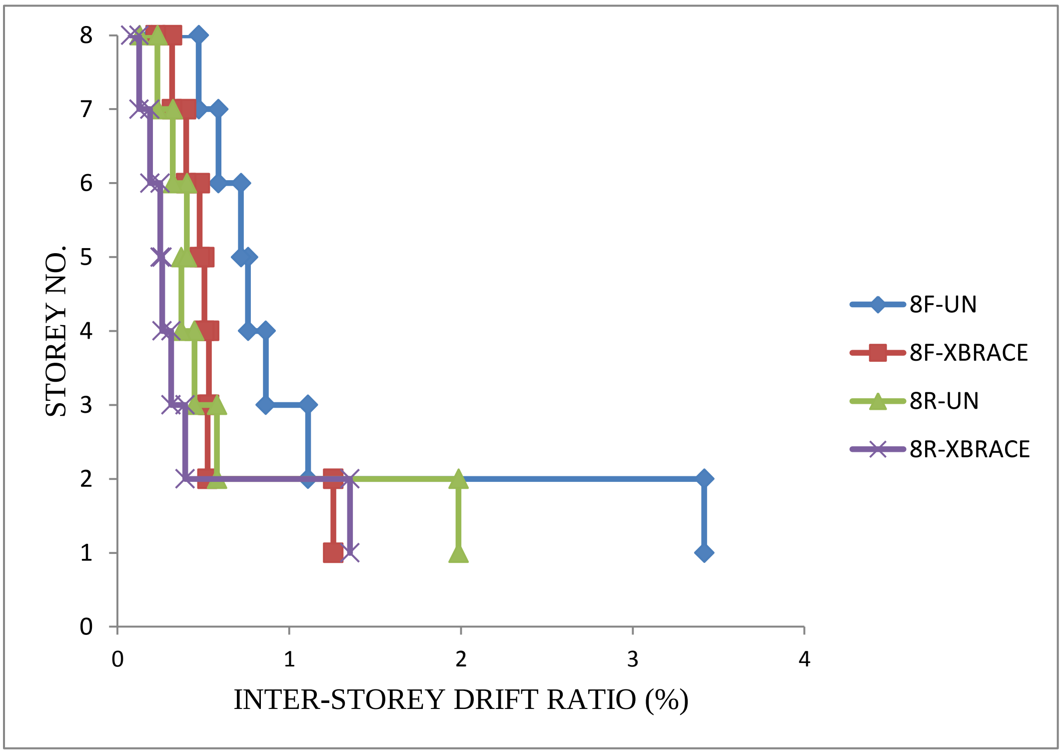
4.6 Inter-storey drift ratio

The previous studies indicated that steel buildings could experience significant lateral deformations after an earthquake ground motion (Pampanin et al., 2003; Ruiz-Garcia and Miranda, 2006). Therefore, inter-storey drift demands over height in the unbraced and braced frames were evaluated as seen in Figure 4.6.

Generally, the inclusion of the bracing element into the frames resulted in a considerable reduction in drift values of the frame systems. For instance, the drift ratio for the unbraced flexible eight storey frame were about 3.4%, however, the effect of X-brace on the same frame was observed that the drift demands was smaller and had peak inter-storey drift of about 1.35%. The figure also revealed that the stiff frames exhibited a better performance than the flexible ones.

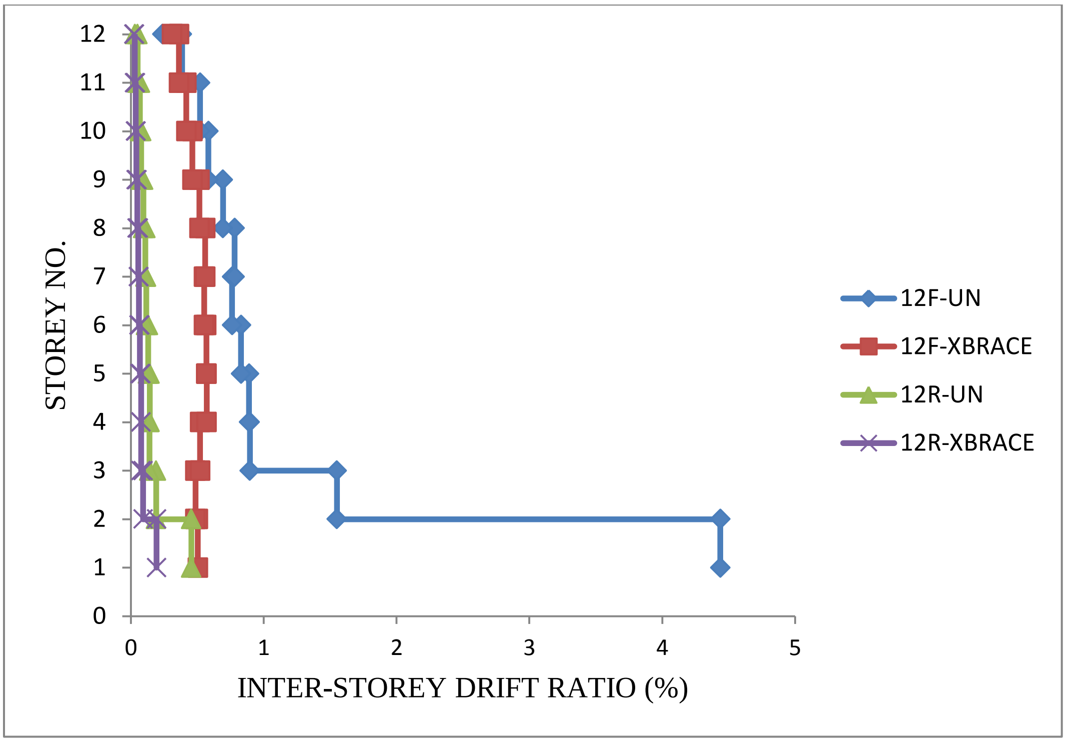

STOREY NO.
INTER-STOREY DRIFT RATIO (%)
4F-UN
4F-X BRACE
4R-UN
4R-X BRACE

a)


STOREY NO.
INTER-STOREY DRIFT RATIO (%)
8F-UN
8F-XBRACE
8R-UN
8R-XBRACE

b)

`



c)

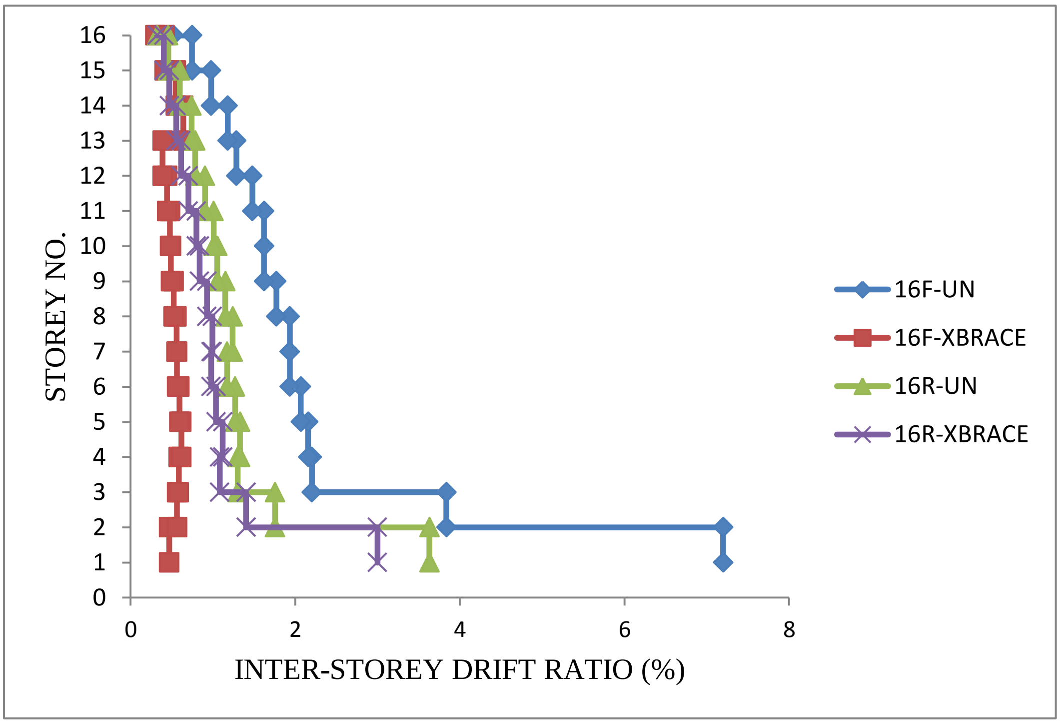


Figure 4.6 Interstorey drift ratio versus storey no. for different frames

`

CHAPTER 5

CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn:

1. Pushover analysis can provide insight into the elastic as well as the inelastic response of buildings when subjected to lateral loads. The results obtained in terms of capacity-demand gave an insight into the real behavior of structures. The steel-braced systems possessed much larger ductility capacities than their equivalent unbraced frames.

2. From the capacity curves, it was evident that the base shear, which was the capacity of the frame to resist lateral loads, was considerably increased in the presence of braces. It was very noticeable in the case of stiff braced frame systems.

3. Based on the concept of using steel braces in strengthening or retrofitting the existing structures, the steel bracings reduce the flexure and shear demands in beams and columns and transferred the lateral loads throughout the axial load mechanism. Regardless of the frame design properties, the utilization of X-brace frames supplied smaller inter-storey drift index in comparison to unbraced frames. Up to 50% reduction in the inter-storey drift value with respect to the original frames was observed, which confirmed that the X-brace was a very effective bracing element to support the frames under the lateral loading.

`

4. The analysis of the results showed that the rigid frames had better structural performance than the flexible ones, irrespective of the number of storey. On the other hand, the plastic hinges were generated in the beam elements and most of the columns had remained in the elastic stage. Generally, it was noticed that there were a lot of columns which had an undesirable failure mode of deformation result in the inelastic range.

5. It was pointed out that pushover analysis had a useful tool of performance based seismic design of a structure by providing the post-yield behavior of a structure and it could be used to assess the performance of different types of structures by controlling their displacements (at local and global levels) and it give a useful data about the behavior of structure under seismic action until the structural collapse.

`

REFERENCES

AIJ (1995). Architectural Institute of Japan. Steel committee of Kinki branch, *reconnaissance report on damage to steel building structures observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) earthquake*, Tokyo, 167 pages.

AISC. (American Institute of Steel Construction). (1997). Seismic Provisions for Structural Steel Buildings, Chicago.

AISC. (American Institute of Steel Construction). (2000).Load and Resistance Factor Design Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago, IL.

Astaneh A., Goel S., Hanson R. (1985). Cyclic Out-of-Plane Buckling of Double-Angle Gusset Plates, *Journal of Structural Engineering*, ASCE **Vol. 111** (5), 1135-1153.

ATC-40. (1996).Seismic evaluation and retrofit of concrete buildings. Report ATC-40, Applied Technology Council, Redwood City CA.

Allahabadi R. (1987). Drain 2DX – Seismic Response and amage Assessment for 2D Structures, Ph.D. *Thesis*, University of California at Berkeley, California.

Antoniou S. (2002). Advanced Inelastic Static Analysis for Seismic Assessment of Structures, PhD Thesis, Engineering Seismology and Earthquake Engineering Section, Imperial College, London, UK.

`

Annan C. D., Youssef M. A., Elnaggar M. H. (2009). Seismic Overstrength in Braced Frames of Modular Steel Buildings, *Journal of Earthquake Engineering*, **13**:1–21.

Antoniou S., Pinho R. (2004a). Advantages and limitations of adaptive and non-adaptive force-based pushover procedures, *Journal of Earthquake Engineering*, **Vol. 8** No. 4, pp. 497-522.

Antoniou S., Pinho R. (2004b). Development and verification of a displacement-based adaptive pushover procedure, *Journal of Earthquake Engineering*, **Vol. 8**, No. 5, pp. 643-661.

Aiken I. (1992). Comparative study of four passive energy dissipation systems, *Bulletin of the New Zealand National Society for Earthquake Engineering*, **25**, 3, pages 175-192.

Braz-César M. T., Barros R. C. (2009). Seismic performance of metallic braced frames by pushover analyses, COMPDYN 2009, *ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Rhodes, Greece, 22–24.

Bertero V.V., Aktan A.E., Charney F.A., Sause R. (1984). U.S.-Japan Cooperative Earthquake Research Program, Earthquake Simulation Tests and Associated Studies of a l/5th Scale Model of a 7-Story Reinforced Concrete Test Structure, Report No. UBC/EERC-84/05, University of California, Berkeley.

Baik S.W., Lee D.G., Krawinkler H. (1988). A simplified model for seismic response prediction of steel frame structures, *Proc. 9th world conference earthquake engineering. Tokyo, Kyoto,* **Vol. 5**, pp. 375-380.

`

Broderick BM, Elnashai AS, Ambraseys NN, Barr JM, Goodfellow RG, Higazy EM. (1994). Observations strong motion and correlative response analysis. Engineering seismology and earthquake engineering, research report no. ESEE 94/4, London, Imperial College, The Northridge (California) earthquake of 17 January.

Bruneau M, Uang CM., Whittaker A. (1998). Ductile design of steel structures, New York (USA): *McGraw-Hill*.

Broderick BM, Goggins JM, Elghazouli AY. (2005). Cyclic performance of steel and composite bracing members, *Journal of Constructional Steel Research*, **61** (4): 493–514.

Bozorgnia Y., Bertero VV. (2004). Earthquake engineering. From engineering seismology to performance-based engineering, Boca Raton (FL, USA): CRC Press, 2004.ISBN 0-203-48624-2 Master e-book ISBN, International standard book number 0-8493-1439-9 (print edition), library of congress card number 2003065422.

Bento R., Falcão S., Rodrigues F. (2004). Avaliação sísmica de estruturas de edifícios com base em análises estáticas não lineares, *SISMICA 2004 – 6º Congresso Nacional de Sismologia e Engenharia Sísmica, Guimarães.*

Chopra A.K., Goel R.K. (2001). A Modal Pushover Analysis Procedure to Estimating Seismic Demands for Buildings, Theory and Preliminary Evaluation, PERR Report 2001/03, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Chintanapakdee C., Chopra A.K. (2003). Evaluation of Modal Pushover Analysis Using Generic Frames, *Earthquake Engineering and Structural Dynamics*, **Vol. 32**, 417-442.

`

Deierlein G., Hsieh S.H. (1990). Seismic response of steel frames with semi-rigid connections using the capacity spectrum method, *Proceedings 4th US National Conference on Earthquake Engineering,* **Vol. 2**, 863-72.

Di Sarno L, Elnashai AS. (2002). Seismic retrofitting of steel and composite building structures, Mid-America earthquake center report, CD Release 02-01. IL (USA): University of Illinois at Urbana-Champaign.

Di Sarno L, Elnashai AS. (2009). Bracing systems for seismic retrofitting of steel frames, *Journal of Constructional Steel Research,* **65**, 452–465.

Deierlein GG. (2004). Overview of a comprehensive framework for earthquake performance assessment, Proceedings of the international workshop performance-based seismic design concepts and implementation, PEER Technical report 2004/05, Richmond (CA): Pacific Earthquake Engineering Research Center.

Elnashai AS, Bommer JJ, Baron I, Salama AI, Lee D. (1995). Selected engineering seismology and structural engineering studies of the Hyogoken Nanbu (Kobe, Japan) Earthquake of 17 January 1995, Engineering Seismology and Earthquake Engineering, Report no. ESEE/95-2, London: Imperial College.

Federal Emergency Federal Agency, FEMA-356. (2000). Pre-standred and Commentary for seismic rehabilitation of buildings, Washington (DC).

Fajfar P., Fischinger M. (1988). N2–A Method for Non-linear Seismic Analysis of Regular Structures, *Proc. 9th World Conf. on Earthquake Engineering*, Tokyo/Kyoto, **Vol**. **5**, 111-116.

Fajfar P., Gaspersic P. (1996). The N2 method for the seismic damage analysis for RC

`

buildings, *Earthquake Engineering Structural Dynamics*, **25** (1), 23–67.

Federal Emergency Management Agency. (2000). State of art report on past performance of steel moment frame buildings in earthquakes, Report no. FEMA 355E, Washington (DC, USA).

Federal Emergency Management Agency. (2000). State of art report on past performance of steel moment frame buildings in earthquakes, Report no. FEMA 355C, Washington (DC, USA).

Federal Emergency Management Agency. (2000). State of the Art Report on Performance Prediction and Evaluation, FEMA-355F, Washington, D.C.

FEMA 273. (1997). NEHRP guidelines for the seismic rehabilitation of buildings.

Federal Emergency Management Agency. (FEMA 2000). Recommended seismic design provisions for new moment frame buildings, Rep. no. FEMA 350, Washington, D.C.

FEMA-273/274. (1997). NEHRP Guidelines for the seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington D.C.

Gaspersic P., Fajfar P., Fischinger M. (1992). An approximate method for seismic damage analysis of buildings, *Proc. I0th world conference in earthquake engineering, Balkema, Rotterdam,* **Vol. 7**, pp. 3921-3926.

Goel R.K., Chopra A.K. (1997). Period formulas for moment-resisting frame buildings, *Journal of Structural Engineering*, **123** (11), 1454-1461.

Gupta B., Kunnath S.K. (1999). Pushover analysis of isolated flexural reinforced

`

concrete walls, *Structural Engineering in the 21st Century, Proc. Structures Congress, New Orleans.*

Goel S. C. (1992). Cyclic Post Buckling Behavior of Steel Bracing Members, Stability and Ductility of Steel Structures under Cyclic Loading, *CRC Press. Boca Raton*, 75-104.

Ghiasvand H. (2012). Investigation of Deformability of Steel Structures with Concentric Braced Frames, *Journal of Basic and Applied Scientific Research*, **2**(4)3302-3310, 2012.

Hisatoku T. (1995). Reanalysis and repair of a high-rise steel building damaged by the 1995 Hyogoken-Nanbu earthquake, *Proceedings 64th Annual Convention, Structural Engineers,* Assn. of California, Sacramento, 21-40.

Hasegawa T., Kamura H. (2008). Seismic response prediction of steel frames utilizing plastic strain energy obtained from pushover analysis, *The 14th World Conference on Earthquake Engineering*, Beijing, China.

Hassan O., Goel S. (1991). Modelling of Bracing Members and Seismic Behavior of Concentrically Braced Steel Structures, Report No. UMCE91-1, University of Michigan, Ann Arbor, MI.

İnel M., Tjhin T., Aschheim A.M. (2003). The Significance of Lateral Load Pattern in Pushover Analysis, *İstanbul Fifth National Conference on Earthquake Engineering*. Paper No: AE-009, İstanbul. Turkey.

Jan T.S., Liu M.W., Kao Y.C. (2004). An Upper-Bound Pushover Analysis Procedure for Estimating the Seismic Demands of High-Rise Buildings, *Engineering*

`

Structures, **Vol. 26**, 117-128.

Jain A. K., Goel, S.C. (1978 a). Hysteresis Behavior of Bracing Members and Seismic Response of Braced Frames with Different Proportions, Report UMCE78-3, University of Michigan, Ann Arbor, MI.

Jain A. K., Goel S. C. (1978 b). Hysteresis models for steel members subjected to cyclic buckling or cyclic end moments and buckling, (user's guide for DRAIN-2D. *EL9 and EL10 UMEE 78R6*, Department of Civil Engineering, University of Michigan, Ann Arbor, MI. Dec., 98 pages.

Jain A. K., Goel S.C., Hanson R.D. (1978). Hysteresis behavior of bracing members and seismic response of braced frames with different proportions, Report UMEE 78R3, Department of Civil Engineering, University of Michigan, Ann Arbor, MI. 365 pages.

Jianguo N., Kai Q., Yan X. (2006). Push-Over Analysis of the Seismic Behavior of a Concrete-Filled Rectangular Tubular Frame Structure, Tsinghua science and technology, ISSN 1007-0214 20/21 pp124-130, **Volume 11**, Number 1.

Kabeyasawa T., Shiohara H., Otani S., Aoyama H. (1983). Analysis of the Full-Scale Seven Story Reinforced Concrete Test Structure, *Journal of the Faculty of Engineering,* The University of Tokyo, **Vol. XXXVII**, 431-478.

Krawinkler H., Seneviratna G.D. (1998). Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation, *Engineering Structures,* **Vol. 20**, 452-464.

Kunnath S.K., Valles-Mattox R.E., Reinhorn A.M. (1996). Evaluation of Seismic Damageability of a Typical R/C Building in Midwest United States, *11th World*

`

Conference on Earthquake Engineering, Acapulco, Mexico.

Kim H., Goel S. (1992). Seismic evaluation and upgrading of braced frame structures for potential local failures, *UMCEE* 92-24.

Krawinkler H. (1996). Northridge earthquake of January 17, 1994, reconnaissance report of steel buildings and Earthquake Spectra, **Vol. 2.**11, Suppl. C, 25-47.

Khatib I., Mahin S. (1987). Dynamic inelastic behavior of chevron braced steel frames, *Fifth Canadian Conference on Earthquake Engineering*, Balkema, Rotterdam, 211-220.

Khatib I. F., Mahin S. A., Pister K. S. (1988). Seismic Behavior of Concentrically Braced Steel Frames, Report UCB/EERC 88/01, University of California, Berkeley, CA.

Kadid A., Boumrkik A. (2008). Pushover analysis of reinforced concrete frame structures, Department of Civil Engineering, *Asian journal of civil engineering (Building And Housing),* **VOL. 9**, NO. 1 PAGES 75-83.

Kamura H., Katayama T., Shimokawa H., Okamoto H. (2000). Energy dissipation characteristics of hysteretic dampers with low yield strength steel, *Proceedings U.S.-Japan Joint Meeting for Advanced Steel Structures, Building Research Institute*, Tokyo.

Lawson R.S., Vance V., Krawinkler H. (1994). Nonlinear Static Pushover Analysis – Why, When and How?, *Proc. 5th US Conf. on Earthquake Engineering*, *Chicago IL,* **Vol. 1**. 283-292.

Liu Z., Goel S. (1987). Investigation of concrete-filled steel tubes under cyclic

`

bending and buckling, Research Report UMCE 87-3, Dept. of Civil Engineering, Univ. of Michigan, Ann Arbor.Apr. 226 pages.

Lee S. (1987). Seismic Behavior of Hollow and Concrete-Filled Square Tubular Bracing Members, Dept. of Civ. Eng, University of Michigan, Ann Arbor, MI.

Lew H.S., Kunnath S.K. (2001). Evaluation of Nonlinear Static Procedures for Seismic Design of Buildings, *Presented at the 33rd Joint Meeting of the UJNR Panel on Wind and Seismic Effects, 1-17.*

López A. M. (2004). A review of existing pushover methods for 2-d reinforced concrete buildings, PhD thesis, European School for Advances Studies in Reduction of Seismic Risk (ROSE School), University of Pavia, Italy.

Moghadam A S., Tso W. K. (2000). Pushover analysis for asymmetric and set-back multi-story buildings, *12WCEE*.

Moghadam A.S, Tso W.K. (1998). Pushover procedure for seismic analysis of buildings, *Progress in Structural Engineering and Materials, Construction Research Communications Limited*, **1**, 3, pp337-344.

Miranda E. (2000). Inelastic Displacement Ratios for Structures on Firm Sites, *Journal of Structural Engineering*, **Vol. 126**, 1150-1159.

Mwafy A.M., Elnashai A.S. (2001). Static Pushover versus Dynamic Analysis of R/C Buildings, *Engineering Structures*, **Vol. 23**, 407-424.

Moghaddam H., Hajirasouliha I. (2006). An investigation on the accuracy of pushover analysis for estimating the seismic deformation of braced steel frames, *Journal of Constructional Steel Research,* **62,** 343–351.

`

Maheri M. R., Sahebi A. (1997). Use of steel bracing in reinforced concrete frames, *Eng. Structures*, **19** (12), 1018-1024.

McCormick J., ASCE S.M., DesRoches R., ASCE M., Fugazza D., Ferdinando A. (2007). Seismic Assessment of Concentrically Braced Steel Frames with Shape Memory Alloy Braces, 10.1061/*ASCE*, 0733, 9445, **133**:6, 862.

Newmark N.M., Hall W.J. (1982). Earthquake Spectra and Design, *Earthquake Engineering Research Institute,* Berkeley, CA. ISBN 0943198224.

Nakashima M., Inoue K., Tada M. (1998). Classification of damage to steel buildings observed in the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures,* **20** (4–6): 271–81.

Naeim F, Lew M., Huang CH., Lam HK., Carpenter LD. (2000). The performance of tall buildings during the 21 September 1999 Chi-Chi earthquake Taiwan, *The Structural Design of Tall Buildings*, **9** (2): 137–60.

Naeim F., Lobo R.M. (1998). Common Pitfalls in Pushover Analysis, *SEAOC Convention, T1-T13.*

Nakashima M., Roeder CW., Maruoka Y. (2000). Steel moment frames for earthquakes in United States and Japan, *Journal of Structural Engineering,* **126** (8): 861–8.

Okamoto S., Kitagawa Y., Nakata S., Yoshimura M., Kaminosono T. (1984). U.S. -Japan Cooperative Research on R/C Full-Scale Building Test, Part *2:* Damage Aspects and Response Properties before Repair Works, *Proceedings 8th WCEE, San*

`

Francisco, Prentice Hall, **Vol. VI**., 593-706.

Osteraas J., Krawinkler H. (1989). The Mexico earthquake of September 19, 1985 -behavior of steel buildings, *Earthquake Spectra*, **5**, 1, 51-88.

Ohi K., Shimawaki Y., Lee S., Otsuka H. (2001). Pseudo dynamic tests on pseudo-elastic bracing system made from shape memory alloy, *Bulletin of Earthquake Resistant Structure Research Center*, No. 34, pages 21-28.

Oğuz S. (2005). Evaluation of pushover analysis. procedures for frame structures, Middle East Technical University, MSC.thesis.

Papanikolaou V. (2000). Development and Verification of Adaptive Pushover Analysis, Procedures MSc dissertation. Engineering Seismology and Earthquake Engineering Section. Impeial College, London, UK.

Providakis C.P. (2008). Pushover analysis of base-isolated steel–concrete composite structures under near-fault excitations, *Soil Dynamics and Earthquake Engineering*, **28,** 293–304.

Pampanin S., Christopoulos C., Priestley M.J.N. (2003). Performance- based seismic response of framed structures including residual deformations, Part II: Multiple-degree-of-freedom systems, *Journal of Earthquake Engineering*, **7**(1), 119–147.

Pankaj P., Lin E. (2005). Material modeling in the seismic response analysis for the design of RC framed structures, *Engineering Structures*, **Vol. 27**, pp 1014-1023.

Pecker A. (2007). Advanced earthquake engineering analysis, Italy, spin 12048397, **Vol. 122**, Courses and lectures-No. 494.

`

Pinho R., Antoniou S., Pietra D. (2006). A Displacement-Based Adaptive Pushover For Seismic Assessment Of Steel And Reinforced Concrete Buildings, *8th US National Conference in Earthquake Engineering*, San Francisco, US, 17th – 21st April 2006 Paper No. 1701.

Rai D., Goel S., Firmansjah J. (1996). SNAP-2DX (Structural Nonlinear Analysis Program), Research Report UMCEE96-21, Department of Civil Engineering, University of Michigan, Ann Arbor, MI.

Rahnama M., Krawinkler H. (1994). Amplification of seismic demands in linear and nonlinear soft soils, *Proc. 5th U.S. Conf. in Earthquake Engrg.*, EERI, Chicago, **2**, 321–330.

Ruiz-Garcia J., Miranda E. (2006). Evaluation of residual drift demands in regular multi-story frame buildings for performance-based assessment, *Earthquake Engineering & Structural Dynamics*, **35** (13), 1609–1629.

Santa-Ana P.R., Miranda E. (2000). Strength reduction factors for multi-degree-of-freedom systems, *Twelfth World Conferenceon Earthquake Engineering* (*12WCEE*) in Auckland, New Zealand.

Saiidi M., Sözen M.A. (1981). Simple Nonlinear Seismic Analysis of R/C Structures, *Journal of the Structural Division*, **Vol. 107**, ST5, ASCE, 937-952.

Sabelli R. (2001). Research on improving the design and analysis of earthquake-resistant steel-braced frames, The 2000 NEHRP Professional Fellowship Report.

`

Sasaki K.K., Freeman S.A., Paret T.F. (1998). Multimodal Pushover Procedure (MMP) A Method to Identify the Effects of Higher Modes in a Pushover Analysis, *Proceedings of the Sixth US National Conference on Earthquake Engineering* (Oakland, California, 1998), Earthquake Engineering Research Institute, 12 pages.

SEAOC (1999). Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California, Sacramento, California.

Tremblay R. (2002). Inelastic seismic response of steel bracing members, *Journal of Constructional Steel Research*, **58** (5–8), 665–701.

Themelis S. (2008). Pushover Analysis For Seismic Assessment And Design Of Structures, Heriot-Watt University, School Of The Built Environment.

Tena-Colunga A., Vergara A. (1997). Comparative study on the seismic retrofit of a mid-rise steel building: Steel bracing vs. energy dissipation, *Earthquake Engineering and Structural Dynamics*, **26** (6), 637–55.

Tremblay R. (1995). Performance of steel structures during the 1994 Northridge earthquake, *Canadian Journal of Civil Engineering*, **22**, 2, 338-360.

Tremblay R. (1996). Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake, *Canadian Journal of Civil Engineering*, **23**, 3, 727-756.

Tang X., Goel S. (1989 a). Brace fractures and analysis of Phase I structure, *Journal of Structural Engineering*, **115**, 8, 1960-1976, Special Issue: U.S./Japan Studies of Steel Structures.

Tang X., Goel S. (1989 b). A fracture criterion for tubular bracing members and its application to inelastic dynamic analysis of braced steel structures, *Proceedings Ninth World Conference on Earthquake Engineering*, *9WCEE* Organizing Committee, Japan Assn. for Earthquake Disaster Prevention, Tokyo, **Vol. IV**, 285-290, Paper 6-3-14.

Teran-Gilmore A., Ruiz-Garcıa J. (2011). Comparative seismic performance of steel frames retrofitted with buckling-restrained braces through the application of Force-Based and Displacement-Based approaches, *Soil dynamics and earthquake engineering,* **31,** 478-490.

Wolfe J, Mar D., Tipping S. (2001). Propped shear walls. Combining steel barces and concrete shear walls for seismic strengthening of existing buildings, Modern steel construction, American Institute of Steel Construction, Chicago, USA.

Watanabe E, Sugiura K, Nagata K, Kitane Y. (1998). Performances and damages to steel structures during 1995 Hyogoken-Nanbu earthquake, *Engineering Structures,* **20** (4–6), 282–90.

Watanabe A., (1996). Some damage control criteria for a steel building with added hysteresis damper, *Eleventh World Conference on Earthquake Engineering Proceedings, Pergamon, Elsevier Science* Ltd., Disc 1, Paper No. 449.

Wight J.K., Burak B., Canbolat B.A., Liang X. (1997). Modelling and Software Issues for Pushover Analysis of RC Structures, Seismic design methodologies for the next generation of codes, Proc. of the International Workshop, P. Fajfar and H. Krawinkler, eds., Bled, Slovenia.

Youssef N, Bonowitz D., Gross J. (1995). A survey of steel moment-resisting frame buildings affected by the 1994 Northridge earthquake, Research report no. NISTIR

`

5625, Gaithersburg (MD, USA), National Institute of Science and Technology (NIST).

Youssef M. A., Ghaffarzadeh H., Nehdi M. (2007). Seismic performance of RC frames with concentric internal steel bracing, *Eng. Struct*, **29**, 1561- 1568.