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Par

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Thème

ETUDE ET MODELISATION DU COMPORTEMENT SISMIQUE LINEAIRE ET NON LINEAIRE DES STRUCTURES METALLIQUES MULTI-ETAGEES CONTREVENTEES PAR DIFFERENTES CONFIGURATIONS

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Dedicate to all members of my family with

all the due respect and love.

Abstract

Current provisions of the Algerian seismic code RPA99 version 2003, are inspired from American seismic design philosophy (UBC and AISC), and are based upon the knowledge of the eighties and nineties of last century in several aspects of technological and analysis of steel frames as they do not take into account the recent advances made in the field of earthquake engineering and do not follow the improved provisions in modern codes. Some of the deficiencies in existing provisions can be summarised: no recommended inelastic analyses (Pushover and Time-history), they not address any provisions on EBFs bracing systems, world-wide used as lateral resisting systems even in developing countries, i.e. Turkey and Iran, the absence of provisions concerning the semi-rigid connections and ambiguity concerning the recognition of soft weak-storey frames mechanism. The philosophy of the weak beam vs. strong column principle, is not explicitly stated in RPA99.

The themes addressed in this PhD research work are part of possible improvements of existing recommendations relating to steel structures in RPA99 in future version. In fact, considering the above remarks and other considerations, the works embodied in this thesis have been aimed to contribute, through the results of compressive parametric studies, to future revision of the code RPA99 regarding the nonlinear analyses, EBFs and the soft-storey conditions.

Actually, the study undertaken in this thesis stems from two different studies. Firstly, this thesis presents the results of comprehensive parametric study through geometrical 2D multi-storeys frames of three kinds of structural systems with different topologies: MRFs, MRFs-CBF and MRFs-EBFs. These models implanted in SAP2000 NL software in order to assess the global elastic and inelastic behaviours, taking into account material and geometric non linearities with relevant parameters thought to influence the seismic response, such as: structure slenderness, aspect ratio, type of bracing systems, number of storeys etc. Another analogous parametric analyses, highlighting the effect soft weak-storey have been performed on frames having irregular vertical stiffness in elastic and inelastic ranges taking into account geometric and material nonlinearities. The second part of the undertaken research work is this thesis concerns the elastic and inelastic behaviour seismic links which characterise EBFs structures. Primarily, a comparative parametric study on shear links was carried out with some available data in literature. Then, a study was conducted to assess the elastic and inelastic behaviours of a large range of long and short links subjected to the AISC (2005) loading protocol throughout 3D FEA models implanted in ABAQUS. Models take into account both geometric and material nonlinearities with large deflection and large strain capability. Also, the influence of web stiffeners on links was numerically investigated with different configuration of links. Encouraging results have been found as compared to recent research works (Čaušević 2008) for shear links and (Imani 2015; Suswanto 2017) for links subjected to monotonic cyclic loadings concerning key parameters.

Finally, the author seriously believes that the time has come for RPA99, in its future version should to include nonlinear pushover analysis at least in its basic form with the fundamental mode of vibration, to adopt EBF structures as a lateral bracing system, with the provisions taken from international seismic codes i.e. EC8, AISC, because their provisions are very close to each other as they are all based on the works of Professor Popov and his colleagues, and to use any type of bracing system to reinforce structures with irregular vertical rigidity in the ground floor to avoid the formation of weak soft-storey mechanism.

Keywords: RPA99, EC8, AISC, multi-storeys, Dual steel frames, MRFs, CBFs, EBF's, elastic, inelastic, pushover, cyclic loading, buckling, soft weak-storey, SAP2000 NL, ABAQUS.

ملخص

الاحكام الحالية للكود الجزائري لرصد الزلازل RPA99 إصدار 2003 مستوحاة من فلسفة التصميم الزلزالي الأمريكية UBC) و (AISC، وتستند إلى معرفة الثمانينيات والتسعينيات من القرن الماضي في العديد من الجوانب التكنولوجية والتحليلية للإطارات الفولاذية كما هي لا تأخذ في الاعتبار التطورات الأخيرة في مجال هندسة الزلازل ولا تتبع الأحكام المحسنة في الأكواد الحديثة. يمكن تلخيص بعض أوجه القصور في الأحكام الحالية: لا توجد تحليلات غير مرنة موصى بها (التمرير والتاريخ الزمني) ، فهي لا تتناول أي أحكام بشأن أنظمة تدعيم EBFs ، المستخدمة في جميع أنحاء العالم كنظم مقاومة جانبية حتى في البلدان النامية ، مثل تركيا وإيران ، عدم وجود أحكام تتعلق بالوصلات شبه الصلبة والغموض المتعلق بالتعرف على آلية إطارات الطوابق الضعيفة اللينة. لم يتم ذكر فلسفة الحزمة الضعيفة مقابل مبدأ العمود القوي صراحة في .RPA99 الموضوعات التي تم تناولها في عمل بحث الدكتوراه هذا هي جزء من التحسينات الممكنة للتوصيات الحالية المتعلقة بالهياكل الفولاذية في RPA99 في الإصدار المستقبلي. في الواقع، بالنظر إلى الملاحظات المذكورة أعلاه والاعتبارات الأخرى، تهدف الأعمال الواردة في هذه الأطروحة إلى المساهمة، من خلال نتائج الدراسات البارامترية الكاملة، في المراجعة المستقبلية للكود RPA99 فيما يتعلق بالتحليلات غير الخطية و EBFsوظروف الطوابق اللينة .في الواقع، الدراسة التي أجريت في هذه الأطروحة تنبع من دراستين مختلفتين. أولاً، تقدم هذه الرسالة نتائج دراسة بارامترية شاملة من خلال إطارات هندسية ثنائية الأبعاد متعددة الطوابق من ثلاثة أنواع من الأنظمة الهيكلية مع طوبولوجيا مختلفة MRFs :و-MRFs CBFو .MRFs-EBFsتم زرع هذه النماذج في برنامج SAP2000 NL من أجل تقييم السلوكيات المرنة وغير المرنة العالمية، مع الأخذ في الاعتبار المواد غير الخطية الهندسية مع المعلمات ذات الصلة التي يعتقد أنها تؤثر على الاستجابة الزلزالية، مثل: رقة الهيكل، ونسبة العرض إلى الارتفاع، ونوع أنظمة التقوية، عدد الطوابق وما إلى ذلك. تم إجراء تحليلات بارامترية مماثلة أخرى، لإبراز التأثير الناعم للطوابق الضعيفة على إطارات ذات صلابة رأسية غير منتظمة في النطاقات المرنة وغير المرنة مع مراعاة اللا خطية الهندسية والمادية. الجزء الثاني من العمل البحثي الذي تم إجراؤه هو هذه الأطروحة المتعلقة بالروابط الزلزالية المرنة وغير المرنة للسلوك الزلزالي التي تميز هياكل .EBFs في المقام الأول، أجريت دراسة بارامترية مقارنة على روابط القص مع بعض البيانات المتاحة في الأدبيات. بعد ذلك، أجريت دراسة لتقييم السلوكيات المرنة وغير المرنة لمجموعة كبيرة من الروابط الطويلة والقصيرة الخاضعة لبروتوكول تحميل (2005) AISC عبر نماذج D FEA 3المزروعة في .ABAQUS تأخذ النماذج في الاعتبار كلا من اللا خطية الهندسية والمادية مع انحراف كبير وقدرة إجهاد كبيرة. أيضًا، تم التحقيق عدديًا في تأثير أدوات تقوية الويب على الروابط باستخدام تكوين مختلف للروابط. تم العثور على نتائج مشجعة مقارنة بالأعمال البحثية الحديثة (Čaušević 2008) لروابط القص و Imani 2015)؛ (Suswanto 2017 للروابط المعرضة لعمليات تحميل دورية رتيبة تتعلق بالمعلمات الرئيسية .أخيرًا، يعتقد المؤلف بجدية أن الوقت قد حان لـRPA99 ، في نسخته المستقبلية يجب أن تتضمن تحليل الدفع غير الخطي على الأقل في شكله الأساسي مع الوضع الأساسي للاهتزاز، لاعتماد هياكل EBF كنظام تقوية جانبي، مع الأحكام مأخوذة من أكواد الزلازل الدولية مثل EC8 وAISC، لأن أحكامها قريبة جدًا من بعضها البعض لأنها تستند جميعها إلى أعمال البروفيسور Popov وزملائه، ولاستخدام أي نوع من أنظمة

الكلمات الرئيسية :

، مرن ، غير مرن ، تحميل EBF، CBFs، MRFs، متعدد الطوابق ، إطارات فولاذية مزدوجة ، AISC ، EC8 ، RPA99

ABAQUS، SAP2000 NLدوري ، انحناء ، طوابق ضعيفة ،

Résumé

Les dispositions actuelles du code sismique algérien RPA99 version 2003, inspiré par la philosophie de la conception parasismique américaine (UBC, AISC) et sont basées sur les connaissances des années 80 et 90 du siècle dernier dans plusieurs aspects technologique et analytique des structures métalliques en acier puisqu'elles ne prennent pas en compte les récents progrès réalisés dans le domaine du génie parasismique. Et ne suivent pas les dispositions améliorées des codes modernes. Certaines des lacunes dans les dispositions existantes peuvent être résumées: absence d'analyses inélastiques (Pushover et Time-history), elles n'avancent aucune disposition sur les systèmes de contreventement EBF, pourtant utilisés dans le monde entier comme systèmes de résistance latérale même dans les pays en développement, à savoir la Turquie et l'Iran, l'absence de dispositions concernant les assemblages semi-rigides et l'ambiguïté concernant la reconnaissance du mécanisme des cadres flexible étage. La philosophie du principe poutre faible contre colonne forte n'est pas explicitement énoncée dans RPA99.

Les thèmes abordés dans ce travail de recherche de doctorat ont pour but d'apporter des suggestions d'améliorations possibles des recommandations existantes relatives aux structures en acier dans RPA99 dans sa future version. En effet, compte tenu des remarques ci-dessus et d'autres considérations, les travaux évoqués dans cette thèse ont eu pour objectif de contribuer, à travers les résultats d'études paramétriques complètes, à la révision future du code RPA99 concernant les analyses non linéaires, les EBF et les conditions d'étage souple.

En fait, l'étude entreprise dans cette thèse découle de deux études différentes. Tout d'abord, cette thèse présente les résultats d'une étude paramétrique complète à travers des structures en 2D multi-étages de trois types de systèmes structurels avec différentes topologies : portiques, duel portiques-concentriques contreventement. Ces modèles implantés dans le logiciel SAP2000 NL afin d'évaluer les comportements globaux élastiques et inélastiques, en tenant compte des non-linéarités matérielles et géométriques avec des paramètres pertinents censés influencer la réponse sismique, tels que : élancement de la structure, rapport d'aspect, type de systèmes de contreventement, nombre d'étages etc. Une autre étude paramétrique analogue mettant en évidence les effets de la rigidité verticale irrégulière en comportements élastique et inélastiques, tenant compte des non-linéarités géométriques et matérielles. La deuxième partie des travaux de recherche entrepris dans cette thèse concerne les tronçons sismiques dans le domaines élastique et inélastique qui caractérisent les structures EBF. Premièrement, une étude paramétrique comparative en 3D (ABAQUS) sur les tronçons courts de cisaillement a été réalisée en considérant certaines données disponibles dans la littérature et d'autres paramètres. Ensuite, une étude a été menée pour évaluer les comportements élastiques et inélastiques d'une large gamme de tronçons longs et courts soumis au protocole de chargement cyclique de l'AISC (2005) à travers des modèles 3D en éléments finis implantés dans ABAQUS. Les modèles prennent en compte les non-linéarités géométriques et matérielles avec une grande déflexion et une grande capacité de déformation. En outre, l'influence des raidisseurs d'âme sur les liens a été étudiée numériquement avec différentes configurations de liens. Des résultats encourageants ont été trouvés par rapport aux travaux de recherche récents (Čaušević 2008) pour les liaisons de cisaillement et (Imani 2015 ; Suswanto 2017) pour les tronçons soumis à des chargements cycliques monotones concernant des paramètres clés.

Mots-clés : RPA99, EC8, AISC, multi-étages, structures en acier, MRF, CBF, EBF, élastique, inélastique, pushover, chargement cyclique, voilement, étage flexible, SAP2000 NL, ABAQUS.

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List of abbreviations

BCBX:	X Braced ground storey
BEBLS:	Braced ground-storey with long link
BEBMS:	Braced ground-storey with medium link
BEBSS:	Braced ground-storey with short link
CBF:	Concentrically Braced Frames
CBFIV:	Concentrically structure with IV diagonals
CBFX:	Concentrically structure with X diagonals
CQC:	Complete Quadratic Combination method
DCM:	Ductility Class Medium
MRF-CBF:	Dual-Concentrically Braced Frames
MRF- EBF:	Eccentrically Braced Frames
FEA:	Finite Element Analysis
FEM:	Finite Element Method
LFM:	Lateral Forces Method
LL:	Long link
MSM:	Multimode spectral method
NDS:	Number Diagonal Stiffeners
NTS:	Number of Transverse Stiffeners
PEEQ:	Equivalent Plastic strain
PEMAG:	Magnitude Plastic strain
SL:	Short link
T:	Transverse stiffeners
U:	Magnitude of displacement
UB:	Unbraced frames (MRFs)

General introduction

GENERAL INTRODUCTION

- **General**

In structural design process, there are lots of parameters affecting the design. Optimization can be described as the process of having the best solution of a given objective(s) while satisfying certain restrictions, or to find the minima or maxima of a given objective function under some constraints. Lateral stability has been one of the important problems of steel structures specifically in the regions with high seismic hazard. The latest major earthquakes, Kobe (1994) and the Northridge (2000) were the two obvious examples where lacks of lateral stability in steel structures were observed. This issue has been one of the important subjects for researchers during the last decades.

Design of seismic resistant structures is based on the assumption that real structures yield when subjected to design level ground acceleration. In the recent years, it has been clearly understood that the elastic analysis does not exactly represent the inelastic (post-elastic) behaviour of the building structures. Due to this fact, the inelastic behaviour of such structures has been a subject of many research studies. For determining the post-elastic behaviour of a building structure, the nonlinear pushover analysis has been adopted in several countries world-wide seismic codes as being capable of capturing the essential of the nonlinear behaviour of structures as well as a more developed analysis the time-history analysis be the best method for obtaining accurate and reliable predictions of the actual behaviour of the structure.

- **Problematic and scopes**

Most of the buildings in Algeria are consisting of reinforced concrete structures elements and very few are constructed using steel elements and are normally designed to resist both static and seismic loadings. However, in recent years, multi-storeys buildings are gaining interest increasingly common in a developing country like Algeria. An overview of RPA provisions for steel structures are clearly well-behind of developed seismic codes throughout the world. These RPA99's provisions are based upon the knowledge of the eighties and nineties of the last century and need obviously an urgent improvement to be at the current and latest advances in seismic engineering.

For steel structures, RPA99 provide insufficient provisions for the time being, which are based upon the knowledge of eighties and nineties of the preceding century, to be applied for elastic and inelastic behaviour of such structures. Indeed, Chapter VIII of RPA99, covering only few pages are devoted to steel structures, excluding the advance made in the field of earthquake engineering. In fact, a big gap is noticeable in RPA99, based up the knowledge of the eighties and nineties of last century, concerning several aspects of technological and analysis of steel frames. One of the most remarkable lack in RPA99 provisions concern EBFs steel structures nor the nonlinear methods of seismic analyses (i.e. Pushover and Time-history), connections are only allowed to be rigid etc.., despite their popularity world-wide even in developing countries like Turkey, Indonesia and Iran for instance,, In addition, RPA99 addresses ambiguously soft weak-storey frames, which is the case of buildings with an open ground storey. The capacity design method is not clearly stated in RPA99 which has demonstrated its effectiveness through the obtained results in this thesis as a way of controlling the plastic mechanism and the collapse of steel structures.

- **Objectives of the research**

The objectives of subjects developed in this thesis have been progressively improved throughout the conduction of the present research work. Initially; the research was concerned with the investigation of the global elastic linear and inelastic behaviours of multi-storey steel structures as defined in RPA99 and CCM97 with similar philosophical concepts as EC3, with and deprived of bracing system to a specific advanced topic dealing with the seismic links in EBFs structures. The nonlinear behaviour of structures is assumed throughout the behaviour coefficient R, which is expected to give some insight in the nonlinear behaviour is obviously not enough and remains imprecise as no real inelastic method is suggested. These provisions need certainly to be improved with a clear statement concerning a true nonlinear analysis.to carry out a complete inelastic analysis. In designing steel structures, and in order to avoid the brittle collapse and the formation of soft-storey mechanism, the SCWB principle is communally used in most international seismic codes while is not clearly identified in RPA99 nor in CCM97 of the structures as a base of designing of all structures analysed in this research work along with the concept of ductility as an important basis of designing steel structures.

Broadly speaking, the work described in this thesis stems from two different studies related to the performance assessment of seismic lateral resistant systems: including MRF, CBF and the newly developed seismic resisting systems EBF (not yet covered in RPA99 Algerian Seismic Code). The first part of the study undertaken compares the results of a global comprehensive parametric analysis dealing with the performance of multi-storeys steel structures in the elastic and inelastic ranges, including material and geometric non linearities. To evaluate RPA99's provisions with those of a more developed seismic code, that is EC8, a comparison between the results from global analysis is usually made. The second part a more advanced study is carried out and investigates a special aspect of EBF links throughout a parametric three dimensional FEA using ABAQUS which highlights many parameters thought to be influencing the linear and nonlinear behaviours of seismic links in order to find out their performance and in order to suggest their use in the future version of RPA 99.

Chapter 4 of the thesis presents the results of comprehensive parametric study analysing the global seismic behaviours of three kind of structural systems: MRFs, MRFs-CBF and MRFs-EBFs in multi-storeys frames. relevant parameters that can influence the seismic response have been assessed, such as: structure slenderness, aspect ratio, type of bracing systems, number of storeys etc, and the global linear and nonlinear behaviours of irregular frames highlighting the effect soft weak-storey. The examined frames have been designed based upon Weak beams vs. Strong columns principle in accordance with requirement preconised by

EN1998-1 (2004). 2D numerical models have been implemented in SAP2000 NL in order to perform both linear and nonlinear analyses.

The early results of comprehensive parametric analyses, especially those predicted by the pushover analysis, have shown that nonlinear behaviour of studied structures can easy included in studied structures as the drift displacements extracted where well beyond the limit imposed, which is 1% of the storey height, for all kinds of structures, including irregular frames braced and unbraced structures. Consequently, in almost all structures, except slender ones and at the top floor, no second-order analysis was necessary. Some results have been summarised in conference papers in national and internationals seminars **(Labed et al 2012, 2013, and 201**4).

Previous research studies show that behaviour of links is fairly complicated and affected by various parameters, and as a result significant amount of research interest has been directed towards both experimental and numerical determination of the nonlinear behaviour and cyclic energy dissipation characteristics of links. Understanding strength, stiffness and ductility properties of EBFs requires studying the nonlinear behaviour of links. Chapter VII gives details on the second part of this work. In the perspective of having a better understanding of the inelastic behaviour of links and parameters that influence that behaviour, a comprehensive finite element study is conducted by the means of the general-purpose finite element analysis software ABAQUS **(ABAQUS 2006)**.

Firstly, the author has analysed the nonlinear behaviour of isolated short seismic links, as very popular in practice, in EBF-D structures under static loadings taken from literature. The main aim of this second part of this particular study is to examine the possibility of increasing the strength and stiffness of shear links from preventing the local stability to occur in the web with the contribution of different kind of stiffeners including diagonal ones. Results have been summarised in an international conference **(Labed et al 2015).** Secondly, as for steel frames which are likely to be subjected to cyclic loads under earthquake and other forms of dynamic loadings will require the study of cyclic and hysteretic behaviours. Full details on the types and their corresponding behaviours of seismic links are explicitly exposed in subsequent chapters 4 and 5 in the thesis. For a better understanding of the inelastic cyclic behaviour and parameters that influence that behaviour, a comprehensive finite element study is conducted on long and short seismic links by the means of the general-purpose finite element analysis software ABAQUS **(ABAQUS 2006),** which has been published in AJCE 2019 **(Labed et al 2019)**, which deals with a more complicated situation, taking into account the effect of cyclic loading, through a cyclic protocol, of long and short links designed in structures analysed in chapter VI.

- **Organisation of the Thesis**

This thesis contains an introduction, seven chapters and concluding remarks.

Introduction: The introduction involves an overview about the objectives and scopes of the study conducted in this thesis. The problematic is mainly related to the necessity to study the seismic behaviour of structures using elastic and inelastic behaviours of steel multi-storeys structures with different topologies including EBFs frames which are not covered in RPA99, and the investigation of elastic and inelastic behaviours of seismic segment (links) of EBFs structures.

Chapter 1: A brief account of reviewing and discussing the current steel structures located in seismic zones is presented. A description of the main structural typologies used on seismic zones is also described. Different types of Lateral Frames Resisting Systems (LFRS) that are commonly used in steel buildings including (MRFs), (CBFs) and (EBFs) are fully discussed in this chapter based upon very new documents, taking into account recent advance in the subject. The chapter also concludes a succinct presentation of the provisions of RPA99, the European EC 8 and the American AISC for steel buildings erected in seismic areas.

Chapter 2: The purpose of this chapter is to describe briefly some of the concepts and procedures underlying modern earthquake engineering, especially as they are applied to buildings. A succinct state-of-the art of the methodology employed on current seismic codes is provided. This chapter delivers as well the recent advances in the design of building, particularly those located in seismic areas for low and medium-rising multi-storeys steel structures. An emphasize on their seismic performance beyond the elastic range under large inelastic deformations is described.

Chapter 3: In this chapter, a brief state-of-the art of the methodology employed in the seismic analyses in current codes is provided. The aim of this chapter is to capture the essence of seismic methods of analysis available in very good recent text books recommended in the subsequent sections, rather than covering in details the theoretical mathematical background of these methods, which can be found in the immense volume of literature that exists on the subject and nominated in references.

Chapter 4: Seismic methods of analysis described in Chapter 3 have been used through study cases of multi-storeys steel frames. The investigation starts with basic cases of multi-storeys steel structures, that is MRFs frames, to the examination of the effect of bracing systems when associated to MRFs, then the behaviour of MRFs-EBF structures in an elastic behaviour, afterwards more complicated cases dealing with the nonlinear analyses by Pushover analysis of regular and irregular (soft-storey) steel structures. Some of the obtained results have been published in some international and national conferences, see references.

Chapter 5: is devoted to present details on EBFs steel structures including their definitions, basic concepts, basic elastic and inelastic mechanisms, their classification and energy dissipation. The concept of links will be used later on chapters 6 and 7 through an advanced analysis of cyclic behaviours with 3D implantation in ABAQUS software. Information included in this chapter are being extracted from the lasted publication available in literature and given in reference section.

Chapter 6: This chapter introduces the necessary background to the next chapter, i.e. chapter 7, and summarises the behaviour of seismic links from basic case to the one under shear and bending cyclic loadings. Design and failure modes and mechanisms of links according to their lengths are being detailed. Also, considering the problematic effects of the stiffeners on the overall behaviour of EBF links is discussed. A succinct presentation of the concept of seismic loading protocol which is used to simulate the number of inelastic cycles, cumulative inelastic demand, and peak displacement demand associated with a design seismic event is discussed together with some of the world-wide used protocols.

Chapter 7: Deals with the numerical modelling of the inelastic behaviour of seismic links in EBF structures. It starts with an overview of the finite element analysis method than some useful information on ABAQUS software including features and capabilities. Then cases studied are presented. The first case covers the nonlinear behaviour of shear links where the data are taken from literature, with and extension studied cases proposed by the author. Results of this work have been summarised in an international conference paper (Labed et al 2015). In the second part of this work, in order to have a better understanding of the inelastic cyclic behaviour and parameters that influence that behaviour, a comprehensive finite element study is conducted by the means of the general-purpose finite element analysis software ABAQUS (ABAQUS 2006). This study deals with a more complicated situation, taking into account the effect of cyclic loading, through a cyclic protocol, of long and short links designed in earlier chapter. Results have been published in AJCE 2019, as original paper in Oct.2019, (Labed et al. 2019)

C**ONCLUSIONS AND PERSPECTIVES**: Based on the obtained results of the studies undertaken in this thesis in both parts, some important conclusions have been drawn to express the necessity of improving the current RPA99 provision to cope with the advances made in seismic engineering from some decency regarding the structural typology (EBFs) and nonlinear seismic analysis. Finally, some suggestions for further work have been mentioned.

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CHAPTER 1:
SEISMIC STEEL RESISTING SYSTEMS

CHAPTER 1: SEISMIC STEEL RESISTING SYSTEMS

1.1 INTRODUCTION

In this chapter, a brief account of the reviewing and discussing concerning the current steel structures located in seismic zones is presented. A description of the main structural typologies used on seismic zones with different types of Lateral Frames Resisting Systems (LFRS) that are commonly used in steel buildings are discussed.

A brief presentation of the various structural systems that ensure the overall lateral stability of the buildings, such as moment resisting frames (MRF), concentric (CBF) or eccentric (EBF) bracing systems, of different types or combination of the above, in association with the diaphragm action of the floor slabs and the rigidity of the connections and joints (simple, rigid or semi-rigid joints) will be succinctly presented. Also, sufficient information to select, for each specific case of a building, the appropriate structural configuration for the overall structure, and for its parts, such as the form and cross-section type of the individual structural elements and the connections and joints. Each of these LFRS may be used solely to resist the lateral force in both orthogonal directions in a building, or a mixed LFRS or a combination of these systems may also be used.

For low and mid-rise buildings, it is typical in design practice to use only one type of LFRS to resist the lateral force in any one direction. Depending on architectural considerations, the LFRS may be located internally within the building or on the exterior face of the building.

The chapter concludes with a succinct presentation of the provisions of RPA99, the European EC8 and the American AISC for buildings constructed in seismic areas. These rules are specific for each type of system ensuring the overall lateral stability of the structure (MRF, CBF, EBF etc.) and are presented in specific sections. The seismic rules are related to the required stiffness and strength, the hierarchy of yielding, known as capacity design, as well as the damage limitation for the non-structural elements of the building in cases of frequent earthquakes, weaker than the design ones.

1.2 STRUCTURAL STEEL AS A BUILDING MATERIAL

1.2.1 Introduction

Steel being a ductile material is equally strong in tension and compression, and is an ideal material for earthquake resistant structures. In other words, steel as structural building material offers competitive advantages when a high strength-to-weight ratio is desired.

The mechanical properties of steel are of importance for design and construction. The usual properties such as the modulus of elasticity, the yield stress or the tensile strength are used in design calculations for any type of structure.

Ductility allows the structure to undergo large plastic deformations without significant loss of strength. The common grades of mild steel have adequate ductility and perform well under cyclic reversal of stresses. High-strength steels provide higher elastic limits but have low ductility and are, therefore, not recommended. Moreover, high-strength steels result in less area of member cross-sections and thereby tend to be more prone to local buckling.

- **Advantages:** Some of the advantages of structural steel as a building material include the following: a high strength-to-weight ratio; uniform and homogeneous properties; and highly predictable associated with high ductility, thus providing adequate warning of any impending collapse.
- **Disadvantages**: However, some disadvantages of steel as a building material can be cited in the following. Its susceptibility to corrosion and consequently, maintenance costs could be high compared to other structural materials. It can be affected by high temperatures and therefore often needs to be protected from fire. Depending on the types of structural details used, structural steel may be susceptible to brittle fracture due to the presence of stress concentrations, and to fatigue due to cyclic or repeated loadings causing reversals of stresses in the members and connections. Also, there are heavy duty steel structures, such as bridges, offshore structures or similar, that are subjected to fatigue loading, exposed to severe environmental conditions or very low temperatures where toughness and through thickness properties are of equal importance for construction and service **(Vayas 2019).**

1.2.2 Steel behaviours

Behaviour of steel buildings under earthquakes has generally been satisfactory from the point of view of strength. The properties of steel that contribute to the elastic resistance of steel structures during moderate earthquakes are the yield strength and elastic stiffness. However, in major earthquakes, a structure may undergo inelastic deformations and rely on its ductility and hysteretic energy dissipation capacity to avoid collapse.

1.2.2.1 Static behaviour

The stress–strain relationship for steel is shown in Figures 1.1 and 1.2. In Figure 1.1, the basic material behaviour is expressed by the stress-strain curve of steel. This curve is determined experimentally by the tension test that may be performed in a universal testing machine. The tension test is performed under deformation control, in order to determine the complete curve including the unloading branch.

Figure 1.2(a) is usually idealized to the bilinear form, shown by the solid lines in Figure 1.2(b), although strain hardening (broken lines) is taken into account in some cases. The yield stress f_y and the ultimate stress f_u are used for steel sections or plates, and f_s is used for reinforcing bars. The value of Young's modulus *is Es* equal to 210 GPa.

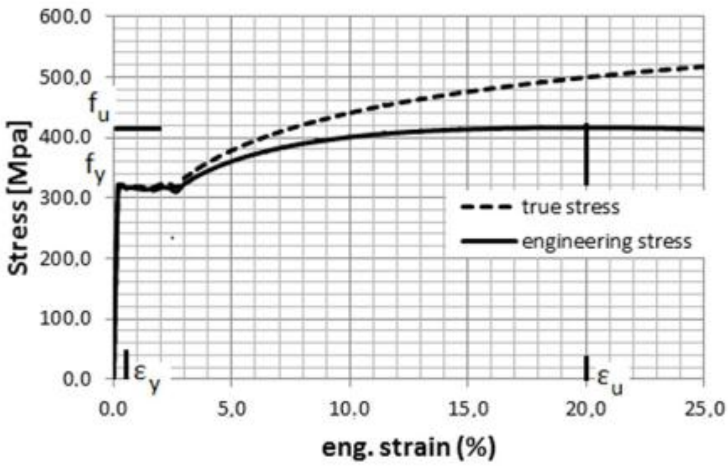


Figure 1. 1. Stress–strain curves of steel from tensile test **(Vayas 2019).**

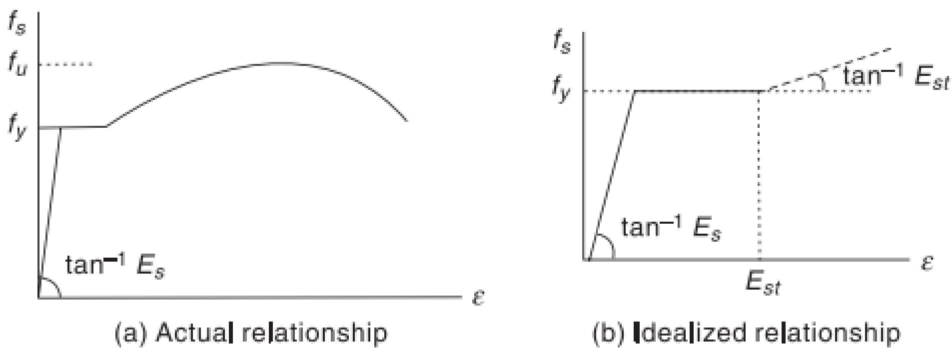


Figure 1. 2. Stress–strain relationship of steel **(Duggal 2013)**.

1.2.2.2 Cyclic behaviour

Hysteretic energy is defined as the energy dissipated by inelastic cyclic deformations is given by the area within the load deformation curve, the hysteretic curve, detailed of such definition will be given in the chapter dealing with the inelastic behaviour of EBF links. Larger area implies more dissipation of hysteretic energy.

It must be mentioned that structures having low hysteretic energy dissipation capacities are likely to collapse due to low cyclic fatigue, even if the deformations are well below the ultimate deformation. In steel structures, good ductility and energy dissipation capacity can be achieved by using thicker sections to avoid local buckling. This implies that plastic and compact sections should be preferred over semi-compact and slender sections. Since earthquakes produce large deformations and low cycle fatigue, both the ductility and energy dissipation capacity are the prime requirements to resist severe earthquakes.

The hysteretic stress–strain relationship for steel, subjected to alternately repeated loading, is shown in Figure 1.3(a). The unloading branch shows an incipient slope equal to the elastic slope and is gradually softened owing to the Bauschinger effect (Appendix XI). Due to the

Bauschinger effect, the plastic deformation of steel increases the tensile yield strength and decreases the compressive yield strength. Some of simple models of hysteretic stress–strain curves are shown in Figures 1.3(b), (c) and (d).

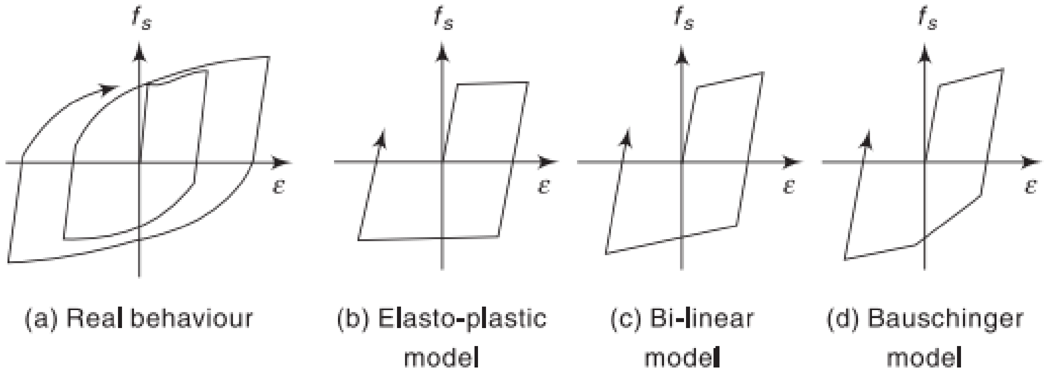


Figure 1. 3. Hysteretic behaviour of steel **(Duggal 2013)**.

1.2.3 Advance in steel technology

It must be also mentioned that the advances on steel production technologies have allowed the improvement of its physical and chemical properties, and hence, new type of steels with attractive characteristics can be produced. In addition to the traditional hot rolling processes, nowadays, other ones such as, controlled rolling, normalizing, and quenching and tempering, various combination of rolling practices and cooling rates have allowed to produce steel with excellent mechanical proprieties **(Miki 2002)**.

1.3 STRUCTURAL STEEL SYSTEMS

The art of structural design is manifested in the selection of the most suitable structural system for a given structure. The arrangement of beams/girders/joints or trusses, and columns to support the vertical (gravity) loads and the selection of a suitable bracing system or a column and beam/truss arrangement to resist the horizontal (lateral) loads poses a great challenge to the structural engineer, since they will determine the economy and functional suitability of the building. The selection of a suitable system is made mainly based on previous data or experience. Steel structures may be classified into the following types:

(a) Single-storey, single, or multi-bay structures may have truss or stanchion frames, or rigid frames of solid or lattice members.

(b) Multi-storey, single, or multi-bay structures of braced or rigid frame construction (which will be discussed in detail in the next section).

(c) Space structures, in the form of single-, double- or multi-layer grids, steel- frame folded plates, braced barrel walls, and domes, are required for very large column-free areas.

1.4 MAIN ELEMENTS OF A STEEL STRUCTURE

1.4.1 General

Steel buildings present many structural advantages, as strength, flexibility and very short erection times, and therefore became the appropriate solution compared with the traditional constructional methods.

Steel buildings provide many advantages, compared to concrete buildings for example, in many fields such as high degree of industrial prefabrication, with the related positive consequences on the quality assurance, shortening of the construction time, lower mass of the structure and their related benefits for foundations, facility in structural strengthening, modifications and additions, lower sensitivity to environmental conditions, possibility to disconnect members and dismantling the structure, etc.

Furthermore, it is common recognised that a very satisfactory seismic behaviour due to the ductility of steel is much more easily achievable if its structural system possesses some characteristics that enable a clear and simple structural response under the action of the seismic event. On the other hand, however, steel buildings are recognised to be more sensitive to corrosion effects of their external members as well as in fire conditions.

Steel buildings today are noticeable for their large variety of shapes, the large spans, the natural lighting and the overall impression they give, as modern and aesthetic constructions. They apply mainly for office buildings, banks, hotels, commercial centres, garages. Applications of steel multi storey buildings in Europe are less, and even less in Algeria compared to USA and Japan. The common types of structural systems (i.e., a combination of several structural members) used in steel building structures include trusses, moment frames, and braced frames.

1.4.2 Types of structural elements in steel buildings

The main elements that compose the structural system of a multi storey steel building are columns, main beams, secondary beams, slabs and vertical bracing systems, if any. At the start of the design the columns' grid is fixed. The main beams join, at the levels of the successive floors, the column heads, usually in all axes, in both main directions of the building. In the following, it must be indicated that the most of the material is reported in **(Vayas 2019)**.

Trusses: May occur as roof framing members over large spans or as transfer trusses used to support gravity loads from discontinuous columns above.
Frames: Are structural steel systems used to resist lateral wind or seismic loads in buildings. The two main types of frames are moment frames and braced frames. Moment Frames which are designed to resist lateral loads through the bending rigidity of the beams/girders and columns. The connections between the beams/girders and the columns are designed to resist moments due to gravity and lateral loads. Braced Frames are being designed to resist lateral

loads through axial compression and/or tension in the diagonal members. These frames are usually more rigid than a typical moment frame and exhibit smaller lateral deflections.

Columns: For the columns of multi storey buildings wide flange I-sections are usually selected, as having a satisfactory strength and stability resistance against flexural buckling, in respect to both main axes of the cross-section. When lateral stability of the building is provided by frame action, or by combined frame and bracing action, crossed double I-sections are often used, where one section is continuous, while the other is divided in two parts, welded to the continuous one. The use hollow sections, hot or cold formed, can be an alternative which, if square, have the same rigidity in respect to both axes.

Steel columns in multi storey buildings could be arranged as pinned or fixed at their bases. In principle the columns, in multi storey buildings, are continuous along the height of the building and the beams span between them. However, it is possible that the beams remain continuous and the columns are interrupted at floor levels, especially when beams transfer to columns mainly vertical loads and the lateral stability of the building is ensured by vertical bracing systems or shear walls.

Beams: Beams of the building join the column heads at floor levels, in both main directions and are usually of I-section, hot rolled or built-up. In cases of larger spans, truss beams could also be employed. The webs of main beams may be provided with systematic or isolated openings, to facilitate the installation of the building facilities equipment. Besides cross-section's strength, beams shall also be verified to lateral-torsional buckling (LTB) stability, at the ultimate limit state (ULS), at both construction and service stages.

Beam to column joints: First of all, it must be mentioned that there is a large variety of connection types. Most commonly; beam to column connections are bolted connections since beams and columns are linear elements transported on-site, where they are connected during assembly. Roles of connections in steel structures can be summarised as follows:

a) To connect different steel members and sheets to complete structures.

b) To form cross-sections and members from the final steel making products, such as cross-sections from steel plates, built-up members from rolled or welded sections and plates etc.

c) To splice members which are delivered in partial length due to transportation restrains and form members of full length.

As it is in EC3 beam to column connections and joints could be classified:

- **Simple connection:** Able to resist only forces, and having sufficient rotation capacity to be modelled as pinned in which the transmission of shear forces is usually realized by bolting the web of the beam to the column, through a plate welded perpendicularly to the column flange or web.

- **Rigid connection:** Able to resist both forces and moments, and having sufficient rigidity to consider that the angle between the connected members remains unchanged during loading. A common arrangement for rigid beam-to-column joints is to weld at the beam an end-plate, and bolt it subsequently to the column flange. The end-plate is usually extended beyond the top flange of the beam to increase the lever arms, by providing additional bolts. The formation of plastic hinges at the interface between the connected parts should be avoided, especially under cyclic loading, as inelastic behaviour at this position relies on inelastic elongation of bolts which is usually limited. The column web should be provided with stiffeners at positions where concentrated forces are transferred, such as the levels of the beam flanges and the haunch. Other interesting details on this kind of connection can be found in EC1.

- **Semi-rigid:** In which a change φ of the angle between connected members appears, when a moment M applies on the connection, which cannot be neglected, and the relation M-φ should be introduced in the analysis. Semi rigid connections behave between simple and rigid ones. They are able to resist bending moments but the change of angle φ, from the initial angle between connected members, when moments apply, may be significant in the measure that influence the bending moments distribution in the frame, to an extent that cannot be neglected in analysis.

In EC3, the connection is characterized by a moment-rotation curve, M-φ, that should be introduced in global analysis, most usually in the form of appropriate springs. Joints are characterized by the shape of their M-φ curve. In practice there are no ideal rigid or ideal pinned joints.

It is reminded that it is important to provide a correct representation of connections and joints in the analysis model, by introducing pins, springs or rigid elements as appropriate.

Dissipative zones: Seismic design is oriented towards the arrangement of specific structural zones or members which, in case of a seismic event, have the possibility to absorb part of the seismic energy through the development of cyclic plastic deformations and therefore to reduce seismic forces to the other structural members. In the above zones or members, called dissipative zones (or dissipative members), damages might appear after a strong seismic event, so that dissipative members should have the possibility to be replaced after the earthquake. Steel, due to its capacity to develop important plastic deformations, before fracture, is an ideal material for such an approach in the design of a structure.

Brace: An axially loaded diagonal member of a frame.

Braced-frame: A structure that relies on braces for lateral resistance.

1.5 BRACING SYSTEMS OF STRUCTURES

1.5.1 General

Bracing is a method used to resist lateral loads in a building's structure. It is widely applied due to its efficiency and low-cost employment. Furthermore, the diagonal bracing members, which are connected to each other, forms a vertical truss that holds the structure's lateral loading. Moreover, the diagonal bracing work in axial stress which needs for member sizes to provide rigidity and support against horizontal shear. In current construction, one or two-storey height bracing system that is applied in earlier structures is still used. Nevertheless, there has been some development that improves the bracing system **(Hejazi 2018)**. An adequate bracing system requires both strength and stiffness. The magnitude of the initial out-of-straightness of the members to be braced has a direct effect on the bracing force. The brace stiffness also affects the brace force **(Ziemian 2010).**

1.5.2 Bracing system categories

Bracing systems in a building are mainly provided to resist horizontal forces acting transverse to the main frames and to transfer them to the foundation. In addition, they offer lateral support to the aforementioned main elements and they play an important role during erection. They are divided into horizontal bracing systems, arranged between successive rafters of frames or top chords of trusses and vertical bracing systems arranged between columns **(Hejazi 2018).** Bracing systems may be of two categories:

Horizontal (or wind) bracing systems also called wind bracing systems are, in general, arranged at the roof of the building. They transversely connect the main plane frames to a complete 3D main structure and provide a diaphragm action in the roof. Its elements are placed at the level of the upper flange of the rafters with I- or H- cross-sections or at the level of top chords in the case of trusses. Horizontal bracing systems connect only a few main frames being usually placed in the end panels (first and last) of the building and at intermediate locations every 4 to 6 panels.

Vertical bracing systems are formed by adding diagonal bars between columns in selected panels, in order to increase significantly their rigidity. These systems resist horizontal actions arising from the horizontal bracings and transfer them to the foundations. In addition, they form the rigid structural part for the anchorage of longitudinal elements such as bars or side rails which offer, when needed, lateral support to the columns and ensure overall stability during erection. The required cross-sections' capacities depend mainly on the magnitude of horizontal forces and whether the building is in seismic areas or not.

1.5.3 Vertical bracing

There are three types of bracing that can be used as lateral resisting system in simple structural system.

1. Concentrically Braced Frame (CBF).
2. Eccentrically Braced Frame (EBF).
3. Buckling Restrained Braced (BRB).

- **Concentrically Braced Frame (CBF)**

Most common CBFs configurations are shown in Figure 1.4. Concentrically braced frames are LFRS that use axially loaded members to transfer lateral forces to the foundation.

The brace members rely on axial strength, in both tension and compression, and stiffness to resist the applied axial loads. In concentric bracings the dissipative zones are mainly located in the tensile diagonals. The most common type of such bracing is the X-bracing with active tensile diagonals, where horizontal forces can be resisted by the tensile diagonals only. Details of connection of such bracing system are shown in Figure 1.5.

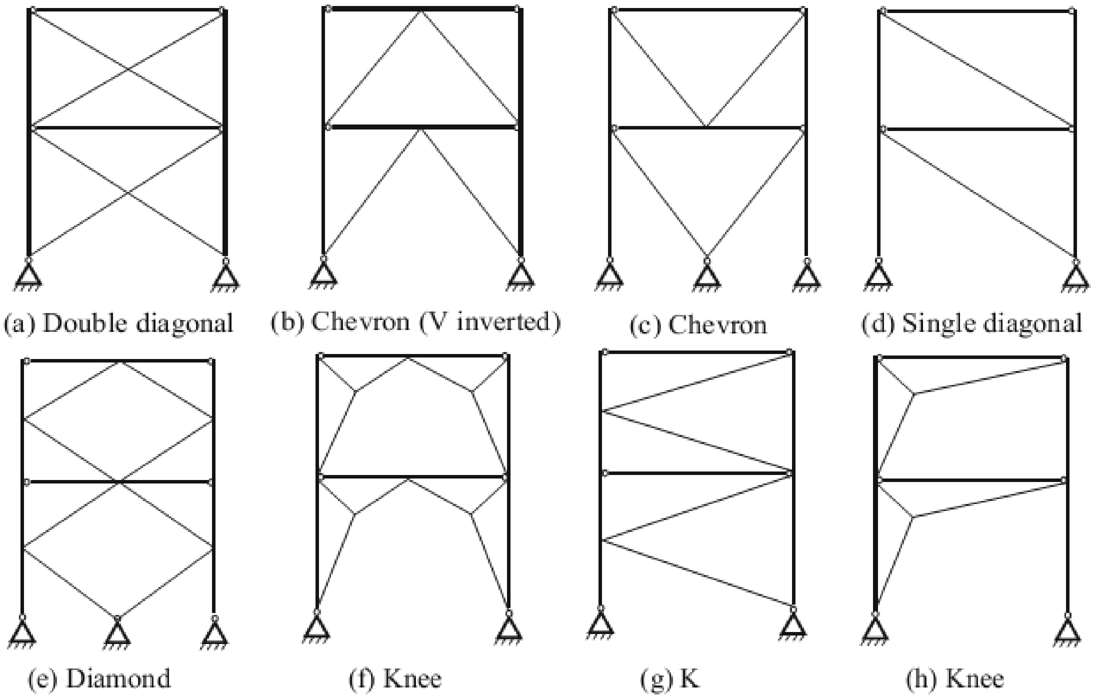


Figure 1. 4. Different types of CBF steel structures **(Hejazi 2018).**

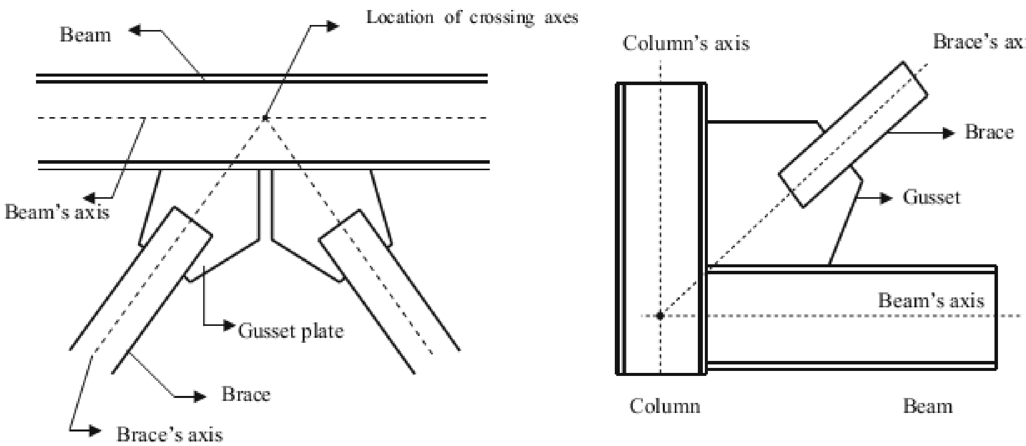


Figure 1. 5. Connection details of a concentrically braced frame **(Hejazi 2018).**

- **Eccentrically Braced Frame (EBF)**

Eccentrically Braced Frame (EBF) is a system that is employed to provide support against lateral events such as wind and earthquakes. Some configurations are represented in Figure 1.6 along with detailed connection in Figure 1.7. EBF consists of a beam, one or two braces and columns and it is fairly similar to traditional braced frames. The only contrast is that at least one end of the braces is eccentrically connected to the frame. Furthermore, optimum EBFs will behave in a ductile manner through the shear forces or flexural yielding of a fuse statement introduced by the connections of the frame and the braces.

The connections also promote bending in the beam adjacent to the brace. Furthermore, in eccentric bracings, the dissipative members are the links (horizontal or vertical) which can develop cyclic plastic deformations either due to bending or shear.

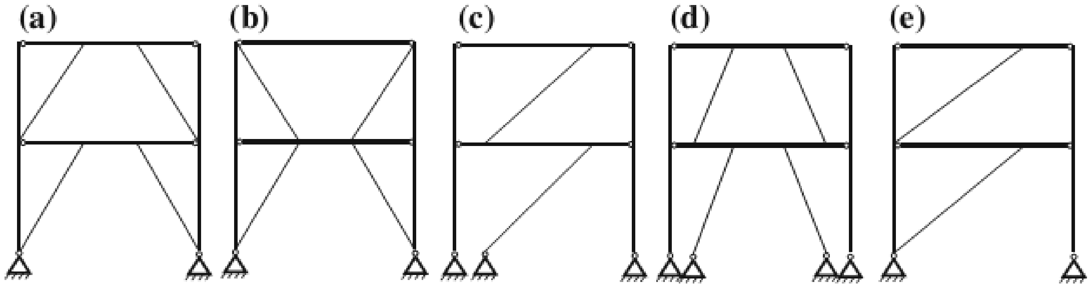


Figure 1. 6. Different types of eccentrically braced frame **(Hejazi 2018).**

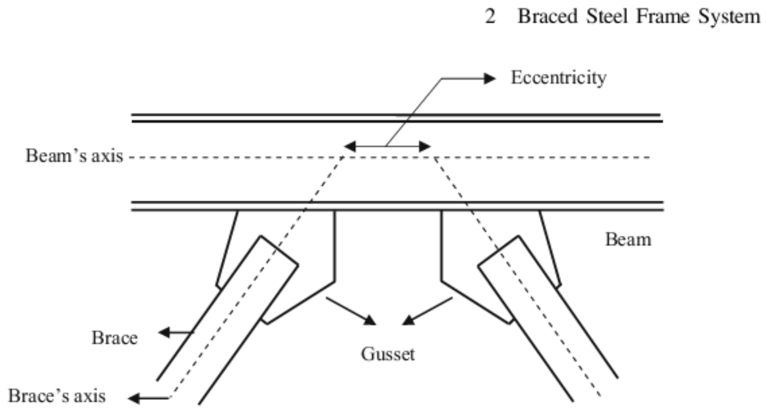


Figure 1. 7. Connection details of EBF **(Hejazi 2018)**.

1.6 OUTLINE TO LATERAL STEEL SEISMIC RESISTING SYSTEMS

In addition to the main function of the system structures to transfer vertical loads to the foundations, they should be also able to transfer horizontal loading to the ground, which results from wind, seismic action or even constructional imperfections, such as deviations of columns from verticality.

In a seismic situation, the structural simplicity implies that a clear and direct path for the transmission of the seismic forces is available. The seismic forces are associated with the different masses of a structure that are set in motion by its dynamic response to the seismic excitation. In buildings, an important part of their mass is located in the floor elements that act simultaneously as originators of the horizontal seismic forces and as the elements that apply these forces to the vertical elements. These, in turn, have to transmit the forces to the ground at the foundation level. It is furthermore very desirable that such a pattern of structural elements ensures similar resistance and stiffness characteristics of the building as a whole in these two main orthogonal directions. Bearing in mind that, even for well-designed structures, a large-intensity earthquake will always be an extreme event which has the potential to drive the structure to its limits and to reveal all hidden weaknesses and defects, simple structures are at an advantage because their modelling, analysis, dimensioning, detailing and construction are subject to much less uncertainty and thus their seismic behaviour is much more consistent.

In steel buildings there are two possibilities to provide lateral stability and resist horizontal forces:

- Using moment resisting frames (MRFs) that require rigid connections between beams and columns. Specifically, the capacity of loading in moment resistance frame shall decrease by increasing the height and distance of the columns. The reduction in moment resistance allows the moment resistance frame base to be ductile. The ductility of plastic deformation is divided into three categories in the USA codes:
  1. Ordinary moment resisting frame;
  2. Intermediate moment resisting frame;
  3. Special moment resisting frame **(Hejazi 2018).**
- Using braced frames BF: frames in which the resistance to both lateral load and frame instability is provided by the combined action of floor diaphragms and a structural core, shear walls, and/or a diagonal, K-brace, or other auxiliary system of bracing (**Ziemian 2010**).
- Using vertical bracing systems possibly combined with shear walls. It is also possible to provide frame action or vertical bracings in both main directions of the building. Vertical bracings should be preferably arranged symmetrical, along the perimeter of the building, in order to avoid eccentricities and increase the torsional rigidity.

1.7 PERFORMANCE OF SEISMIC STEEL RESISTING STRUCTURES

In addition to what has been already said in previous sections, in the following, details concerning the behaviours of seismic resisting systems will be provided.

1.7.1 Overview of the evolution of the lateral resisting steel systems

MRFs have a high level of ductility, making them an excellent option to dissipate energy for high seismic events. However, the high level of ductility comes at a cost: a low level of lateral stiffness. MRFs have a lower level of lateral stiffness than CBFs since they lack braces, and the low lateral stiffness of MRFs can cause storey drift at levels exceeding drift limitations. As such, MRFs are designed around drift instead of strength, resulting in reduced economy. Conversely, CBFs have a high level of lateral stiffness and a low level of ductility. For CBFs to be utilized in high seismic regions, special detailing is required to ensure that the frames behave in the prescribed manner. In the 1970s, a new set of frame configurations was proposed for seismic design that would combine the advantages of MRFs and CBFs.

The seismic-resisting EBF is the product of decades depicts a modified chevron configuration in which there is one mid-beam link per level; the braces of the above level could be inverted to form a modified two-storey X configuration, which would reduce the axial load transferred to the beams. The frame configuration in Figure 1.7 (b) depicts a column-link configuration in which the link is adjacent to one of the frame columns. Figure 1.6 (d) depicts a second modified chevron configuration in which two links are created due to brace-column eccentricity; in this case, one link is considered active and one passive. In figure 1.7 an aspect of connection details of EBF **(Hejazi 2018)**. The passive link can introduce uncertainty in the inelastic behaviour of the frame as the two links do not necessarily equally share the inelastic deformation, as the nomenclature suggests.

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

1.7.2 Steel structures seismic performance in past earthquakes

Steel structures have been always considered as a suitable solution for constructions in high seismicity areas. In the design practice, and for long time, it was generally recognised that steel is an excellent material for seismic-resistant structures, due to its performances in terms of strength and ductility, as it is capable of withstanding substantial inelastic deformations. This is commonly true, as the percentage of failed of steel structures has been always very minor as compared to other constructional materials, that is the reinforced concrete for example.

However, during last decades, specialists have recognized that the so-called good ductility of steel structures under particular conditions may be only a belief, which is denied by the reality. In fact, the recent earthquakes of Mexico City (1985), Lorna Prieta (1989), Northridge (1994) and Kobe (1995) have seriously compromised this ideal image as a perfect material for seismic areas. The performance of steel joints and steel members have shown in some cases a very poor and the same type of damage was produced in different events which clearly shows that there are some lacks in the current design practices. Thus, it seems to be the right moment to analyse the progress recently achieved in conception, design and construction, considering the lessons learned from the last dramatic events.

In buildings, earthquake performance can be divided into two categories: structural and non-structural, both of which when unsatisfactory can be hazardous to building occupants when damage occurs. Structural damage means degradation of the building’s structural support systems (i.e., vertical and lateral force resisting systems), such as the building frames and walls. Non-structural damage refers to any damage that does not affect the integrity of the structural support system **(Hamburger 2006).**

1.7.3 Basic performance seismic steel structural typologies

1.7.3.1 General

Steel frames can be subdivided into two categories: unbraced and braced frames. Unbraced frames are frame in which the resistance to lateral load is provided primarily by the bending resistance of the frame members and their connections. While braced frames are frames in which the resistance to both lateral load and frame instability is provided by the combined action of floor diaphragms and a structural core, shear walls, and/or a diagonal, K-brace, or other auxiliary system of bracing. Usually, two types of structural systems have been used in multi-storey buildings in seismic zones: Moment-resisting Frames (MRFs) and Braced Frames (BFs). The braced frames may be divided into two structural systems: Concentrically Braced Frames (CBF) and Eccentrically Braced Frames (EBF). It must be noted that a given structure can suffer severe damage under seismic loadings if the choice of adequate structural system due to the lack of knowledge is made.

1.7.3.2 Moment-resisting frames MRFs

MRF's are preferred for their architectural adaptability with no bracing schemes which block wall openings and the maximum flexibility and then a wider space utilisation is provided. However, this advantage is accompanied by a poor lateral stiffness of the whole structure, particularly when the height of the building increases under moderate earthquakes or severe earthquakes. MRFs are rectilinear assemblages of beams to columns. Furthermore, in moment-resisting frames, the horizontal forces are mainly resisted by members acting in an essentially flexural manner. Their main source of lateral stiffness and strength are the bending rigidity and strength of the frame members.

Steel moment resisting frames (MRFs) are typically drift sensitive structures. Then, for high rise buildings the fulfilment of the requirements, which are necessary to guarantee the check against the serviceability limit state, can be very severe and consequently MRFs become uneconomical in developing the design stiffness required by the drift control **(Bruneau 2011),** hence leading to design larger member sizes than those required by the strength condition, due to the need to contain sway (lateral) deflections within the drift limits, imposed by the seismic codes **Bruneau (2011),** which influences the design of MRF's. As a direct consequence, for MRFs the need to reduce inter-storey drift ratios and P-Δ effects could lead to oversize the structural members in order to increase the lateral stiffness **(Tenchini 2014).**

1.7.3.3 Concentrically braced frames (CBFs)

In order to control lateral drifts, concentric braced frames (CBF) can be used. Since diagonal braces increase the lateral stiffness of the system, CBF can resist against lateral forces during both minor and moderate earthquakes. Using this kind of structures is generally more economical than increasing element sizes and using doubler plates. However, during a major earthquake, these lateral forces can increase significantly and generally diagonal bracing struts buckle due to the cyclic axial load. Concentrically braced frames resist lateral loads primarily by developing high axial forces in diagonal members that is in the braces. In general, the dissipative zones are located in the tensile diagonals, because of the assumption usually made that the compression elements may buckle.

The seismic performance of bracing members of CBFs is characterized by cyclic buckling under compression and subsequent tension yielding when the diagonals are straightened. The buckling mode induces the formation of plastic hinges at both the centre and the ends of the braces and the relevant plastic engagement depends on the orientation of the brace cross section and its connection to the beams and columns, which can be designed as rotationally rigid or pinned. In both cases, the brace end connections must be able to transfer the axial cyclic tension and compression to the rest of the structure and they must be detailed to confine plastic rotation to the ends of bracing (if designed as rigid) or to guarantee adequate flexural ductility to allow end rotations of the braces (if designed as pinned).

The inelastic cyclic performance of concentric braces is affected by an energy dissipation capacity degradation, because of the repeated buckling of diagonal members. For this reason, the q reduction factor is assumed to be lower than in MRF structures depending on the location of the diagonal members. Due to the presence of braces, CBF structures have an additional high elastic stiffness.

- **X-braced frames (CBFXs)** are among the most economical and popular configurations of braced frame structure, particularly in industrial applications, where the architectural impacts of large diagonals across a bay are often not a concern. Many X-braced frames are designed assuming that the braces are capable of resisting tensile loads only. When this approach is taken, the designer typically assumes that the compressive braces buckle under negligible load and can therefore be neglected in the analysis, resulting in a statically determinate structure, and a simplification of the design.
- **Chevron and V-braced frames (CBFV and IVs)** are very popular for commercial building construction because they are an economical alternative and also because the V or inverted-V pattern of the bracing provides opportunity for relatively large and unobstructed window and door openings. When these frames are loaded laterally, the lateral loading will tend to induce tensile forces in one brace in each bay and compression forces in opposing braces.

1.7.3.4 Eccentrically braced frames (EBFs)

Eccentric braced frames (EBFs) are a relatively recent innovation, developed in the early 1980s, primarily on the basis of research conducted at the University of California at Berkeley **(Popov 1988).** Eccentrically braced frame (EBF) is a hybrid system which is a combination of moment resisting frame (MRF) and concentrically braced frame (CBF) **(Okazaki 2004).** In this framing system, the diagonal braces are intentionally configurated such that their work points are non-concentric either with beam column joints in the case of single-diagonal systems, or with other braces in multi-diagonal systems. These systems are intentionally designed so that nonlinear behaviour is developed through plasticity in the beams **(Hamburger 2003)**.

This protects the braces from buckling and results in a substantial amount of energy dissipation, a desirable property for earthquake resistance. Either shear plasticity or flexural plasticity may occur, depending on the link length. The resulting eccentricity induces flexural and shear stress in the beams. The segment of the beam in which the plasticity occurs is called the link and the beam itself is called the link beam. Beams with long links will be controlled by flexural plasticity and the development of plastic hinges at either end of the link. Beams with short links will develop plasticity through shear yielding distributed along the length of the link.

With a proper design, ductility of MRF and drift control capacity of a CBF can be obtained economically through the use of an eccentrically braced frame. The horizontal forces are mainly resisted by axially loaded members, but the eccentricity of the layout of the bracings is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear. EBFs should be designed to enforce plastic engagement of all dissipative links (EC8 clause 6.3.1(4)). In fact, they combine the individual advantages of moment-resisting frames and concentrically braced frames, assuring a high elastic stiffness, together with stable inelastic response under cyclic lateral loading and good ductility and energy dissipation capacity.

EBFs are characterised by diagonals eccentrically located in moment-resisting frames, producing a stiffening effect. In such a way the beams are divided in two or three parts, being the shortest one the dissipative element of the eccentrically braced frame, called "link", which dissipates the earthquake input energy by means of the inelastic cyclic shearing and bending. The assumed value of the q-factor in this case is almost the same than the one corresponding to MRF, depending on the location of the diagonal members **(Mazzolani 2012; Bruneau 2011).** However, critical consideration in the design of EBFs include providing adequate bracing of the link beam, so that it can develop plasticity without experiencing flexural-torsional buckling, and provision of sufficient stiffener plates within the web of the link beam to avoid lateral buckling of the web. Design specifications include prescriptive requirements for these aspects of the design. In addition, the columns of the EBF frame must be designed with sufficient strength to resist the axial forces resulting from development of a full mechanism in the frame **(Hamburger 2003).**

1.7.3.5 Comparison

A comparison between MRF and braced frames (CBF and EBF) can be qualitatively given on the base of the requirements which a seismic resistant structure has to satisfy: strength and stiffness against moderate ground motions with a small return period; strength, ductility and energy dissipation capacity against severe earthquakes with a great return period. It can be synthesised as in Table 1.1 **(Mazzolani 1995).**

Table 1. 1. Comparison between steel lateral systems **(Mazzolani 1995).**

System	Strength	Stiffness	Ductility
MRF	Good	Poor	Good
CBF	Good	Good	Poor
EBF	Good	Good	Good

However, in most design cases, the building structure combines MRF with braced frames (either CBF or EBF), resulting in Dual Frame systems, whereby both components are able to dissipate seismic energy. EC8 gives roughly some requirements for dual frames made of MRFs with X-CBFs, but it does not provide recommendations for other cases.

1.7.3.6 Dual structures

Dual systems, composed of a combination of the two systems, designed to act in unison **(Hamburger 2003).** The structural solutions ranging between the CBF, or EBF and the MRF are called dual structures, because horizontal loading are resisted in part by moment resisting frames and in part by bracing systems acting in the same plane. A similar evolution can be achieved starting from a classical one bay CBF system, with e = 0, which is transformed in a MRF passing through EBF systems, characterised by different dimensions of the links, from short to long.

However, in most design cases, the building structure combines MRF with braced frames (either CBF or EBF), resulting in Dual Frame systems, whereby both components are able to dissipate seismic energy. EC8 gives roughly some requirements for dual frames made of MRFs with X-CBFs, but it does not provide recommendations for other cases.

In conclusion, the generation of such structural systems offers the possibility to obtain a wide range of structures, laying between a very rigid CBF up to a very ductile MRF, which can be characterised by a given combination of stiffness and ductility, for the same strength requirement.

1.8 STRUCTURES WITH SOFT OR WEAK STOREY

1.8.1 General

Buildings that have simple, regular, symmetric configuration generally display the best performance in earthquakes. Configuration or the general vertical and/or horizontal shape of buildings is an important factor in earthquake performance and damage. When a moment-resisting frame structure becomes completely plastic it is said to form a mechanism, that is, a behavioural mode in which the structure exhibits neutral stability and can deform laterally without application of load. Frame structures can form a variety of different types of lateral mechanisms.

The reasons for this are: (a) non symmetric buildings tend to have twist (i.e. have significant torsional modes) in addition to shaking laterally and (b) the various ‘‘wings’’ of a building tend to act independently, resulting in differential movements, cracking, and other damage. Rotational motion introduces additional damage, especially at re-entrant or ‘‘internal’’ corners of the building.

Broadly speaking, in earthquake resistant design, the term of soft-storey is the structural condition of a significant disparity between lateral resistances in a multi-storeys structure in lateral load resistance than the stories above, while the term weak storey refers to irregularity in strength.

The soft first storey condition is recognized as an undesirable condition of structural irregularity for seismic design worldwide. The soft first-storey is the most common feature of soft-storey irregularity and cannot be eliminated because of its important functional requirement of almost all the urban multi-storey buildings. The problem has to be tackled with a holistic approach. The codes set a somewhat arbitrary threshold to identify the parameters that are involved in such situations. It is worth to remark that Algerian Seismic Code RPA99 version 2003 **(RPA99 2003)** does not address clear provisions about the concept of soft/weak-storey design methodologies for steel structures. In the United States, the UBC code **(UBC 1997)** also included the soft storey condition since 1988 as one of the recognized vertical structural irregularities to account for design, and most recent regulations such as ASCE (2010) and IBC-06 (2006) endorse also these recommendations.

The term configuration also refers to the geometry of lateral load resisting systems as well as the geometry of the building. Asymmetry can exist in the placement of bracing systems, shear walls, or MRFs that are used to provide earthquake resistance in a building **(Chen 2006).**

1.8.2 Soft-storey

1.8.2.1 Introduction

The soft first storey condition is recognized as an undesirable condition of structural irregularity for seismic design worldwide. The soft first-storey is the most common feature of soft-storey irregularity and cannot be eliminated because of its important functional requirement of almost all the urban multi-storey buildings.

The problem has to be tackled with a holistic approach. The codes set a somewhat arbitrary threshold to identify the parameters that are involved in such situations. It is worth to remark that Algerian Seismic Code RPA99 version 2003 does not address clear provisions about the concept of soft/weak-storey design methodologies for steel structures.

1.8.2.2 Concept

This concept is an attempt to reduce acceleration in a building by allowing the first-storey column to yield during an earthquake and produce energy-dissipation action. However, excessive drifts in the first storey coupled with P-Δ effects on the yielded columns make buildings collapse. Soft storey creates a major weak point during an earthquake event, and since soft stories are classically associated with retail spaces and parking garages, they are often on the lower stories of a building, which means that when they collapse, they can take the whole building down with them, causing serious structural damage which may render the structure totally unusable **(Hejazi 2011; Ricci 2012).** As reported in **Hejazi (2011),** the concept of the soft storey has been recognized long back by **Fintel (1969).** The concept has been studied further since then and it is now well understood and discussed in literature.

1.8.2.3 Definition

Soft-storeys create hazardous conditions and experience large earthquake - induced displacements and, in turn, cause extensive damage and even collapse. Soft storeys often (but not always) occur on the ground floor, where commercial or other reasons require a greater storey height, and large windows or openings for ingress or commercial display (e.g., the building might have masonry curtain walls for the full height, except at the ground floor, where these are replaced with large windows for a store's display), Figure 1.8.

As stipulated in **(UBC 1997)** soft storey irregularity exists when the storey stiffness is less than 70% of the storey above or 80% of the average of the three stories above. In multi-storeys structures, extreme soft storey irregularity exists when the storey stiffness is less than 60% of the storey above or 70% of the average of the three stories above.

1.8.2.4 Characteristics

Soft storey (irregularity in stiffness) occurs in buildings whenever the stiffness of a storey to resist lateral demands is significantly less than that of adjacent storeys. This is because structural systems with this configuration tend to develop inelastic behaviour at the most vulnerable storey **(Elnashai 2008).** As a result, significant changes in load paths and deformation patterns arise. Due to inadequate stiffness, a disproportionate amount of the entire building's drift is concentrated at the soft storey, resulting in non-structural and potential structural damage. Many older buildings with soft stories but built prior to recognition of this aspect collapse due to excessive ductility demands at the soft storey.

Furthermore, as in **(Chen 2006),** even structures that have uniform distributions of elastic stiffness can develop inelastic soft-storey conditions if an inappropriate pattern of plastic hinges form in the structure as it undergoes nonlinear lateral deformation. Frame structures can form a variety of different types of lateral mechanisms.

The design is accordingly safer, as soft stories are assigned higher sway imperfections. It must be mentioned that the intent of this irregularity check is to compare values of lateral stiffness of individual stories. It is common to compare displacements rather than the stiffness of the floors of the structure as the stiffness can be difficult to compute and the displacements can be readily obtained from computer generated analyses programs. When the storey heights vary significantly from floor to floor, storey drift ratios (the storey drift divided by the storey height) are a better comparison.

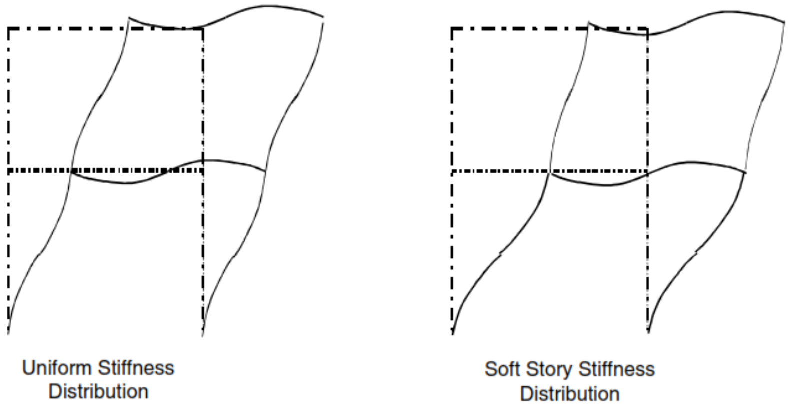


Figure 1. 8. Mode shape for structures with uniform stiffness and soft-storey stiffness distributions.

1.8.3 Weak-storey

Weak storey (irregularity in strength) is the case where the storey strength is irregular between two adjacent storeys. The weak storey effect consists in the concentration of inelastic activity which can produce the total or partial collapse of the storey itself **(Gioncu 2011).**

In some seismic codes EC8 and AISC, provisions are provided to assure that weak storey conditions will not occur in this system i.e. SCWB principle, in the moment capacity of the columns exceeds that of the beams framing into and consequently discouraging a soft storey mechanism to occur in a shear-type structures.

1.8.4 Collapses of weak/soft-storey steel structures

In a general sense, collapse refers to a system's failure in its capacity to carry vertical loads. In earthquake engineering, structural collapse is defined as the local or global failure of a system that occurs due to the loss of vertical load-carrying capacity in the presence of seismic events. The two primary modes of global collapse are side-sway and vertical collapse. Commonly, in ductile frame structures, side sway collapse is the predominant mode of collapse during earthquake events. Sideway or incremental collapse comes as a result of dynamic instability, which can be defined **(Bernal 1998)**: Disproportionate system response to a dynamic loading's small variation of intensity in a lapse of time. Historically, a typical mistake when designing a building has been to use columns in the bottom floor and stiffer bracing elements in the other stories **(Ricci 2012)**. If plastic hinges form in the columns first, there is a risk of developing a soft-storey mechanism. This happens when one of the stories develops a horizontal stiffness that is a lot smaller than the horizontal stiffness of the other floors **(Bachmann 2002).**

Side-sway collapse is the global failure of the system caused by a reduction of the lateral load-bearing capacity due to large horizontal displacements, whereas vertical collapse is caused by a direct loss of the gravity load-bearing capacity in one or several structural components

(Krawinkler 2009a). Vertical collapse is a type of progressive collapse, which is the total or disproportionate failure of the system triggered by an initial local failure that spreads out from element to element.

In Figure 1.9, is shown the effect of WBSC principal and WCSB effect on the plastic mechanism. It is evident that the formation of a soft storey plastic mechanism should also be avoided.

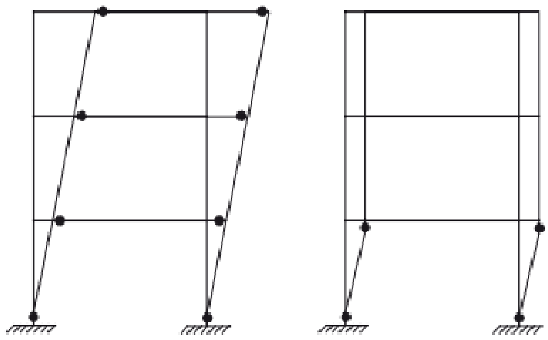


Figure 1. 9. Weak beam/strong column and weak column/strong beam behaviour in moment-resisting **(Elghazouli 2016).**

1.9 STEEL STRUCTURES TO SEISMIC CODES

1.9.1 RPA99 PROVISIONS

Most of the buildings in Algeria are consisting of reinforced concrete elements and very few are constructed using steel elements, are normally constructed to resist both static and seismic loads. However, for latter structures, i.e. steel structures, insufficient provisions are being addressed in RPA99 Seismic Algerian code to be applied for elastic and inelastic behaviour of such structures. However, this leads to deficiencies in the design of the structures. Generally, multi-storey steel framed buildings are required by building codes, not yet in RPA99, to exhibit a degree of fire resistance that is dependent on the building form and size.

1.9.1.1 General principals

The design, sizing and execution of the steel structures in seismic zones must simultaneously satisfy the rules set out in this document and those prescribed by the other regulations such as Algerian Steel Structures Code **(CCM97 1997)**, which is roughly the corresponding EC3 Chapter 8 of the current Algerian Seismic Code **(RPA99 2003)** is devoted to steel structures in association with other chapters for general rules. The whole Chapter 8 is consisting of less than 4 pages including tables and Figures. Indeed, very few provisions are presented dealing with general concept in steel structures. It seems that RPA99 version 2003 has not uses the advanced concepts and philosophy of steel design over the world. A full discussion of this matter will be held at the end of the chapter when comparing these provisions with more developed seismic codes: EC8 and AISC 2006.

1.9.1.2 Ductility

Structures must have a satisfactory ductility and energy dissipation capacity to allow the structure to undergo inelastic displacements with limited damage and without collapse or loss of stability, in the face of a major, rarer earthquake. In RPA99 seismic code, ductility is defined as the ability of a material, a section, an element or a structure to undergo irreversible deformations before rupture without significant loss of resistance under alternating stresses. It is also stated that ductility failure is defined as the failure which is preceded by irreversible deformations undergo by the structure or a member of structure unlike a brittle failure which is sudden and almost instantaneous. Structures have to meet the following criteria of ductility, firming and elongation at break:

- Ductility criteria: $(\varepsilon_u/\varepsilon_y) \geq 20$
- Strengthening criteria: $(f_u/f_y) \geq 1.20$
- Criterion of elongation at break: $A_r \geq 15\%$

In addition, in seismic zones, only rigid connections are authorized (semi-rigid connections are not allowed).

1.9.1.3 Different structures topology

A series of classical steel configurations have been adopted in the current seismic Algerian code.

- **Ductile moment resisting frames system DMRFs**

The whole structure carries the total vertical loads. The ductile moment resisting undergo the totality of horizontal forces. These frames should be designed and executed according to the requirements given in the paragraph 8.2.

- **Structures braced with ordinary moment resisting space frames BOMRFs**

The complete structure carries the total vertical loads. The moment resisting frames resist the total horizontal loads and should satisfy to the requirements given in paragraph 8.1. The height of all the buildings using this bracing system should be limited to 5 stories or 17 m in height.

Note: The bracing systems 7 and 8 suppose the use of light infill elements compatible with the considered structural systems and that do not prevent the displacements of the structure.

- **Structures braced by concentric braced frames CBF's**

The complete structure carries the total vertical loads and the braced frames carry the total horizontal ones. The concentrically braced frames should satisfy the requirements given in the paragraph 8.4. The height of the buildings using this braced system should be limited to 10 stories or 33 m in height.

For this bracing system category, there are (02) subcategories, that are X and V bracing systems (the K bracing system category is not tolerated).

- **Structure CBFX**

In this system, for a C frames, the axes of the diagonals, the beam and the column are convergent to one point located in the centre of the node. Besides, in this system, only tensioned diagonals, with a dissipative behaviour, contribute to the resistance to seismic action.

- **Structure CBFV**

In this system, the beams of each braced frame are continuous and the point of intersection of the diagonal axes of the braced frame is located on the axis of the beam, see Figure 1.10.

The resistance and the dissipation capacity of the braced frame in regard to the seismic action are provided both tensioned and compressed diagonals, see Figure 1.11.

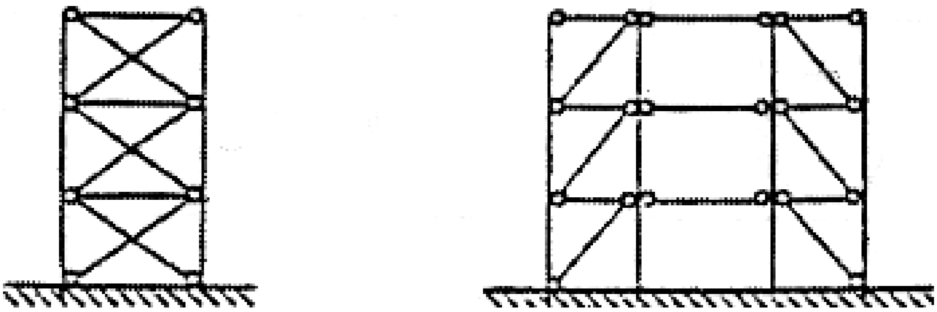


Figure 1. 10. CBFX structures **(RPA99 2003)**

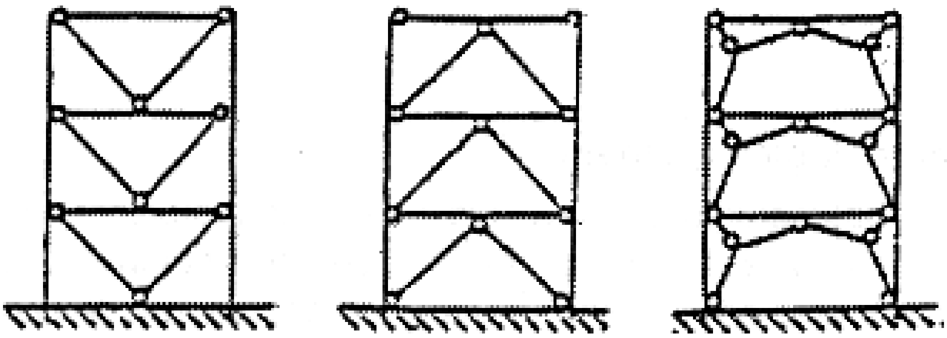


Figure 1. 11. CBFV and IV structures **(RPA99 2003)**

- **Dual bracing systems**

In this case, the braced frames should not carry more than 20% of the vertical loads. A dual bracing system is a combination of two (02) types of bracing systems, chosen among those previously defined.

It is composed by ductile moment resisting space frames coupled with X or V braced frames or closer to the V type (double brackets system).

The complete structure carries the total vertical loads while the dual bracings (moment resisting frames and braced frames) carry the total horizontal loads.

The moment resisting frames and the braced frames should be designed to resist horizontal load according to their relative rigidities considering the interaction at all levels.

The ductile moment resisting frames should have the capacity to resist alone not less than 25% of the global horizontal loads.

The requirements concerning these bracing system categories are specified in the paragraph 8.5.

- Structural system braced with ductile frames and X braced frames

In this system, the dual bracing system is a combination of ductile moment resisting space frames and concentric X braced frames.

- Structural system braced with ductile frames and V braced frames

In this system, the dual bracing system is a combination of ductile moment resisting frames and concentric V braced frames.

- Vertical cantilever frame system

This category of structural system with small degree of redundancy concerns essentially classical single-storey frames with a rigid transversal beam and slender structures of « tube » type where the resistant structural elements are essentially the columns located on the periphery of the structure. These particular structures have a dissipative behaviour located uniquely at the ends of the columns.

- System including transparencies (soft storeys)

The most illustrative examples are given by the reception levels or lobbies of hotels (rare separation walls or storey height more important than for the current stories) or absence of separation walls at some stories for some specified reasons (computer rooms, special equipment, etc.).

In general, these systems should be avoided. Otherwise, besides the procedures previously recommended for the systems 1a and 1b for the specific case of the ground floor (change of dual systems), all the arrangements to decrease the unfavourable effects should be taken.

In this respect, the procedure to increase the rigidity should be adopted in order to decrease or attenuate the phenomena (see the definition of the soft storey given previously as remarks for systems 1a and 1b).

1.9.1.4 Connections

In seismic zones, only rigid connections are authorized (semi-rigid connections are not allowed) see clause 8.1.1 RPA99. The joints must be calculated to allow the maximum forces to be developed in the element or must be calculated on the basis of 1.6 times the force determined in 4.2.1of RPA99. Connections in tension must be used with prestressed bolts, with high resistance, and with controlled tightening. Further provisions are given in the clause 8.2.4:

a) Each connection in MRFs must be of the rigid type and be capable of developing the total plastic capacity in the beam.

b) In the case where the connection of MRFs are of bolted type, these connections must be designed, calculated and produced as connections in shear, and resisting to sliding in the ultimate limit state under seismic action by means of high strength prestressed bolts with controlled clamping.

c) For steel with specified ultimate strength is less than 1.6 times the specified yield strength, the plastic hinges must be developed in beams, during the inelastic deformations of the frame, must not appear in places where the flange section has been reduced, for example by bolt holes. As an indication, this condition is fulfilled for steels of grades FE 360 and FE 430 but it is not necessarily so for grade FE 510.

d) The weld-filled seams of welded assemblies of freestanding gantry cranes must be checked by non-destructive testing methods in accordance with standards, particularly for structures of groups 1A and 1B located in seismic zone III.

1.9.2 Lateral steel resisting systems to EC8

Moment Resisting Frames (MRF) and Concentrically Braced Frames (CBF) and Eccentrically Braced Frames EBF are the most commonly utilized systems in the EC3, EC8, and ANSI/AISC, etc. Steel buildings shall be assigned to one of the following structural types according to the behaviour of their primary resisting structure under seismic actions (see Figures 1.11 and 1.12).

- **Moment resisting frames (MRFs)**

Are those in which the horizontal forces are mainly resisted by members acting in an essentially flexural manner. Three types of moment resisting frame are considered, as shown in Figure below: Portal frames; one bay multi-storey frame; Multi bay multi-storey frame, see Figure 1.12.

In moment resisting frames, the dissipative zones should be mainly located in plastic hinges in the beams or the beam-column joints so that energy is dissipated by means of cyclic bending. The dissipative zones may also be located in columns:

− At the base of the frame;
− At the top of the columns in the upper storey of multi-storey buildings;
− At the top and bottom of columns in single storey buildings in which N_{Ed} in columns conform to the inequality: $N_{Ed}/N_{pl,Rd} < 0.1$.

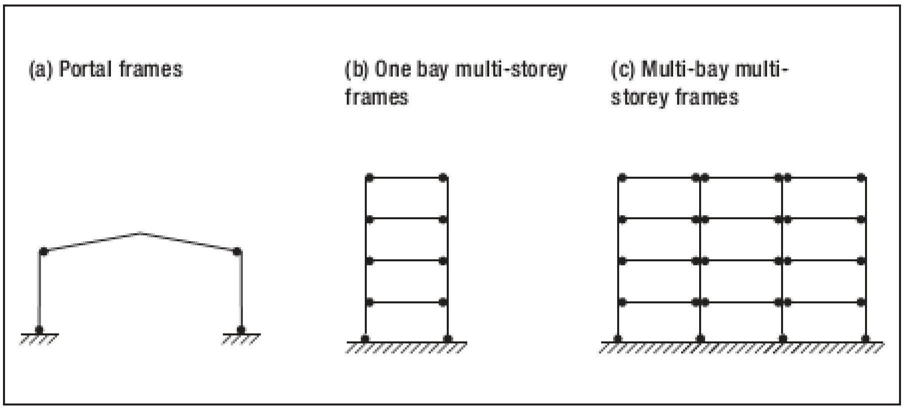


Figure 1. 12. Moment resisting frames (dissipative zones in beams and column bases) **(EC8 2005).**

- **Frames with concentric bracings**

Are those in which the horizontal forces are mainly resisted by members subjected to axial forces. In frames with concentric bracings, the dissipative zones should be mainly located in the tensile diagonals.

The bracings may belong to one of the following categories:

− **Active tension diagonal bracings:** In which the tension diagonals can resist the horizontal forces only, neglecting the compression diagonals;

− **V bracings:** In which the horizontal forces can be resisted by taking into account both tension and compression diagonals. The intersection point of these diagonals lies on a horizontal member which shall be continuous.

− K **bracings:** In which the intersection of the diagonals lies on a column (Figure 1.13) may not be used.

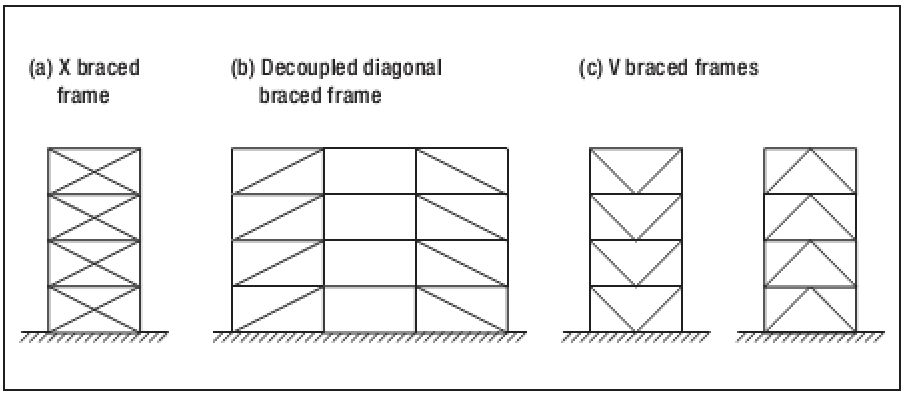


Figure 1. 13. Concentrically braced frames (dissipative zones in tension braces) (EC8 2005).

- **Frames with eccentric bracings (not covered in RPA99):**

Are those in which the horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear. For frames with eccentric bracings configurations should be used that ensure that all links will be active, as shown in Figure 1.14.

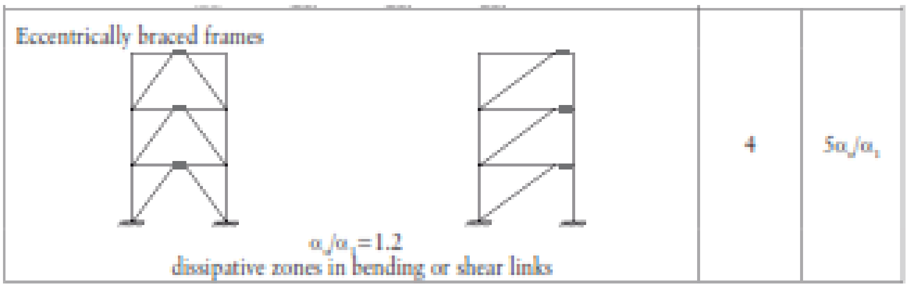


Figure 1. 14. Eccentrically braced frames (dissipative zones in tension braces) (EC8 2005).

- **Inverted pendulum structures:**

Are structures in which dissipative zones are located at the bases of columns. Inverted pendulum structures may be considered as moment resisting frames provided that the earthquake resistant structures possess more than one column in each resisting plane and that the following inequality of the limitation of axial force: $N_{Ed} < 0.3\ N_{pl,\ Rd}$ is satisfied in each column.

- **Structures with concrete cores or concrete walls:**

Are those in which horizontal forces are mainly resisted by these cores or walls.

- Moment resisting frames combined with concentric bracings.
- Moment resisting frames combined with in fills.
- Dual frames.

A dual system is one in which, according to the results of the analysis, between 35 and 65% of the seismic base shear is (or rather should be) resisted by frames of primary seismic beams columns, and the rest of the seismic base shear resisted by primary seismic walls. Dual systems combine the satisfactory stiffness, force resistance and cost-effectiveness of walls with the ductility and large deformation capacity of frames, which can act as a second line of defence in case (some of) the more brittle walls of the system fail. Moreover, dual systems use to advantage the beams and columns that carry (most of the) gravity loads for the lateral force resistance, as well as the capacity of columns to resist lateral forces in both horizontal directions. Their inelastic behaviour, though, is much more uncertain than that of pure Frame or wall systems.

1.9.3 Structural systems to AISC

Five basic moment-resisting frame systems are available in the building codes **(Hamburger 2003)**. These are termed:

- Special moment-resisting frames
- Intermediate moment-resisting frames
- Ordinary moment-resisting frames
- Special-truss moment frames
- Non detailed moment frames

Three groups of structural systems are considered in the AISC standard to resist seismic effect: regular moment frames, truss moment frames and braced frames. Regular frames include ordinary moment frames (OMF), intermediate moment frames (OMF) and special moment frames (SMF). Only one type of truss moment frame is given in the code: special truss moment frames (STMF). Braced frames include ordinary concentrically braced frames (OCBF), special concentrically braced frames (SCBF) and eccentrically braced frames (EBF). Key details on each of these systems will be presented in the next section.

- **Ordinary Moment Frames (OMF)**

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformation in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OMF shall meet the requirements of this section 11in (AISC 2005). Connections in conformance with sections 9.2b and 9.5 or sections 10.2b and 10.5 in (AISC 2005) shall be permitted for use in OMF without meeting the requirements of sections 11.2a, 11.2c, and 11.6. The code permits fully restrained (FR), requirements for FR Moment Connections can be found in 11.2a, as well as partially restrained (PR) requirements, PR Moment Connections (see 11.2b) moment connections to be used in OMF. To achieve this expectation, the code has requirements for beam-to-column connections and continuity plates. Beam-to-column connections shall be made with welds and/or high-strength bolts.

- **Special Moment Frames (SMF)**

Special moment frames are expected to achieve significant inelastic deformations capacity when subjected to the forces resulting from the motions of the design earthquake. Moment frame system that meets the requirements of section 9 **(AISC 2005).** The requirements of special moment frames are given in terms of beam-to-column connections, panel zones of beam-to-column connections, lateral support of beams and strength requirements.

- **Special truss moment frame (STMF)**:

(STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the design earthquake. STMF shall be limited to span lengths between columns not to exceed 65 ft (20 m) and overall depth not to exceed 6 ft (1.8 m). The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in section 12 in (AISC 2005).

- **Configuration of Concentrically Braced Frames**

X-braced frame: Concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

V-braced frame: Concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.

- **Ordinary concentrically braced frame (OCBF)**

Diagonally braced frame meeting the requirements of section 14 in which all members of the bracing system are subjected primarily to axial forces. Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OCBF above the isolation system in seismically isolated structures shall meet the requirements of sections 14.4 and 14.5 (AISC 2005) and need not meet the requirements of sections 14.2 and 14.1. Bracing members shall meet the requirements of section 8.2b. Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-column connection and shall meet the requirements detailed in section 14.4.

- **Special concentrically braced frame (SCBF)**

Diagonally braced frame meeting the requirements of section 13 in which all members of the bracing system are subjected primarily to axial forces are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. K-type braced frames are not permitted for SCBF. Requirements of V-Type and Inverted -V-Type Bracing are detailed in section 11.a.

- **Eccentrically braced frame (EBF)**

Diagonally braced frame meeting the requirements of section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection. In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column see section 15.2. The length of the *link* is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face. Y-braced frame. Eccentrically braced frame (EBF) in which the stem of the Y is the *link* of the EBF system. Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the *links* when subjected to the forces resulting from the motions of the *design earthquake*.

The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links, except where permitted in this section. In buildings exceeding five stories in height, the upper storey of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the applicable building code.

If the EBF system factors in the applicable building code require moment resisting connections away from the link, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in sections 11.2 and 11.6.

If the EBF system factors in the applicable building code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

Links in EBFs are a *protected zone*, and shall satisfy the requirements of section 7.4. Welding on links is permitted for attachment of link stiffeners, as required in section 15.1.

- **Moment resisting frames (MRFS)**
    - **Frame characteristics**

Moment resisting frames (MRFs) are designed based on strong column-weak beam design methodology, as results of which that plastic hinges occur predominantly in beams rather than in columns (weak beam/strong column design) as shown in Figure.1.13(a). The plastic hinges occur near the beam column joints during an earthquake event without failure and the inelastic action distributed in the structure at beam ends can dissipate large amount of energy. This provides favourable performance, compared to strong beam/weak column behaviour through which significant deformation and second order effects may arise in addition to the likelihood of premature storey collapse mechanisms as clearly shown in table 1.1. The only exception to this requirement is at the base of the ground floor columns, where plastic hinges may form.

Through this philosophy, MRFs can be designed to remain ductile and subsist from a major earthquake without failure. However, there are also some drawbacks of this framing system. Since the beams have large flexural deformation capacity, large lateral drifts can be observed. Increasing the element sizes can be the first solution coming up in mind to solve this problem but it may also be uneconomical. The second problem which can be observed about this kind of framing is the distortion at column panel zones due to the large shear forces that occurs by transferring of moment from beam to column. This situation leads in an increase of the lateral drift of the system and results in larger P-Δ second order effects. To strengthen the columns, web doubler plates are generally used in column panel zones. Consequently, the final cost of the structure rises with these expenses.

CHAPTER 2:

FUNDAMENTALS OF SEISMIC DESIGN

CHAPTER 2: FUNDAMENTALS OF SEISMIC DESIGN

The purpose of this chapter is to describe briefly some of the concepts and procedures underlying modern earthquake engineering, especially as it applies to buildings. This chapter will provide the recent advances in seismic design of building; particularly those located in seismic areas and are low and medium-rising steel structures. An emphasize on their seismic performance beyond the elastic range under large inelastic deformations.

2.1 INTRODUCTION TO THE DESIGN PROCESS

2.1.1 General

Whether enshrined in law, statutory regulation, contractual requirements, good practice or moral code, designing for safety is recognised as an absolute requirement. Important principles which should be implemented when carrying out project design work are: identification of the designer's absolute duties, and those where the designer has to take reasonable steps identification of design responsibilities that lie within the construction chain, from client, to subcontractor to user hazard elimination and risk reduction (eliminate, reduce, inform, control), to guarantee the fundamental need for salient information particularly at sensibility or contractual and interfaces **(Rathbone 2008)**.

It is of great value for engineers to understand the intricacies of the design process and its impact on final product. Design can be described as the process of conceptually creating something that does not yet exist. To accomplish this, designers make use of their knowledge of material behaviour and/or of processes coupled with their ability to analyse (i.e. predict) the future behaviour of their design to meet some specified need. The success of the design is highly dependent on the thoroughness of the designer's knowledge and a clear understanding of the expected behaviour of the final product (**Quimby 2011)**.

2.1.2 Structural design

The design process for a structural steel building is iterative in nature and usually starts out with some schematic drawings developed by the architect for the owner of a building. Using these schematic drawings, the structural engineer carries out a preliminary design to determine the preliminary sizes of the members for each structural material and structural system under gravity and lateral loadings.

This information is used to determine the most economical structural material and structural system for the building. After the structural material and systems are determined, then comes the final design phase where the roof and floor framing members and the lateral load systems are laid out and all the member sizes are proportioned to resist the applied loads with an adequate margin of safety.

This results in a set of construction documents that include structural plans, sections, details, and specifications for each of the materials used in the project. After the final design phase comes the shop drawing and the construction phases during which the building is actually fabricated and erected.

During the shop drawing phase, the steel fabricator's detailer uses the structural engineer's drawings to prepare a set of erection drawings and detail drawings that are sent to the structural engineer for review and approval. The shop drawing review process provides an opportunity for the design engineer to ensure that the fabrication drawings and details meet the design intent of the construction documents **(Aghayere 2009)**.

2.1.3 Structural design steps

Preliminary design is the time for identifying the best arrangement, shape, and sizes of structural members. During this phase, types and forms of materials are selected; and design of attachments is also developed. Finally, it includes the beginning of manufacturing plan, and development of testing. Structural design consists of three phases: conceptual design, preliminary design, and detailed design.

Conceptual design is the phase of establishing feasibility and estimate cost and risk for one or more spacecraft configurations or sets of derived requirements to support system trade studies or proposals. It also contains deriving requirements, identifying candidate types of structures, materials, and attachments. Finally, it develops the designs far enough to estimate and compare weight, cost, and risk; and select from options. Detailed design is the time for final dimensions and manufacturing tolerances, identifying fastener sizes and installation torques, designing tertiary structures such as cable-support brackets, and doing all analyses necessary to justify decisions. At the same time, the product team develops manufacturing processes and plans verification tests. Detailed design ends when the last engineering drawing for manufacturing is released.

All three phases are important. One overlooked detail during the design process and can lead to expensive drawing changes, a test failure that drives a redesign, or even mission failure. Decisions during preliminary design and how these decisions can be documented will determine how well problems are avoided during detailed design and budgeted cost and program schedules are met. Conceptual design heavily influences all that follows. The requirements and ideas carried forward from conceptual design will affect performance and cost far more than anything done later.

2.2 NATURE OF THE PROBLEM

As detailed in the preceding sections, the foremost purpose of structural design is to produce an appropriate structure, which requires a clear understanding of the role of each of element. It requires a general view on the total design process, in which the designer must consider not only the initial cost, but also the cost of maintenance, damage and failure, together with the benefits derived from the structure function. It can be observed that buildings, which are poorly designed and constructed, suffer much more damage in moderate earthquakes, than well-designed and constructed buildings in strong earthquakes.

Seismic design is slowly transforming from a stage where a linear elastic analysis for a structure was sufficient for both its elastic and ductile design, to a stage where a specially dedicated non-linear procedure is to be done, which finally influences the seismic design as a whole. The paradox is that the failure of a structure contributes more to the evolution of design concepts than Structures standing without accidents, on the condition that the engineers have the capability to understand what happened. So, the damage of a structure during an earthquake represents a challenge for structural engineers to improve the design methods.

Surprisingly enough, the failure of a structure contributes more to the improvement of the design concepts than structures standing without accidents providing that the designers have the capability to understand what exactly happened. This particular aspect can be very well illustrated with the example of steel structures.

For a long-time it was widely recognised, in design practice of structures, that steel as structural material is an excellent for seismic-resistant structures because of its high performance in terms of material strength and ductility etc. Contrary to this believe, the last severe earthquakes of Michoacan (1985), Northridge (1994) and Kobe (1995) have seriously compromised this perfect image of steel as the most suitable material for seismic resistant structures. Several causes can lead to failure of a structure and the damages depend upon and well summarised in the general characteristics of the earthquake; the local soil behaviour; the seismic vulnerability of buildings; the incomplete knowledge in seismic behaviour of structures; the inadequacy of code provisions; the wrong design, in opposition with code provisions; the bad construction; and the lack of maintenance.

2.3 CHARACTERISTICS OF THE SEISMIC LOADINGS

2.3.1 Causes

According to the theory of plate tectonics, the entire surface of the earth can be considered to be like several plates, constantly on the move. These plates brush against each other or collide at their boundaries giving rise to earthquakes. Therefore, regions close to the plate boundary are highly seismic and regions farther from the boundaries exhibit less seismicity. Earthquakes may also be caused by other actions such as underground explosions. Also, earthquakes cause the ground to shake violently thereby triggering landslides, creating floods, causing the ground to heave and crack and causing large-scale destruction to life and property.

2.3.2 Nature of earthquake loading

The nature of earthquake load differs from other loads in many respects, which makes it more difficult to design for it. Then, the behaviour under this loading is fundamentally different from wind or gravity loading, requiring much more detailed analysis, and application of a number of stringent detailing requirements to assure acceptable seismic performance beyond the elastic range. So, it is important to understand the nature of earthquakes in order to understand further their effect on structures and consequently to develop rational methods of their analysis.

2.3.3 Important characteristics

An important characteristic of earthquake loading is the uncertainty associated with its amplitude, duration, and frequency content. Another characteristic of seismic loading is the large inelastic, cyclic deformation which leads to severe damage caused by lateral loads arising from horizontal earthquake ground motion unless special provisions are made to resist them. Earthquake ground motion cyclic loading can induce reversal of stresses. Consequently, axially loaded members may have to resist both tension and compression stresses while beam cross-sections will have to resist both positive and negative bending moments.

Generally speaking, earthquake measures quantify the size and effect of earthquakes. The severity of an earthquake is described both in terms of its magnitude and its intensity. These two frequently confused concepts refer to different, but related, observations. Magnitude characterizes the size of an earthquake by measuring indirectly the energy released. By contrast, intensity indicates the local effects and potential for damage produced by an earthquake on the Earth's surface as it affects population, structures, and natural features (**Sen 2009)**.

The size of an earthquake is measured by the amount of energy released at the source, its magnitude, whereas the effect of an earthquake at different locations is measured by its intensity at a specific site. Magnitude is a quantitative measure of earthquake size and fault dimensions. It is based on the maximum amplitudes of body or surface seismic waves. It is therefore an instrumental, quantitative and objective scale. The size of an earthquake at its source is known as the magnitude of the earthquake and is measured by the Richter scale.

The intensity is a subjective measure of the local destructiveness of an earthquake at a given site and is a non - instrumental perceptibility measure of damage to structures, ground surface effects, e.g. fractures, cracks and landslides, and human reactions to earthquake shaking. Also, it is a descriptive method which has been traditionally used to establish earthquake size, especially for pre - instrumental events. Intensity scales are based on human feelings and observations of the effect of ground motion on natural and man-made objects **(Nazzal 2008)**. Basic consequences of earthquakes include collapsed buildings, fires (San Francisco, 1906), tsunamis (Indonesia, 2004) and landslides and these impacts depend on magnitude and intensity **(Sen 2009)**.

2.4 EFFECT OF EARTHQUAKE LOADINGS ON STRUCTURES

The study of the characteristics of the earthquake ground motion and its effects on engineered structures are the subjects of earthquake engineering. In particular, the effect of earthquakes on structures and the design of structures to withstand earthquakes with no or minimum damage is the subject of earthquake resistant structural design. Earthquakes can be defined as natural phenomena, which cause the ground to shake.

Unlike most other types of dynamic actions, earthquake effects are not imposed on the structure but generated by it. Therefore, two structures founded on the same soil a few metres apart may have to accommodate vastly different action and deformation demands, depending on their own mass, stiffness, strength and ductility. t is argued herein that the fundamental quantities are not period and damping, since period is a function of mass and stiffness (as well as strength in the inelastic range) and the main source of damping in earthquake engineering is energy absorption by inelastic deformation **(Elnashai 2008).**

2.5 STRUCTURAL RESPONSE PROPERTIES OF STRUCTURES TO EARTHQUAKE LOADINGS

2.5.1 General

Unfortunately, the earthquake hazard for which an element or component should be designed is subject to high degree of uncertainty **(Newmark 1982)**. So, it should be kept in mind that the great uncertainty associated with the seismic behaviour of structures.

The important properties of structures, which contribute to their elastic resistance under moderate earthquakes, are its yield strength and elastic stiffness. However, during a severe earthquake, the structure is likely to undergo inelastic deformations and has to rely on its ductility and hysteretic energy dissipation capacity to avoid collapse. Characteristics of a structure to resist an earthquake are: Ductility, Stiffness, Strength, Hysteretic energy, Overstrength, Damping. These parameters are being related to one another to the material, the section, the member, the connection and the structural system. In the following, some details will be presented for each parameter.

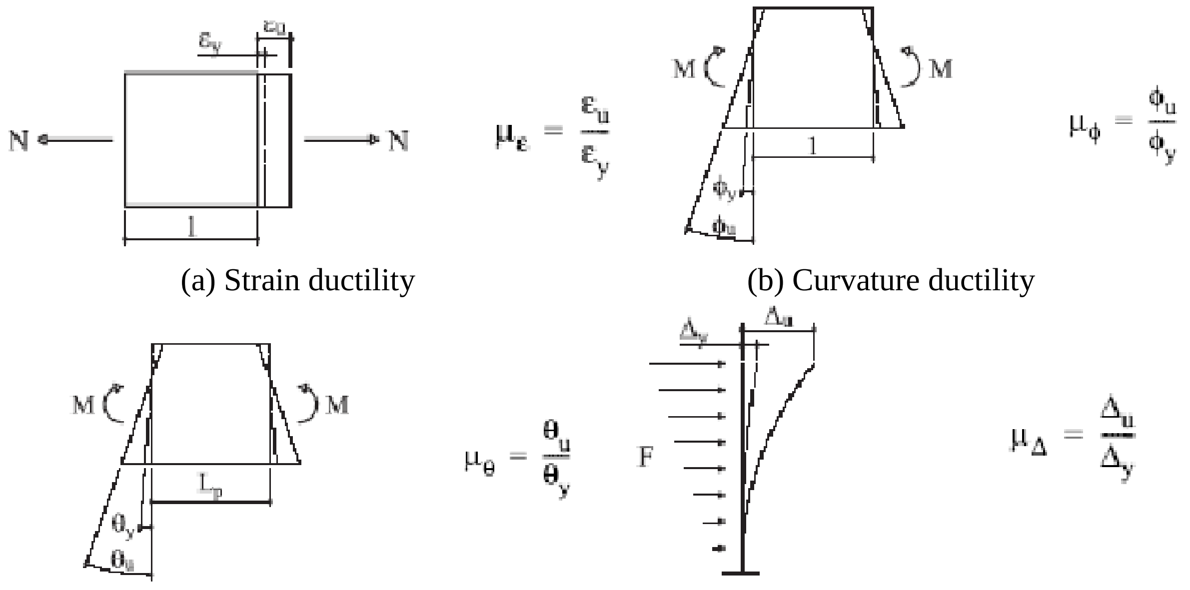
2.5.2 Ductility

It is generally acknowledged that structural safety depends on the ductility that the structural system can provide against the design loads. Ductility can be defined as the ability of a material, component, connection or structure to undergo inelastic large plastic deformations with acceptable stiffness reduction and without significant loss in strength.

Indeed, ductility represents the capacity of a mechanical system (e.g. a beam, a structure, etc.) to deform in the plastic domain without substantially reducing its bearing capacity, **(Elnashai 2008)**. According to **Bruneau (2011)**, the design of steel structures for ductile response requires: (1) material ductility; (2) cross-section and member ductility, and (3) structural ductility (it is generally preferable to protect connections against yielding, although in some instances such as with semi-rigid connections, connection ductility may be unavoidable and substitute for member ductility). Further, a hierarchy of yielding must be imposed on a structure to ensure a desirable failure mode.

Ductility coefficient is noted μ and is defined as the ratio of ductility depending on the type of studied parameter (strain, curvature, rotation and displacement) as explained in the following in which the ultimate deformation δ_u corresponds to an assumed collapse point, to the yield deformation δ_y. It may be noted that the collapse point may be assumed to lie on the descending branch of the load-deformation curve. This is still safe because earthquake loading is transient and will cease to act after a short time and so the structure will not be toppled. The ratio of the post-ultimate strength curvature at 85% of the moment of resistance capacity of a member corresponding to the curvature at yield, provided that the limiting strains of concrete and steel $f_{su,k}$ and f_y are not exceeded **(Bisch 2010).**

According to **Borzi (2009)**, types of ductility are as follows (Figure 2.1):



(a) Strain ductility (b) Curvature ductility

(c) Rotation ductility (d) Displacement ductility

Figure 2. 1. Types of ductility **(Borzi 2009)**.

Some general comments may be made about the ductility as structural property **(Borzi 2009)**:

- The ductility capacity is a property of the structural member;
- The ductility demand is a result of the seismic excitation and also a function of the dynamic properties of the structure;
- A structural member survives the earthquake if the condition is satisfied, that is: Ductility capacity ≥ Ductility demand;
- The structural member collapse when locally the deformation capacity of the structural materials (i.e., their strain capacities) are reached and exceeded. The ductility capacity is therefore exhausted.

More details will be given later on in this chapter of the ductility concept when dealing with the design capacity method.

2.5.3 Stiffness

Stiffness defines the relationship between actions and deformations of a structure and its components. Whereas member stiffness is a function of section properties, length and boundary conditions, system stiffness is primarily a function of the lateral resisting mechanisms utilized, e.g. moment–resisting frames, braced frames, walls or dual systems. Relationships between geometry, mechanical properties, actions and deformations can be established from principles of mechanics. Their complexity depends on the construction material used. Several factors can influence the stiffness: Materials properties; Section properties; Member properties; Connection properties and System properties **(Elnashai 2008)**.

2.5.4 Strength

Strength defines the capacity of a member or an assembly of members to resist actions. This capacity is related to a limit state expressed by the stakeholder. It is therefore not a single number and varies as a function of the use of the structure. Usually, strength defines in terms of the type of applied action, that is axial, bending and shear resistances are utilized to quantify the capacity of structures and their components in earthquake structural engineering **(Elnashai 2008)**. The definition of strength parameters is often more straightforward than that of stiffness.

Relationships between geometry, mechanical properties and strength can be derived from principles of engineering mechanics depending on the type of construction materials employed. Similarity to the stiffness, some factors can affect the strength: Material properties; Section properties; Member properties; Connection properties and system properties.

2.5.5 Hysteretic energy

Most structures are designed to behave inelastically under strong earthquakes for reasons of economy. The response amplitudes of earthquake - induced vibrations are dependent on the level of energy dissipation of structures, which is a function of their ability to absorb and dissipate energy by ductile deformations.

A most important property of steels subjected to large cyclic inelastic loading is their ability to dissipate hysteretic energy **(Bruneau 2011)**. Hysteric energy is the energy dissipated by inelastic cyclic deformations and is given by the area within the load-deformation curve also called the hysteretic curve. In structures having low hysteretic energy dissipation capacities, even if the deformations are well below the ultimate deformation, the structure is likely to collapse due to low-cycle fatigue effect as the degradation of strength and stiffness under repeated inelastic cycling. Ensuring that the structure is able to dissipate a large amount of hysteretic energy in each cycle can minimise low-cycle fatigue effect. The area enclosed by the force-deformation loops gives the hysteretic energy. Larger area implies more dissipation of hysteretic energy as shown in Figure 2.2(c).

One way of ensuring good ductility and energy dissipation capacity in steel structures is to use thicker sections thereby avoiding early local buckling. Thus, plastic and compact sections are preferred over semi-compact and slender sections. Other parameters, which control ductility, are slenderness ratio and axial load ratio of the members.

This hysteretic energy absorption is often replaced by equivalent damping in analytical formulations. Ductility is directly related to energy dissipation; high ductility is required to dissipate large amounts of seismic energy **(Elshanai 2008)**.

The difference between dissipative and non-dissipative behaviours is dictated by both the ductility and energy dissipation capacity that the structure can provide **(Landolfo 2017)**.

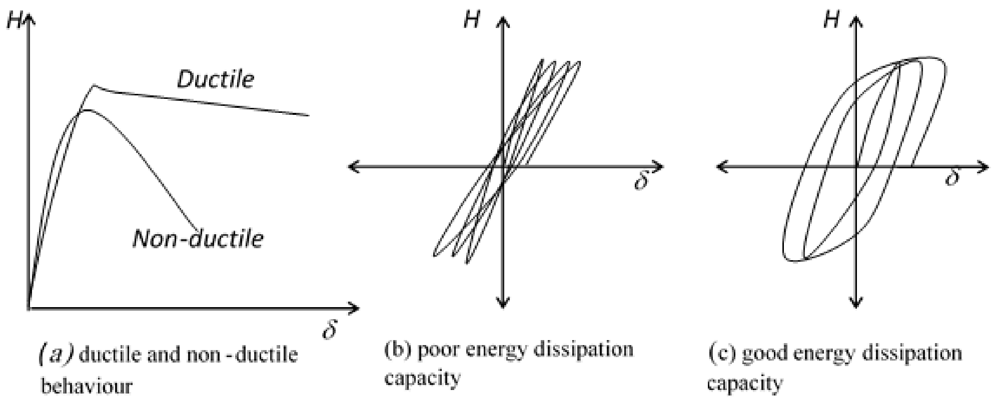


Figure 2. 2. Earthquake resistant properties of structures**.**

2.5.6 Overstrength

The two most important 'implications' of stiffness, strength and ductility are overstrength and damping. As defined in **(Elnashai 2008)**, the overstrength is a parameter used to guaranty the difference between the required and the actual strength of a material, a component or a structural system. Structural overstrength is generally expressed by the 'overstrength factor' Ω_d defined as follows:

$$\Omega_d = V_y/V_d \qquad (2.1)$$

where V_y and V_d are the actual and the design lateral strengths of the system, respectively. The Ω_d factor is often termed observed overstrength' factor.

Overstrength is mobilized by large deformations within the inelastic range.

2.5.7 Damping

When materials are cyclically stressed, energy is dissipated within the material itself due to primarily to internal friction caused by the slipping and sliding of particles at internal planes during deformations. Damping is utilized to characterize the ability of structures to dissipate energy during dynamic response. Unlike the mass and stiffness of a structure, damping does not relate to a unique physical process but rather to a number of possible processes. Damping values depend on several factors; among these are vibration amplitude, material of construction, fundamental periods of vibration, mode shapes and structural configurations **(Bachmann 2012)**.

Damping in a structural system is usually expressed as a percentage of critical damping. Critical damping is that which is required to bring the system to rest in one-half cycle **(Aghayere 2009)**. Typical damping values are given by the seismic codes.

Seismic energy transmitted to structures can be dissipated through different damping mechanisms. Primary sources of damping are, however, as follows:

(i) Structural damping: due to energy dissipation in materials of construction, structural components and their connections;
(ii) Supplemental damping: due to energy dissipation of devices added to structural systems to increase their damping;
(iii) Foundation damping: due to the transfer of energy from the vibrating structure to the soil, through the foundations;
(iv) Radiation damping: due to radiation of seismic waves away from foundations.

2.5.8 R Factor design procedure

The minimum required level of strength is strictly dependent on the required energy dissipation capacity and, for buildings; it is commonly expressed by a coefficient, namely q-factor in Eurocode 8 or reduction factor R in the American codes UBC and AISC, which reduces the elastic design spectrum. Different values of the reduction factor are given by the seismic codes for each structural typology (RPA 99, EC8, UBC, AISC).

Seismic specification uses a response modification factor, R, to compute the design seismic forces in different parts of the structure. The origin of the R factor design procedure can be traced back to the **ATC 3-06 (1984)** for building design. Since requirements in seismic provisions for member design are directly related to the R factor, it is worthwhile to examine the physical meaning of the R factor.

To illustrate the concept of R-factor, consider a structural response envelope shown in Figure 2.3. If the structure is designed to respond elastically during a major earthquake, the required elastic force would be high. For economic reasons, modern seismic design codes usually take advantage of the inherent energy dissipation capacity of the structure by specifying a design seismic force level, which can be significantly lower than Q.

$$Q_s = \frac{Q_e}{R} \tag{2.2}$$

The energy dissipation (or ductility) capacity is achieved by specifying stringent detailing requirements for structural components that are expected to yield during a major earthquake. The design seismic force level is the first significant yield level of the structure, which corresponds to the level beyond which the structural response starts to deviate significantly from the elastic response. Idealizing the actual response envelope by a linearly elastic–perfectly plastic response shown in Figure 2.3, it can be shown that the R factor is composed of two contributing factors:

$$R = R_{\mu}\Omega \tag{2.3}$$

The ductility reduction factor, $R\mu$, accounts for the reduction of the seismic force level from Q_e to Q_y. Such a force reduction is possible because ductility, which is measured by the ductility factor μ (Δ_u/Δ_y), is built into the structural system. For single-degree-of-freedom systems, relationships between R_{μ} and R and have been proposed **(Newmark 1982)**.

The structural overstrength factor, **Ω**, in Equation (2.2) accounts for the reserve strength between the seismic resistance levels Q_y and Q_s. This reserve strength is contributed mainly by the redundancy of the structure. That is, once the first plastic hinge is formed at the force level the redundancy of the structure would allow more plastic hinges to form in other designated locations before the ultimate strength, Q_y, is reached. Although modern seismic codes for building and bridge designs both use the R factor design procedure, there is one major difference. For building design, the R factor is applied at the system level **(Newmark 1982)**. That is, components designated to yield during a major earthquake share the same R value, and other components are proportioned by the capacity design procedure to ensure that these components remain in the elastic range.

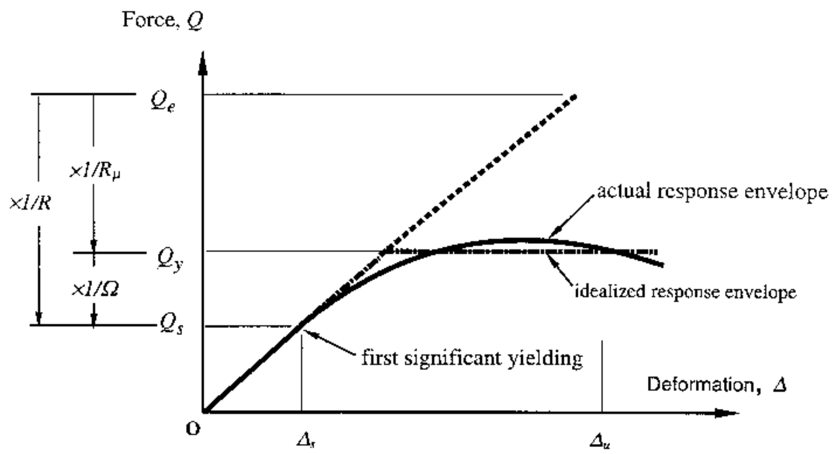


Figure 2. 3. Concept of Response Modification Factor, R.

Furthermore, the AASHTO assumes that cyclic inelastic action would occur only in the substructure; therefore, no R value is assigned to the superstructure and its components. The table shows that the R value ranges from 3 to 5 for steel substructures. A multiple-column bent with well-detailed columns has the highest value of R due to its ductility capacity and redundancy. The ductility capacity of a steel member is generally governed by instability.

2.6 HISTORICAL ADVANCES IN SEISMIC DESIGN METHODOLOGIES

2.6.1 Code's criterion

The most desirable type design code criteria specification is one that puts the least restrictions on the initiative, imagination, and innovation of the designer. Such code might involve only criteria for: (1) the loading or other seismic effects, (2) the level of response, say stress or deformation, or the performance of the structure under the specified loading or other effects. Such an approach need not, and preferably should not, indicate how the designer is to reach his objective, provided he can demonstrate, through documentation of adequacy or through demonstration of adequacy from past performance that the building has achieved a structural capability to resist the specified conditions; and, even owner within the limits of their powers and knowledge, must be convinced that the design is adequate **(Newmark 1982)**.

Design concepts before Northridge and Kobe of the last century, when the dissipation of seismic energy through plastic deformations was first considered by Housner. The utility of the response spectrum lies in the fact that it gives a simple and direct indication of the overall displacement and acceleration demands of the earthquake ground motion, for structures having different period and damping characteristics, without needing to perform detailed numerical analysis.

2.6.2 Modern seismic codes

As reported in (**Gioncu 2011**), the start of modern seismic design may be fixed during the 1950s with the apparition of the notion of the dissipation of seismic energy through plastic deformations was first considered by Housner and developed limit design type analysis for ensuring a sufficient energy absorbing capacity to guarantee an adequate safety factor against collapse in case of extremely strong ground motions. Followed by the study performed by Velestos and Newmark in 1960, on the inelastic response spectrum in which they obtained the maximum response deformation for elastic-perfectly plastic structures. Since then the response spectrum has become a standard measure of the demand of ground motion of the inelastic response in seismic design. A further development has been made by **Newmark (1982)** with the consideration of a new concept proposed, by constructing spectra based on accelerations, velocities, and displacements, in the short, medium and long period ranges, respectively.

Based on these advances, the first practical seismic design philosophy stated the requirements for a minimum level of safety of building. In these codes, the designed structures are expected to fulfil the following recommendations in terms of the earthquake levels:

-To resist minor level of earthquake without significant damage.
-To resist moderate level with some non-structural damage and repairable structural damage.
-To resist major level of earthquake without collapse.

The main objective modern seismic design is only to reduce economic losses, and simultaneity saving human life, not to completely eliminate them.

However, the majority of the seismic design methodologies today explicitly consider only one performance objective that is the case of rare major earthquakes, defined as protection of occupants against injury or loss of life. In the other hand, criteria for checking buildings against minor or moderate earthquakes, which may occur relatively frequently in the life of the building, are not explicitly specified.

In the period 1988-1997, and in spite of the great efforts made in solving satisfactorily the problem of building design in seismic areas, and with the advent of major earthquakes, namely Loma Prieta, Northridge and Kobe which have caused more damage than ever before. So, in the recently, a new philosophy for the seismic design of constructions has been discussed within the engineering community. The main characteristic of the new seismic design is the participation of the owners and users for establishing target and appropriate performance levels. The bases of the new seismic design started in the United States, with the Vision 2000 document **(SEAOC 1995)**, which provides the main concepts of this approach **(Bertero 1996)**. The goal of this design philosophy is to produce structures that have predictable seismic performance under multiple levels of earthquake intensity.

It means that a comprehensive performance-based seismic design involves several steps, that is:

- Selection of performance objectives and definition of the acceptable damage level.
- Definition of multi-level appropriate design criteria and specification of ground motion levels, corresponding to the different design criteria.
- Consideration of a conceptual overall seismic design in function of these levels and option for a suitable structural analysis method for each level.

As reported in **(Elnashai 2008)**, the different approaches to seismic design are represented in the Figure 2.4.

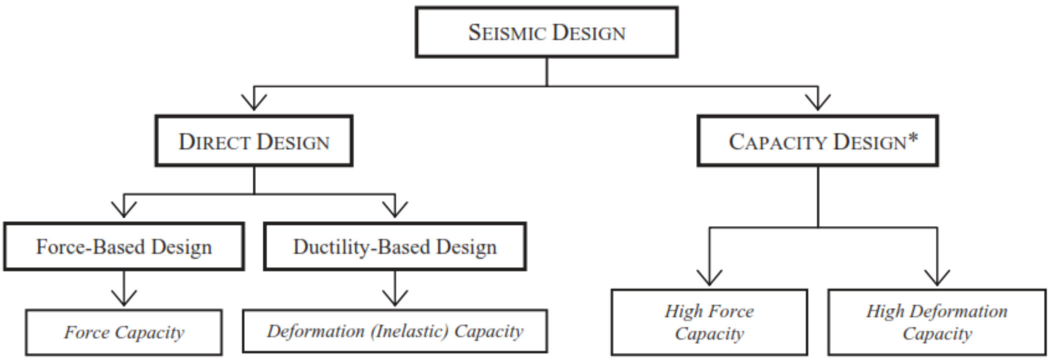


Figure 2. 4. Different approaches to seismic design**.**

In Figure 2.4, traditional force-based seismic design has relied on force capacity to resist the earthquake effects expressed as a set of horizontal actions defined as a proportion of the weight of the structure. Past 20 to 30 years, there has been a tendency to substitute ductility (or inelastic deformation capacity) for strength (or force capacity). The latter approach was developed in recognition of the great uncertainty associated with estimating seismic demand.

A ductility designed structure is significantly less sensitive to unexpected increase in the force demand imposed on it than its strength - designed counterpart.

2.7 EARTHQUAKE-RESISTANT DESIGN METHODS

According to **(Duggal 2013),** conventional civil engineering structures are designed on the basis of two main criteria: strength and stiffness. The strength is related to damageability or ultimate limit state, whereas the stiffness is related to serviceability limit state for which the structural displacements must remain limited. In case of earthquake-resistant design, a new criterion, the *ductility* should also be added. The first two criteria can be achieved by (a) specifying severe (or moderate) design earthquake levels, (b) limiting the maximum stresses or internal forces in critical members, and (c) limiting the storey drift ratio. The third criterion, which is prevention of building collapse, is achieved by providing sufficient strength and ductility to ensure that the structures do not collapse in a service earthquake. Based on the three criteria, rigidity (serviceability), strength (damageability), and ductility (survivability), the methods of seismic design are classified as lateral strength design, displacement- or ductility-based design, capacity design method, and energy-based design. These design approaches are described below **(Duggal 2013)**

- **Lateral strength design:** Lateral strength design is the most common seismic design approach used today. It is based on providing the structure with the minimum lateral strength to resist seismic loads, assuming that the structure will behave adequately in the non-linear range. Concept of response reduction factor is used with an intent to account for both ductility and damping inherent in a structural system.
- **Displacement or ductility-based design:** It is a well-known fact that due to economic reasons, structures are not designed to have sufficient strength to remain elastic in severe earthquakes. The structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive the shock. The ductility-based design operates directly with deformation quantities and, therefore, gives a better insight to the expected performance of structures, rather than simply providing strength, as the lateral strength design approach does.
- **Capacity design method:** The capacity design method is a design approach in which the structures are designed so that hinges can only form in predetermined positions and sequences. It is a procedure of the design process in which strengths and ductilities are allocated and the analyses are interdependent. The capacity design procedure stipulates the margin of strength that is necessary for elements to ensure that their behaviour remains elastic. The capacity design method is so called because in the yielding condition, the strength developed in the weaker member is related to the capacity of the stronger member.
- **Energy-based design:** One of the promising approaches for earthquake-resistant design in the future is the energy-based design approach. In this approach, it is recognized that the total energy input E can be resisted by the sum of the kinetic energy $E\ k$, the elastic strain

energy *Et*, the energy dissipated through plastic deformations (hysteretic damping) *Epes*, and the equivalent viscous damping *E*. The energy equation for a single-mass vibrating system is the energy balance. The energy equation for a single-mass vibrating system is the energy balance between the total input energy and the energies dissipated by viscous damping and inelastic deformations.

2.8 STRUCTURAL SEISMIC DESIGN CONSIDERATIONS

2.8.1 General considerations

The process of earthquake-resistant design requires selection of the earthquake hazard and associated load and deformation effects, as well as an estimation of the structural resisting strengths, as an integral part of the process **(Newmark 1982).** The design of structures under static loads is commonly based upon the limit states criteria. The response of structures to this kind of action, determined as possible values using a probabilistic approach, is analysed with a linear elastic model (only in very special cases a non-linear model is used).

The load effects in the members are then compared with their resistance capacity, thus giving rules for sizing members so that the probability of exceeding various limit states (serviceability or ultimate) is sufficiently low. So, no damage for these static loads is accepted. However, structural design of buildings for seismic loading is primarily concerned with structural safety during major earthquakes, but serviceability and the potential for economic loss are also of concern. Structures are usually designed for gravity loads and checked for earthquake loading. In conformity with the design philosophy, this check consists of two steps: the first ensures elastic response under moderate earthquakes and the second ensures that collapse is precluded under a severe earthquake.

Due to the uncertainties associated in predicting the inelastic response, the second check may be dispensed with, by providing adequate ductility and energy dissipation capacity. In this section, the various methods of performing these checks are described.

2.8.2 Design philosophy

For building-type structures, seismic design procedures, such as included in the building codes usually prevail and are enforceable under the applicable jurisdictional authority. The seismic provisions of standard building codes generally focused around the philosophy expressed in the following sub-sections.

The most important conceptual aspect for avoiding the structural collapse during earthquake is to design the structure in such a way to allow the possibility of redistributing the internal actions when some critical members are damaged. It is worth to recall that the seismic analysis and design of buildings has traditionally focused on reducing the risk of loss of life in the largest expected earthquake. Also, it must be mentioned that severe earthquakes have an extremely low probability of occurrence during the life of a structure.

If a structure has to resist such major earthquakes elastically, it would require an expensive lateral load resisting system, which is economically unjustified. On the other hand, if the structure loses its aesthetics or functionality quite often due to minor tremors and needs repairs, it will be considered as a very unfavourable design. Thus, it is well worth the risk to let them get damaged beyond repair in case the severe earthquake occurs, the chances of which are low. Therefore, a dual strategy, akin to the limit state design, is adopted.

The frame on the far right in Figure 2.5 develops a mechanism through the formation of plastic hinges at the tops and bottoms of the columns in a single storey. This permits free lateral displacement of the structure through rigid body rotation of the columns in that particular storey, without participation of the other columns. This condition is known as a single-storey mechanism. Like the weak storey configuration, it is undesirable because it concentrates deformation and damage in a relatively small region of the structure and can result in large inter-storey drifts in a single storey and possible P-delta instability.

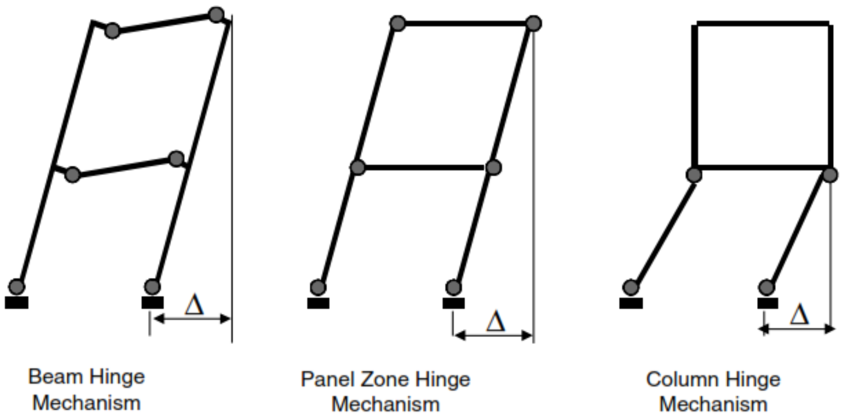


Figure 2. 5. Various lateral frame mechanisms.

2.8.3 Concept of ductility in structural engineering design

2.8.3.1 General

To meet code's requirements, a structure should have sufficient strength, stiffness and energy dissipation capacity, and these requirements basically yielded to the popular use of moment resisting steel framed structures and concentrically braced steel framed structures in construction industry. It is generally acknowledged that structural safety depends on the ductility that the structural system can provide against the design loads. In fact, the ductility can be defined as a representation of the capacity of a mechanical system (e.g. a beam, a structure, etc.) to deform in the plastic domain without substantially reducing its bearing capacity **(Landolfo 2017)**.

In addition to what has been already said in paragraph 2.5.2, ductility is one of the most important design criteria of a structural system. Hence, a ductile structure may take damage during an earthquake but will not collapse if it is in the required strength limits. In the context of structural engineering, a “ductile” material is one that is capable of undergoing large inelastic deformations without losing its strength.

In **Bruneau (2011)**, more formally, (ASM 1964) defines “ductility” as “the ability of a material to deform plastically without fracture”. “Brittleness”, on the other hand, is the “quality of a material that leads to crack propagation without plastic deformations.” In that perspective, structural steel is the most ductile of the widely used engineering materials **(Bruneau 2011)**.

2.8.3.2 Ductility and earthquake motion

Ductility is an essential attribute of a structure that must respond to strong ground motions and serves as the shock absorber in a building, for it reduces the transmitted force to one that is sustainable. Therefore, the survivability of a structure under strong seismic actions relies on the capacity to deform beyond the elastic range, and to dissipate seismic energy through plastic deformations. So, the ductility check is related to the control of whether the structure is able to dissipate the given quantity of seismic energy considered in structural analysis or not.

2.8.3.3 Classes of ductility to EC8

Three classes of ductility are recognised by EC8 in steel and concrete structures, namely DCH, DCM and DCL. Only one ductility class may be used for the design of a particular building. DCM (ductility class medium) and DCH (ductility class high) structures are designed as dissipative. It may be observed that DCH structures generally are designed for lower lateral resistance than DCM, but require more stringent analysis and detailing. DCL (ductility class low) structures (also secondary elements – see Section 5.2 EC8 (2004)) are not designed to be dissipative and can be detailed to non-seismic rules **(Bisch 2010)** (For the rules governing steel DCL structures). DCL structures are only permitted in areas of low seismicity **(Bisch 2010).**

2.8.3.4 Ductility and energy dissipation

The ductility is the property, which allows the structure to undergo large plastic deformations without significant loss of strength. Ductility μ is defined as the ratio of the ultimate deformation δ_u at an assumed collapse point, to the yield deformation δ_y. It may be noted that the collapse point may be assumed to lie on the descending branch of the load-deformation curve. This is still safe because earthquake loading is transient and will cease to act after a short time and so the structure will not be toppled.

Ductility is also important in seismically designed steel frames to absorb and dissipate energy from cyclic loadings. For cyclic behaviour, the ductility may be expressed as an accumulation of plastic deformations.

Two substantially different concepts can be used to design structures located in seismic areas, which correspond to two different structural behaviours (**Landolfo 2017)**:

- Concept (a): low-dissipative (and/or non-dissipative) behaviour;

− Concept (b): dissipative behaviour.

The difference between dissipative and non-dissipative behaviours is dictated by both the ductility and energy dissipation capacity that the structure can provide. The ductility represents the capacity to deform in the plastic domain without substantially reducing its bearing capacity. However, there are other properties that significantly influence the seismic performance, namely the displacement and dissipative capacity. These properties are not synonyms, but all of them contribute towards a satisfactory seismic behaviour. Some examples may be helpful to clarify the differences between ductility, displacement and dissipative capacity.

2.8.3.5 Evaluation of ductility

Two major factors may affect evaluation of the ductility of structural systems. First, the ductility is often measured by the hysteretic behaviour of the critical components. First, the ductility is often measured by the hysteretic behaviour of the critical components. The hysteretic behaviour is usually examined by observing the cyclic force-deflection (or moment-rotation) behaviour. The slope of the curves represents the stiffness of the structure or component. The enclosed areas represent the energy that is dissipated, and this can be large, because of the repeated cycles of vibration of the repeated cycles of vibration.

The hysteretic curves also show the inelastic deformation that can be tolerated at various resistance levels. Structural framing with curves enclosing a large area representing large dissipated energy, and structural framing which can tolerate large inelastic deformations without excessive loss in resistance, are regarded as superior systems for resisting seismic loading. As a result, these systems are commonly designed with larger *R* values and smaller seismic loads **(Brockenbrough 2006)**.

2.8.3.6 Ductility and dissipative zones

A dissipative structure is one which is able to dissipate energy by means of ductile hysteretic behaviour and/or by other mechanisms. Dissipative zones (also called critical regions) are predetermined parts of a dissipative structure where the dissipative capabilities are mainly located. Other parts of the structure, known as non-dissipative, are protected against yielding by means of capacity design procedures.

2.9 CODE PREVISIONS

2.9.1 Introduction

Building codes have based their provisions on the historic performance of buildings and their deficiency and have developed provisions around life safety concerns, i.e., to prevent collapse under the most intense earthquake expected at a site during the life of a structure. Safety is of paramount importance in any structure, and requires that the possibility of collapse of the structure (partial or total) is acceptably low not only under normal expected loads (service loads), but also under less frequent loads (such as due to earthquakes or extreme winds) and accidental loads (blasts, impacts, etc.). Collapse due to various possibilities such as exposure to a load exceeding the load-bearing capacity, overturning, sliding, buckling, fatigue fracture, etc. should be prevented. Another aspect related to safety is structural integrity and stability-the structure as a whole should be stable under all conditions. (Even if a portion of it is affected or collapses, the remaining parts should be able to redistribute the loads.) In other words, progressive failure should be minimized **(Subramanian 2011).**

2.9.2 Code's criterion

Code's provisions are based on the concept that the successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility manifested in the details of construction, and the presence of a fully interconnected, balanced, and complete lateral-force-resisting system. In regions of low seismicity, the need for ductility reduces substantially. In fact, in some instances, strength may even substitute for a lack of ductility. Very brittle lateral-force-resisting systems can be excellent performers as long as they are never pushed beyond their elastic strength.

Most seismic codes specify criteria for the design and construction of new structures subjected to earthquake ground motions with three goals:

1) Minimize the hazard to life for all structures;
2) Increase the expected performance of structures having a substantial public hazard due to occupancy or use; and
3) Improve the capability of essential facilities to function after an earthquake.

Some structural damage can be expected as a result of design ground motion because the codes allow inelastic energy dissipation in the structural system. For ground motions in excess of the design levels, the intent of the codes is for structures to have a low likelihood of collapse.

In most structures that are subjected to moderate-to-strong earthquakes, economical earthquake resistance is achieved by allowing yielding to take place in some structural members. It is generally impractical as well as uneconomical to design a structure to without collapse, but must also remain operational after an earthquake. Therefore, in addition to life safety, damage control is an important design consideration for structures deemed vital to post-earthquake functions.

In general, most earthquake code provisions implicitly require that structures be able to resist:

1. Minor earthquakes without any damage.
2. Moderate earthquakes with negligible structural damage and some non-structural damage.
3. Major earthquakes with some structural and non-structural damage but without collapse.

The structure is expected to undergo fairly large deformations by yielding in some structural members respond in the elastic range to maximum expected earthquake-induced inertia forces. Therefore, in seismic design, yielding is permitted in predetermined structural members or locations, with the provision that the vertical load-carrying capacity of the structure is maintained even after strong earthquakes. However, for certain types of structures such as nuclear facilities, yielding cannot be tolerated and as such, the design needs to be elastic. Structures that contain facilities critical to post-earthquake operations—such as hospitals, fire stations, power plants, and communication centres—must not only survive.

2.9.3 The behaviour factor R

Since seismic loading is an inertial loading, the forces are dependent on the dynamic characteristics of the acceleration record and the structure. The behaviour factor R (q to EC8) is a structure-dependent parameter used to reduce seismic design forces (but not design seismic deflections) below those corresponding to elastic response. These forces are usually reduced in accordance with the ductility of the structure. This reduction is accomplished by the R factor in the static-force method, and the reduction may be quite large (Art. 8.5 RPA99 version 2003).

R factor is a function of the ductility of the structure (i.e. its ability to sustain repeated deformations into the inelastic range without significant degradation of stiffness and strength) and the ratio of ultimate lateral strength to lateral strength at effective yield. The design value of the behaviour factor R(q) depends on the structural material, form of lateral resisting structure; ductility class. A single value of q must be used for the design of the lateral load resisting system in each principal direction of a building, but this need not be the same as the q value used for the other principal direction. The designer must ensure that the structure is capable of developing the required ductility, as it is well-known that the available ductility varies with different structural systems.

2.9.4 Effects of inelastic deformations.

The distribution of inelastic deformation is a second factor that can affect the inelastic seismic performance of a structural system. Some structural systems concentrate the inelastic deformation (ductility demand) into a small portion of the structure. This can dramatically increase the ductility demand for that portion of the structure. This concentration of damage is sometimes related to factors that cause pinched hysteretic behaviour, since buckling may change the stiffness distribution as well as affect the energy dissipation. Ductility demand, however, can also be related to other factors.

The difference in design concept results in a significant difference in seismic response and ductility demand. Design codes attempt to assure greater ductility from structures designed for smaller seismic forces, but attaining this objective is complicated by the fact that ductility and ductility demand are not fully understood **(Brockenbrough 2006).**

2.10 CAPACITY DESIGN PRINCIPLES

2.10.1 Introduction

Capacity design was developed in the late 1960s in New Zealand as an approach to resist the effects of severe earthquakes. In capacity design, acknowledging that inelastic action is unavoidable during severe earthquakes, the designer dictates where inelastic response should occur. Such zones of possible inelastic action are selected to be regions where large plastic deformations can develop without significant loss of strength; these regions are detailed to suppress premature undesirable failure modes, such as local buckling or member instability in the case of steel structures **(Bruneau 2011; Paulay 1992)**.

2.10.2 Definitions of the capacity design

According to **Gioncu (2011),** the capacity design method is a design method in which the elements of the structural system are chosen and suitably designed and detailed for energy dissipation under severe earthquakes, while all other elements are provided with sufficient strength so that the chosen energy dissipation can be maintained. Another definition of the capacity design approach can be stated as follows: is an approach whereby the designer establishes which elements will yield (and need to be ductile) and those which will not yield (and will be designed with sufficient strength) based on the forces imposed by yielding elements.

The reason to name the capacity design is that, in the yielding condition, the strength developed in weaker member is related to the capacity of the stronger member. In other words, the capacity design is based on deterministic allocation of strength and ductility in the structural elements for successful response and collapse prevention during a catastrophic earthquake by rationally choosing the successive regions of energy dissipation so that pre-decided energy dissipation mechanism would hold throughout the seismic action.

2.10.3 Basic concepts

The basic concept of capacity design of structures is the spreading of inelastic deformation demands throughout the structures in such a way that the formation of plastic hinges takes place at predetermined positions and sequences. In other words, the capacity design is based on deterministic allocation of strength and ductility in the structural elements for successful response and collapse prevention during a catastrophic earthquake by rationally choosing the successive regions of energy dissipation so that predicted energy dissipation mechanism would hold throughout the seismic action.

The opposite of 'capacity design' is 'direct design', as described in **Elnashai (2008),** which is the dimensioning of individual components to resist the locally evaluated actions with no due consideration to the action -redistribution effects in the system as a whole. Direct design can be either ductility-based or strength-based. Capacity design is based on both strength and ductility of components.

In EC8, the capacity design is defined as: Design method in which elements of the structural system are chosen and suitably designed and detailed for energy dissipation under severe deformations while all other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

2.10.4 Capacity design levels

Capacity design has two major implications, one at the member level, and the other at the system level **(Sucuog 2014)**:

Member level: Flexural failure mode is ensured by suppressing shear failure (capacity shear principle in beams, columns, walls and connections).

System level: The spreading of plastic regions that undergo flexural yielding follows a hierarchy for obtaining a more ductile system response (strong column–weak beam principle at the connections).

2.10.5 Advantages of this strategy include

Some advantage of the use of capacity design in seismic regions **(Gioncu 2011):**

- Protection from sudden failures in elements that cannot be proportioned or detailed for ductile response.
- Limiting the locations in the structure where expensive ductile detailing is required
- Greater certainty in how the building will perform under strong earthquakes and greater confidence in how the performance can be calculated.
- Reliable energy dissipation by enforcing deformation modes (plastic mechanisms) where inelastic deformations are distributed to ductile components.

2.10.6 Examples of capacity design strategy:

- **SCWB requirement**

It is a design approach in which the structures are designed in such a way that hinges can only form in predetermined positions and sequences. It is a procedure of the design process in which strengths and ductility are allocated and the analyses are interdependent.

The well-known “strong column/weak beam” requirement is an example of a capacity design strategy, where the intent is to avoid inelastic hinging in columns that could lead to premature storey mechanisms and rapid strength degradation in columns with high axial loads. In order to decrease the probability of plastic hinge formation in columns, MRF's must be designed to have strong columns and weak beams (SCWB), not clearly adopted by RPA99. To this scope, different simplified design criteria have been proposed in earlier works **(Akiyama1985; Rosenblueth 1980; Paulay 1992; Mazzolani 2012)** and the so- called beam-column hierarchy criterion has been introduced in Eurocode8 However, according to the capacity design philosophy, it is important to know the maximum yield stress of the dissipative parts.

- **Yielding links in EBF**

The design of yielding links and elastic braces in eccentrically braced frames is another example of capacity design. Where inelastic analysis is used, capacity design can be implemented by modelling the specified yielding elements with their "expected" strengths and the protected elements as elastic. This permits the determination of and design for the maximum expected force demands in the protected elements (**Deierlein 2010).**

- **Dissipative structures**

A dissipative structure is able to dissipate energy by means of ductile hysteretic behaviour and/or by other mechanisms at the dissipative zones, which are the predetermined parts of a dissipative structure where the dissipative capabilities are mainly located. These are also called critical regions. The basic principles of Capacity Design state that dissipative zones have to be designed according to the internal actions arising from the load combinations provided by seismic codes, whereas non-dissipative zones have to be proportioned on the basis of the maximum internal actions transmitted by dissipative zones in the fully yielded and strain-hardened state (**Park 1986**; **Mazzolani 2012; Bruneau 2011).**

The condition needs to be fulfilled also in the case of eccentrically braced frames (EBFs) where dissipative zones are constituted by the so-called link elements, whose yielding is significantly affected by moment-shear interaction **(Kasai 1997; Roeder 1978; Hjelmstad 1983a).**

As mentioned in **(Mazzolani 2011),** seismic resistant dissipative structures are usually designed to withstand severe earthquakes by asking for a proper combination of strength and energy dissipation capacity. This design goal is pursued in current seismic codes by means of provisions aiming at assuring a minimum level of strength and by means of design and detail rules for obtaining the required energy dissipation capacity.

2.10.7 Collapse mechanism

Collapse mechanism control is universally recognized as one of the primary goals of the structural design process. The aim is to avoid partial collapse mechanisms, such as soft storey mechanisms, which are unsatisfactory in terms of energy dissipation capacity. The optimization of the seismic structural response is, conversely, obtained when a collapse mechanism of global type is developed, because, in such case, all the dissipative zones are involved in the corresponding pattern of yielding, leaving all the other structural parts in elastic range.

In order to maximise the energy dissipation capacity, plastic hinges have to develop in beams and at the column base. The corresponding failure mode is called "global collapse mechanism". Moreover, moment frames are also preferred for their architectural versatility:

there are no bracing elements which block wall openings and the maximum flexibility for space utilisation is provided. This advantage is companied by a poor lateral stiffness of the whole structure, so that the member sizes are larger than those required for strength, due to the necessity to contain sway deflections within the drift limits, imposed by the codes **(Mazzolani 2012; Bruneau 2011)**.

2.11 CAPACITY DESIGN METHOD

The principle known as "capacity design" which can be explained by considering the chain model, introduced by **Paulay (1992)** and depicted in Figures 2.6 for Ductility of a chain with brittle and ductile rings (a), (b) and (c) respectively. Figures 2.7 and 2.8 for brittle and ductile overall response of a chain with brittle and ductile rings respectively. The chain represents a structural system made of both ductile elements (e.g. the ring “1”) and brittle zones (e.g. the ring “i”). In this chain, one link is designed to absorb a large amount of plastic energy in a stable manner prior to failure (e.g., ring 1). Therefore, the other links (e.g., 1, 2, 3, 5, 6, and 7) can be designed without concern for plastic deformations, provided their capacities exceed the maximum capacity of the plastic link, thus avoiding the need for special detailing in all but one link.

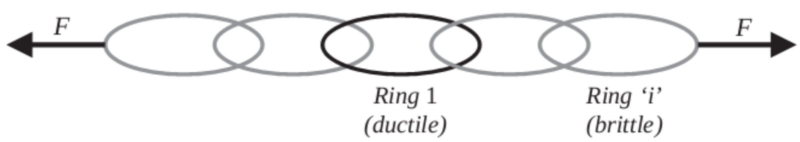
According to direct design procedures with non- seismic effect and for quasi-static loads, the design force is the same for all elements belonging to the chain, because the applied force is equal for all rings, being a system in series. Hence, the design resistance $F_{y,I}$ is the same for all elements. Under this assumption, the yield resistance of the ductile chain $F_{y,1}$ is equal or even slightly larger than $F_{y,i}$.

As illustrated in Figure 2.6(b), with the direct design approach the system cannot develop strength larger than F_y and the ultimate elongation of the chain is given as:

$$\delta_u = \sum_i \delta_y = 5\delta_y \tag{2.2}$$

Additionally, and according to capacity design principles, in order to improve the ductility of the chain, some rings should be designed with ductile behaviour and lower strength, as is the case of ring “1” as shown in Figure 2.6(c).

The remaining rings “i” that are brittle should be designed to provide a resistance $F_{y,i}$ larger than the maximum resistance F_u,1exhibited by the ring “1” beyond yielding. The ductile ring “1” behaves as a sacrificial element, i.e. a ductile fuse, which filters the external actions and limits the transfer of forces into the brittle elements. Hence, the maximum force that the chain can sustain is equal to the maximum resistance $F_{u,1}$ of the ductile ring “1”.



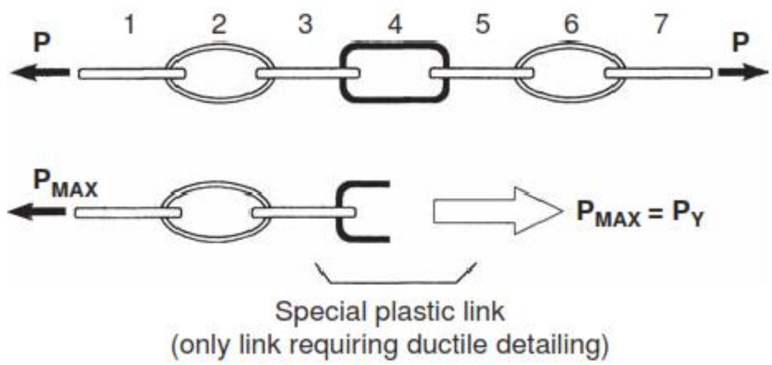


Figure 2. 6. Ductility of a chain with brittle and ductile rings (a), (b) and (c) respectively

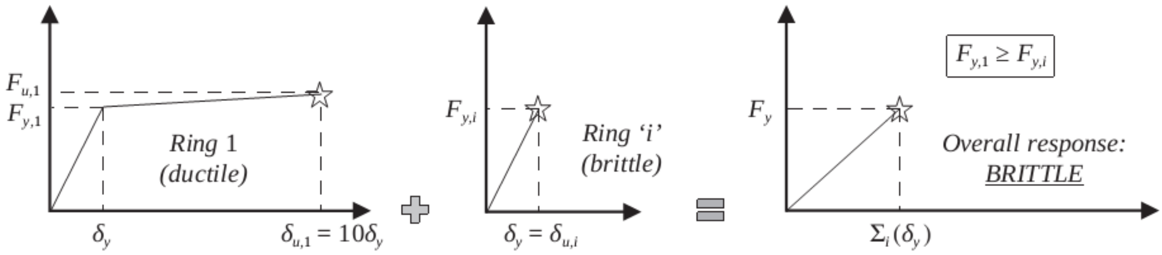


Figure 2. 7. Brittle overall response of a chain with brittle and ductile rings**.**

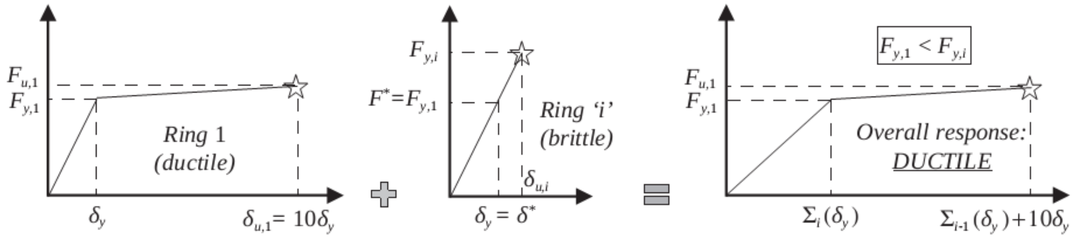


Figure 2. 8. (a) Ductile overall response of a chain with brittle and ductile rings **(Paulay 1992).**

In case of steel structures the best way to dissipate energy is to exploit the tensile capacity of the material, which can be obtained by enforcing plasticity into specific zones called plastic hinges that can involve either flexural, tensile or shear mechanisms depending on the type of adopted structural scheme (e.g. moment resisting frame, concentrically or eccentrically braced frame), while preserving the rest of the structure from damage.

It is generally acknowledged that steel structural safety depends on the ductility that the structural system can provide against the design loads. Indeed, ductility represents the capacity of a mechanical system (e.g. a beam, a structure, etc.) to deform in the plastic domain without substantially reducing its bearing capacity. However, in seismic design of structures, it is generally not economical or simply not possible to ensure that all the elements of the structure behave in a ductile manner.

Indeed, a dissipative or ductile structure includes both dissipative (ductile) elements and non-dissipative (brittle) ones. In order to achieve a dissipative (ductile) design for the whole structure, the failure of the brittle elements must be prevented. This may be done by prioritizing structural elements strength, which will lead to the prior yielding of ductile structural elements, preventing the failure of brittle structural elements.

2.12 DESIGN ASSISTED BY TESTING

Design assisted by testing is a powerful tool for evaluating the performance characteristics of materials, components or kits. In Europe, the principles and application rules are given in EN 1990 **EC1)**, with additional specific requirements in the different parts of the Eurocodes. Design assisted by testing was initially developed for Eurocode 3 (Annex Z of ENV 1993-1-1 **EC8 1998** but it is now standardized in Annex D of EN 1990 and it is applicable to all kinds of materials and ways of construction. According to EN 1990, all design rules are based on test evaluations using an appropriate test evaluation method.

The type of test depends on the relevant properties to be measured and the conditions of loading or load effect models **(Dubina 2008)**. The evaluation of the seismic response of structures may enter in this category, as the structure and implicitly the materials can suffer partial or complete damage (breaks, cracks, large deformations, local or global instability).

As a result, seismic codes, e.g. EC8 (2005) and AISC 341-16 (2016), require that whenever provisions do not adequately cover the calculation of resistance or other parameters, appropriate experimental studies should be performed.

Under seismic actions, ductile components are expected to experience large inelastic cyclic deformation and high strain rates, which can reduce their available ductility and may induce failure by low cycle fatigue.

2.13 MODERN CODE SEISMIC DESIGN CONCEPTS

In many ways structural steel is an ideal material for the design of earthquake-resistant structures. It is strong, light weight, ductile, and tough, capable of dissipating extensive energy through yielding when stressed into the inelastic range. Given the seismic design philosophy of present building codes, which is to rely on the inherent ability of structures to undergo inelastic deformation without failure, these are exactly the properties desired for seismic resistance. In fact, other construction materials rely on these basic properties of steel to assist them in attaining adequate seismic resistance.

2.13.1 Introduction

For many types of structures, the seismic action represents a very severe design action, more critical than the other loading conditions, and providing an elastic response of the structure under the effect of the design seismic action at ULS will lead to excessive size of the structural elements and, consequently, to an excessive material consumption. Global and local brittle failure or the premature formation of unstable plastic mechanisms should be avoided should be avoided.

Design and detailing requirements for these various structural systems are governed by the building code and building standards law requirements specific to each country.

The seismic analysis and design procedures to be used in the design of building structures and their components will be described in this section. The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand.

2.13.2 General consideration and bases of seismic design

In seismic design, knowledge of the maximum probable yield strength is as important as the knowledge of the reliable yield design. Ductility is one of the most important design criteria of a structural system. A ductile structure may take damage during an earthquake but will not collapse if it is in the required strength limits. Two noticeably different concepts can be used to design structures situated in seismic areas, which correspond to two different structural behaviours:

− Concept (a): low-dissipative (and/or non-dissipative) behaviour;
− Concept (b): dissipative behaviour.

Fundamentally, any structure can be designed according to one of the two concepts. Designing a structure as dissipative or low-dissipative is a decision of the structural engineer.

The difference between dissipative and non-dissipative behaviours is dictated by both the ductility and energy dissipation capacity that the structure can provide. It is worth to recall that the ductility represents the capacity to deform in the plastic domain without substantially reducing its bearing capacity. The ductility of steel generally reduces with an increase of the yield stress. However, there are other properties that significantly influence the seismic performance, namely the displacement and dissipative capacity. These properties are not synonyms, but all of them contribute towards a satisfactory seismic behaviour. Some examples may be helpful to clarify the differences between ductility, displacement and dissipative capacity.

Figure 2.9 illustrates the load-deflection response curves of two different frames subjected to monotonically increasing horizontal loads.

Normally, choosing the seismic design concept accounts for economic aspects, depending on the type of the structure and the location of seismic area. With this regard, it should be noted that structural details and design demands necessary to provide ductility and dissipative behaviour may lead to higher constructional and design effort.

The maximum strength f_y of the frame corresponds to the yield strength and/or stability limit load, and the deformation capacity δ_u corresponds to the sudden decrease in the strength that can be caused by the rupture of steel material, global and/or local buckling of steel members and/or crushing of concrete. Even though the strength of both frames is identical, the one with the response curve shown in Figure 2.9 represents a ductile behaviour, which is substantially different from that of Figure 2.10 that corresponds to a brittle performance that is the occurrence of soft-storey mechanism.

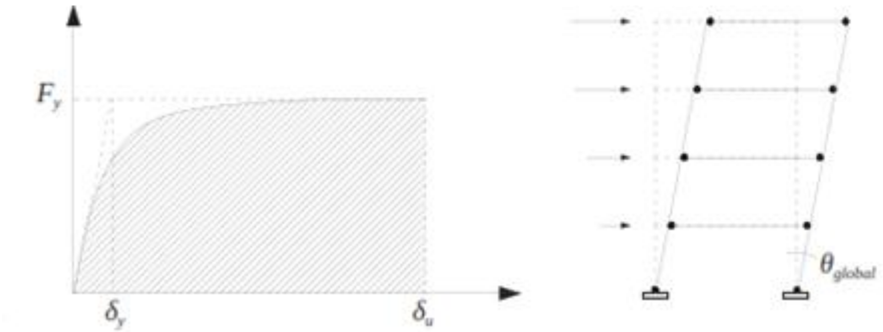


Figure 2. 9. Ductility of frames with high ductility **(Landolfo 2017).**

Actually, the first structure is characterized by a larger ductility $\mu = \delta_u/\delta_y$ and also a larger displacement capacity δ_u which is the capacity of the structural system to experience large ultimate displacements. Also, the amount of energy absorbed by the frame shown in Figure 2.9 before it reaches the limit deformation δ_u is larger than that of the frame shown in Figure 2.10. It is obvious that the response of the frame shown in Figure 2.9 is more efficient for an earthquake resisting structure.

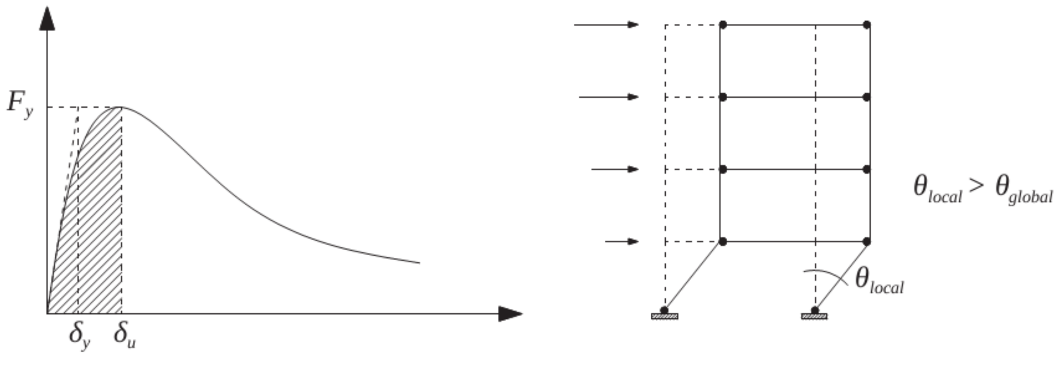
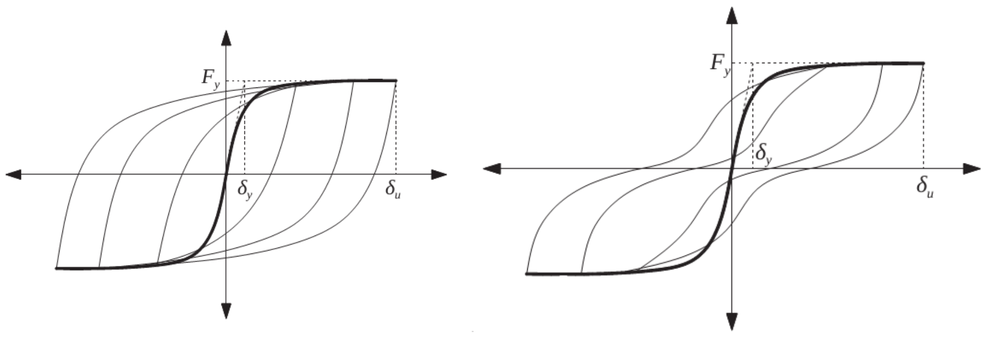


Figure 2. 10. Ductility of frames with poor displacement capacity **(Landolfo 2017).**

This may be the case of structures which may induces a soft storey mechanism with brittle failure and a poor plastic redistribution along the building height. The general design approach recommended by EC8 aims to control the inelastic structural behaviour by avoiding the formation of soft storey mechanisms.

Nevertheless, an adequate seismic behaviour also depends on the shape of the cyclic response of both the structure and the dissipative zones. With this regard, Figure 2.11 shows two examples of hysteresis loops of frames under repeated horizontal load, having the same monotonic response and displacement capacity δ_u. In these cases, in addition to the effects indicated above, the shape of the hysteresis loops also depends on the number of loading cycles, since deformation phenomena associated with fatigue caused by the repeated loading may have some effect on it.



(a) (b)

Figure 2. 11. Dissipative capacity of frames: a) high b) poor energy absorption

The structure shown in Figure 2.11 (a) dissipates larger energy before failure than it is the case for the structure in Figure 2.11(b), thus providing a better seismic performance, the energy being the area within a loop. Hence, dissipative capacity can be defined as the ability to dissipate energy by means of stable and compact hysteretic loops.

Ductile and dissipative structures are very convenient because they avoid brittle phenomena and lead to less expensive constructions. In order to exploit the ductility, ductile structures are generally designed to resist seismic forces substantially smaller than those needed to obtain an elastic response under seismic action corresponding to the Ultimate Limit State (ULS).

However, it is worth to notice that plastic deformations imposed by the seismic action must not exceed the deformation capacity of the structure in the plastic domain, in order to prevent excessive damage that may compromise the stability against gravity loads and/or make unfeasible a subsequent renewal. Thus, the minimum strength F_y of the structure against lateral forces that is needed to avoid excessive damage is directly related to the structure's deformation capacity in the plastic domain. For the ULS seismic action, different strength/ductility combinations can be determined that satisfy the design demands.

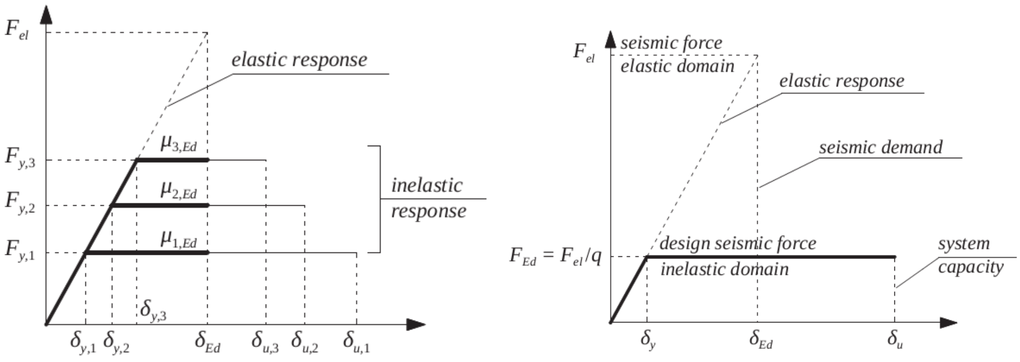


Figure 2. 12. Strength vs. displacement demand relationship

Modern codes like EC8 part 1, RPA99 (with less developed details), give the possibility to choose different ductility levels for a structure, providing different ductility classes. It is understandable that choosing a ductility class instead of another has direct consequences on the design process as shown in Figure 2.12.

In case of EC8 first part, there are at least two major features. The first is the value of the design seismic load, which is obtained by scaling the elastic design forces by a behaviour factor q (R factors for RPA99). The structures that are designed to behave in a more ductile way (i.e. on a higher ductility class) have higher values of the behaviour factor q, and, consequently, lower design seismic forces. The second consequence of choosing a ductility class is the necessity of providing a certain ductility level to the structure. To achieve this purpose, the codes provide specific detailing and design requirements for all structural materials (e.g. steel, reinforced concrete, timber, etc.) and relevant types of structures (e.g. moment resisting and braced frames, structural walls, etc.).

In EBF, see Figure 2.13, the braces are configured to form stiff triangulation with part of the beams, leaving the other part as a link. For example, Figure 2.13 shows some arrangements that create qualified links. In principle, the links are expected to yield under seismic excitation while the rest of the members, including beams and braces, remain elastic. This philosophy is in line with the capacity design philosophy given in previous section with limitations.

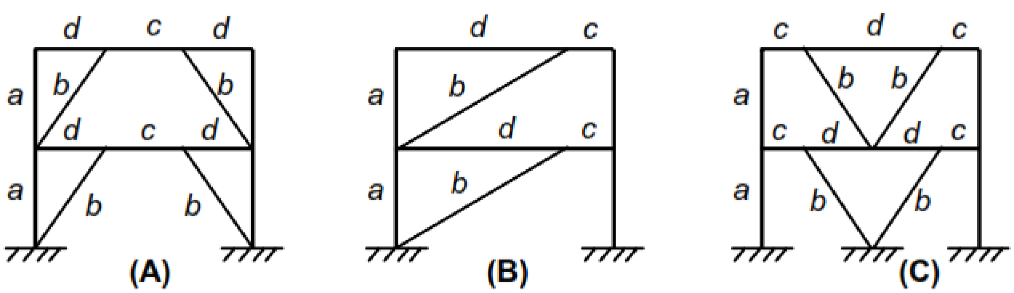


Figure 2. 13. EBF Structures**.**

2.14 ALGERIAN SEISMIC CODE RPA 99 DESIGN PHILOSOPHY

The Algerian Seismic Code (RPA99 version 2003) specifies two basic steps for seismic design of steel structures:

(i) Sizing of structural element to fulfil an inter-storey drift requirement of 1% by an elastic behaviour;

(ii) Checking the sections to fulfil the resistance requirements ‘for critical load combinations where the seismic forces correspond to the strength limit state. The factor used to reduce the elastic seismic forces to obtain the design seismic forces is known as the structural response modification factor. More details will be provided on RPA99 design method when dealing with cases considered, regular and non-regular multi-storeys structures considering linear and nonlinear behaviours.

2.15 SUMMARY OF THE CODES SEISMIC DESIGN OF STRUCTURES STEP BY STEP

As very well-summarised in **(Sucuog 2014)**, the seismic design procedures in earthquake codes mainly include the following steps:

1. Calculate lateral earthquake forces for linear elastic response: lateral earthquake forces are calculated from a linear elastic design: spectrum which represents design ground motion intensity.

2. Reduce linear elastic forces to account for inelastic response using the response reduction factor (R or q) in order to account for the inelastic deformation capacity (ductility) of the system.

3. Apply reduced forces to the structural model, carry out response spectrum analysis and determine the internal seismic design forces acting on structural members as well as inter-storey drifts under the reduced acceleration spectrum (inelastic design spectrum).

4. Combine internal seismic design forces with the internal forces from gravity loads (use load combinations in the relevant design code). The internal seismic design forces determined in Step 3 are combined with the results of gravity analysis in Step 4.

5. Design structural members under these combined design forces. Hence, this is a ‘‘force-based’’ design procedure since the design of structural members is based on internal forces which indirectly account for the inelastic deformation capacity of the conceived structural system.

6. Check the calculated inter-storey drifts with the permitted drift limits. Excessive lateral deformations are not permitted in terms of inter-storey drift ratios. In the case of low seismic forces, inter-storey drift limits may control design.

CHAPTER 3:

METHODS OF STRUCTURAL SEISMIC ANALYSIS

CHAPTER 3: METHODS OF STRUCTURAL SEISMIC ANALYSIS

For structural engineers, earthquake engineering can be broadly divided into three areas, namely, seismology (including ground effects), seismic analysis, and seismic design. The aim of this chapter is to capture the essence of seismic methods of analysis available in very good text books recommended in the subsequent sections, rather than cover in detail the immense volume of literature that exists on the subject. There are many excellent books that cover these three areas in varying proportions, that contain all aspects of the seismic analysis of structures, combining new concepts with existing ones, some of them are being listed in the reference section.

The present chapter attempts to provide the essential of material for the seismic analysis of structures in totality. It offers a comprehensive and unique treatment of all aspects of the seismic analysis of structures. The materials in this chapter are arranged and presented in a manner that is expected to be equally useful to both undergraduate and postgraduate students, researchers, and practicing engineers.

3.1 INTRODUCTION TO STRUCTURAL ANALYSIS

3.1.1 Definition of a structure

A structure may be regarded as a number of components, referred to as elements, connected together to provide for the transmission of forces. A structure can also be defined as a body that resists external effects such as loads, temperature changes, and support settlements, without undue deformation. Building frames, industrial building, bridges, halls, towers, dams, reservoirs, tanks, retaining walls, channels, pavements are typical structures of interest to civil engineers. The forces arise from loads on the structure and the elements are designed to transmit these forces to the foundations.

3.1.2 Definition of structural analysis

Structural analysis is the determination of the response of a structure to external effects such as loading, temperature changes and support settlements, deformations of the structure due to applied loads. Structural design involves the arrangement and proportioning of structures and their components in such a way that the assembled structure is capable of supporting the designed loads within the allowable limit states **(Chen 1999)**.

The determination structural design is the selection of a suitable arrangement of members, and a selection of materials and member sections, to withstand the stress resultants (internal forces) by a specified set of loads, and satisfy the stress and displacement constraints, and other requirements specified by the utilized code of practice (**Kaveh 2014)**.

An analytical model is an idealization of the actual structure. The structural model should relate the actual behaviour to material properties, structural details, and loading and boundary conditions as accurately as is practicable **(Chen 1999)**.

3.1.3 Analysis and design of structures

The design of structures, of which analysis is an integral part, is frequently undertaken using computer software. This can only be done safely and effectively if those undertaking the design fully understand the concepts, principles and assumptions on which the computer software is based. It is vitally important therefore that design engineers develop this knowledge and understanding by studying and using hand methods of analysis based on the same concepts and principles, e.g. equilibrium, energy theorems, elastic, elasto-plastic and plastic behaviour and mathematical modelling (**McKenzie 2013).**

A correct solution of any designed structure should satisfy the following requirements (**Kaveh 2014)**:

1. Equilibrium: The external forces applied to a structure and the internal forces induced in its members should be in equilibrium at each node.
2. Compatibility: The members should deform so that they all fit together.
3. Force-displacement relationship: The internal forces and deformations satisfy the stress–strain relationships of the members.

3.1.4 Structural analysis

The fundamental objective of structural analysis is to determine the response of the structure subjected to a set of loads. As such, this involves consideration of the loads, materials and the geometry and the form of the structure. A structure can be considered as an assemblage of members and nodes. Structures with clearly defined members are known as skeletal structures (**Kaveh 2014)**.

Structural analysis, whether linear or nonlinear, is mostly based on matrix formulations to handle the enormous amount of numerical data and computations. Plastic structural analysis is used to obtain the behaviour of a structure at collapse. As the structure approaches its collapse state when the loads are increasing, the structure becomes increasingly flexible in its stiffness **(Wong 2011).**

3.2 SEISMIC BEHAVIOUR OF STRUCTURAL STEEL

3.2.1 General

Behaviour of steel buildings under earthquakes has generally been satisfactory from the point of view of strength. The properties of steel that contribute to the elastic resistance of steel structures during moderate earthquakes are the yield strength and elastic stiffness. However, in major earthquakes, a structure may undergo inelastic deformations and rely on its ductility and hysteretic energy dissipation capacity to avoid collapse.

The stress–strain relationship for steel, shown in Figure 3.1(a), is usually idealized to the bilinear form, shown by the solid lines in Figure 3.1(b), although strain hardening (broken lines) is taken into account in some cases.

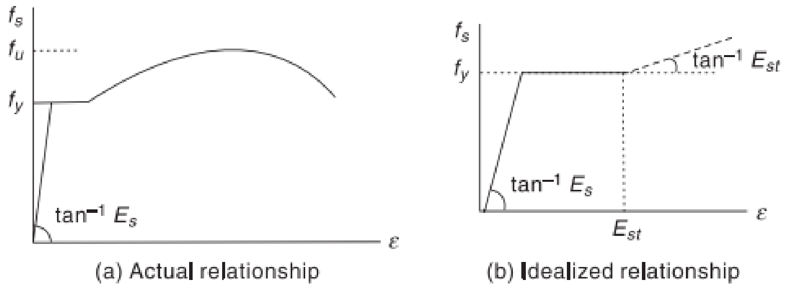


Figure 3. 1. Stress–strain relationship of steel **(Duggal 2013).**

The hysteretic stress–strain relationship for steel, subjected to alternately repeated loading, is shown in Figure 3.2(a). The unloading branch shows an incipient slope equal to the elastic slope and is gradually softened owing to the Bauschinger effect. Due to the Bauschinger effect, the plastic deformation of steel increases the tensile yield strength and decreases the compressive yield strength. Some of simple models of hysteretic stress–strain curves are shown in Figures 3.2(b), (c) and (d).

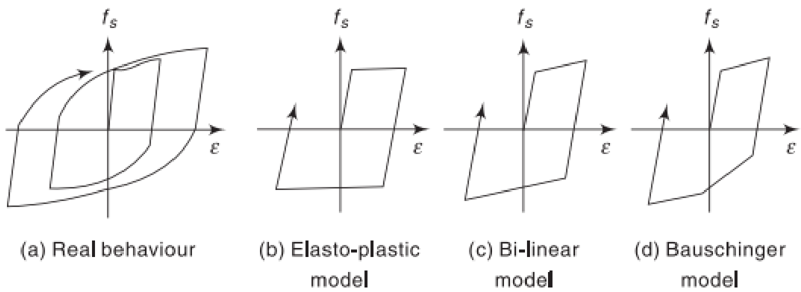


Figure 3. 2. Hysteretic behaviour of steel **(Duggal 2013).**

3.2.2 Elastic and Plastic Behaviour of Steel

This section first describes the structural behaviour of a cross section from its elastic state to a fully plastic state under increasing load. The general elasto-plastic behaviour of a structure will then be given and its application to plastic design method, under certain limitations, is compared with the elastic design method.

Most structural materials undergo an elastic state before a plastic state is reached. This applies to both material behaviour of a cross section and the structure as a whole. For a simply supported steel beam with a cross section symmetrical about a horizontal axis under an increasing load applied at midspan, the general stress and strain variations in the cross section at midspan from a fully elastic state to fracture are shown in Figure 3.3. The beam is initially loaded producing an elastic stress.

For design purposes, it is prudent to ignore the extra strength provided by strain hardening, which becomes smaller in magnitude as the grade strength of steel becomes greater. Hence, for simplicity, steel is always idealized as an elastic-perfectly plastic material with a stress–strain relationship shown in Figure 3.3 and the corresponding cross-section plastification of a symmetric section in Figure 3.3.

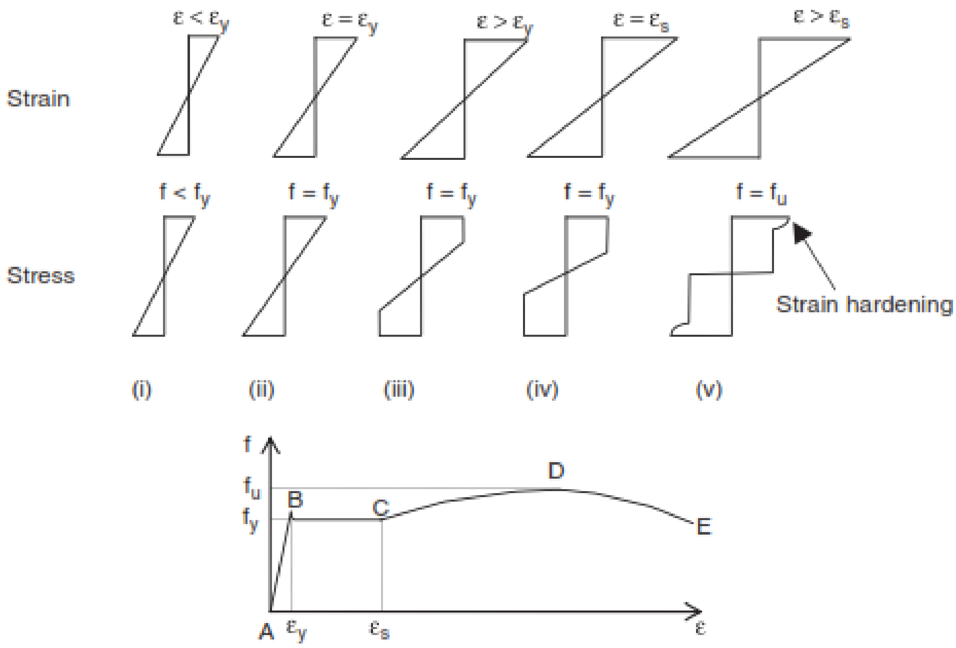


Figure 3.3. Stress–strain behaviour of a cross section **(Wong 2011).**

3.2.3 Classification of cross-sections to EC3

Definitions of the four classes are as follows (EC3 clause 5.5.2(1)),

Class 1: cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.

Class 2: cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.

Class 3: cross-sections are those in which the elastically calculated stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.

Class 4: cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

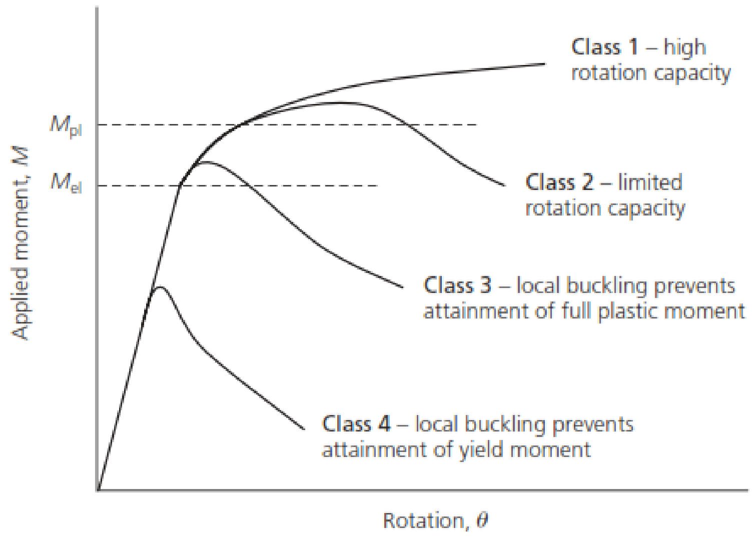


Figure 3. 4. The four behavioural classes of cross-section defined by EC3 **(Gardner 2011, EC3 2003).**

The moment–rotation characteristics of the four classes are shown in Figure 3.4.

Class 1: cross-sections are fully effective under pure compression, and are capable of reaching and maintaining their full plastic moment in bending (and may therefore be used in plastic design).

Class 2: cross-sections have a somewhat lower deformation capacity, but are also fully effective in pure compression, and are capable of reaching their full plastic moment in bending.

Class 3: cross-sections are fully effective in pure compression, but local buckling prevents attainment of the full plastic moment in bending; bending moment resistance is therefore limited to the (elastic) yield moment.

Class 4: cross-sections, local buckling occurs in the elastic range. An effective cross-section is therefore defined based on the width-to-thickness ratios of individual plate elements, and this is used to determine the cross-sectional resistance.

N.B. In hot-rolled design the majority of standard cross-sections will be Class 1, 2 or 3, where resistances may be based on gross section properties obtained from section tables (EC3).

3.3 LINEAR AND NONLINEAR ANALYSES

3.3.1 Introduction

Before the strength of cross-sections and the stability of members can be checked against the requirements of the code, the internal (member) forces and moments within the structure need to be determined from a global analysis. Four distinct types of global analysis are possible:

1. First-order elastic – initial geometry and fully linear material behaviour
2. Second-order elastic – deformed geometry and fully linear material behaviour
3. First-order plastic – initial geometry and non-linear material behaviour
4. Second-order plastic – deformed geometry and non-linear material behaviour.

Typical predictions of load–deformation response for the four types of analysis are shown in up mentioned Figure 3.5 **(Gardner 2011, EC3 2003).**

3.3.2 Linear analysis

Linear analysis is the most usual one in practical applications and explicitly used for verifications at the serviceability limit state. It is simpler, quicker, more straightforward and has the advantage that it allows linear superposition of the results of the individual load cases for combined effects. However, it does not consider the effects of stability and plasticity that must be accounted for later in design.

Displacements and strains are small so that material behaviour is elastic and analysis may be performed on the basis of the initial, un-deformed geometry of the structure. This analysis is also called elastic analysis according to 1st order theory. These linear analyses pertain to structural systems which have linear inertia, damping and restoring forces.

In linear analysis, displacements and strains are small so that material behaviour is elastic and analysis may be performed on the basis of the initial, un-deformed geometry of the structure. This analysis is also called elastic analysis according to 1storder theory. Design is made separately by application of code-prescribed formulas.

3.3.3 Nonlinear analysis

Whenever the structural system has any or all of the three reactive forces having non-linear variation with the response parameters (that is, displacement, velocity, and acceleration), a set of non-linear differential equations are evolved and need to be solved. The most common non-linearities among the three are the stiffness and the damping non-linearities. In the stiffness non-linearity, two types of non-linearity may be encountered, namely, the geometric non-linearity and the material non-linearity

3.3.4 Resources of nonlinearities

- **Geometrically non-linear elastic analysis (GNA)**

Due to large displacements, equilibrium is defined in the deformed state of the structure under loading. Material behaviour is elastic. This analysis is also called elastic analysis according to 2nd order theory. It is appropriate for stability investigations up to the buckling load under moderate displacements.

- **Materially non-linear analysis (MNA)**

Displacements are small but strains are large. Analysis may be performed on the basis of the initial, un-deformed geometry of the structure but the effects of non-elastic irreversible strains must be taken into account. This analysis is also called 1st order plastic analysis.

3.4 SUMMARY OF THE TYPES OF STRUCTURAL ANALYSES

Broadly speaking, structural analysis may be divided into three large principal groups. They are static analysis, stability, vibration and seismic analysis. The purpose of non-linear static analysis is to determine the displacements and internal forces due to time-independent loading conditions, as if a structure is nonlinear. There are different types of nonlinearities. They are physical (material of a structure which does not obey Hook's law), geometrical (displacements of a structure are large), structural (structure with gap or constraints are one-sided, etc.), and mixed nonlinearity (Figure 3.5).

3.4.1 Global frame analysis

Global Analysis is aimed at deriving the values of the internal forces and of the displacements in the considered structure when this one is subjected to a given set of loads. It is based on assumptions regarding the component behaviour (elastic or plastic) and the geometrical response (first-order or second-order theory) of the frame.

Once the analysis is complete, i.e. all relevant internal forces and displacements are determined in the whole structure, then the design checks of all the frame components is performed. These ones consist in verifying whether the structure satisfies all the required design criteria under service loads (serviceability limit states – SLS) and under factored loads (ultimate limit states – ULS).

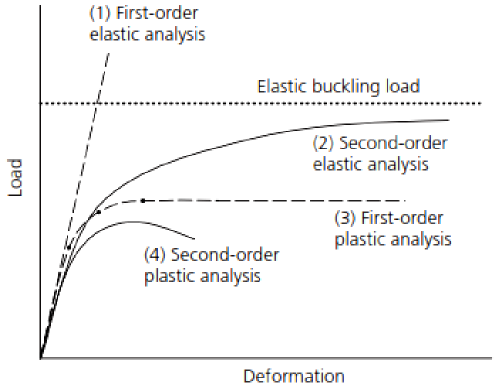


Figure 3. 5. Prediction of load–deformation response from structural analysis **(Gardner 2011).**

3.4.2 Stability analysis

Stability analysis deals with structures which are subjected to compressed time in dependent forces. For tall and flexible structures, the transversal displacements may become significant. Therefore, it should be taken into account the additional bending moments due by axial compressed loads P on the displacements caused by the lateral loads, that is the P-delta analysis, the structural analysis is performed on the basis of the deformed design diagram. The purpose of buckling analysis is to determine the critical load (or critical loads factor) and corresponding buckling mode shapes.

3.4.3 Dynamical analysis

Dynamical analysis means that the When structures are subjected to time-dependent loads, the shock and seismic loads, as well as moving loads with taking into account the dynamical effects.): The purpose of Free-vibration analysis (FVA) or model analysis is to determine the natural frequencies (eigenvalues) and corresponding mode shapes (eigen functions) of vibration. This information is necessary for dynamical analysis of any structure subjected to arbitrary dynamic load, especially for seismic analysis. This analysis can be applied to linear and nonlinear structures.

3.4.4 Linear buckling analysis (LBA)

This analysis provides buckling eigenvalues and buckling modes under the assumption of small displacements, elastic material behaviour and no imperfections. It is a useful tool to explore to what extend geometric effects influence the structural behaviour and to provide deformed structural shapes that may be used as geometrical imperfections.

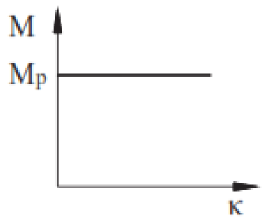
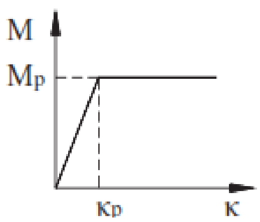
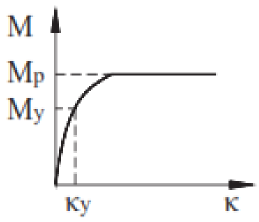
3.4.5 Geometrically non-linear elastic analysis (GNA)

Due to large displacements, equilibrium is defined in the deformed state of the structure under loading. Material behaviour is elastic. This analysis is also called elastic analysis according to 2nd order theory. It is appropriate for stability investigations up to the buckling load under moderate displacements.

3.4.6 Materially non-linear analysis (MNA)

Displacements are small but strains are large. Analysis may be performed on the basis of the initial, un-deformed geometry of the structure but the effects of non-elastic irreversible strains must be taken into account. This analysis is also called 1st order plastic analysis. For frame structures it is further distinguished into rigid plastic analysis, plastic hinge analysis or plastic zone analysis as illustrated in Table 3.1. It may be used for design as long as geometric effects and imperfections can be ignored.

Table 3. 1. Non-linear moment-curvature diagrams of cross-sections for beam elements **(Vayas 2019)**.

	Cross-section behavior		
Moment – curvature diagram			
Types of frame analysis	Rigid plastic analysis	Plastic hinge analysis	Plastic zone analysis

3.4.7 Seismic analysis

The concern about seismic hazards has led to an increasing awareness and demand for structures designed to withstand seismic forces **(Duggal 2013)**. Nowadays, a central issue, in seismic design, is the accurate evaluation of structural response due to the large uncertainty in the non-linear complex behaviour of the buildings. Assessment of the seismic response of buildings should provide load distributions and displacements resulting from design seismic action. In particular, the analysis should evaluate seismic demand in terms of strength and ductility within each structural element, taking part in the absorption of seismic forces. These forces act in dynamic equilibrium.

3.5 EFFECT OF SEISMIC INPUT CHARACTERISTICS ON STRUCTURAL RESPONSE OF BUILDINGS

A dynamic loading can be defined as any loading which varies with time, and seismic loading is a complex variant of this. The way in which a structure responds to a given dynamic excitation depends on many factors including the nature of the excitation and the dynamic characteristics of the structure, i.e. on the manner in which it stores and dissipates vibrational energy. In addition, seismic excitation may be described in terms of displacement, velocity, or acceleration varying with time. When this excitation is applied to the base of a structure it produces.

Seismic inputs are the earthquake data that are necessary to perform different types of seismic analysis. In the context of seismic analysis and design of structures, various earthquake data may be required depending upon the nature of analysis being carried out

. Seismic inputs for structural analysis are provided either in the time domain or in the frequency domain, or in both time and frequency domains. In addition, a number of earthquake parameters are also used as seismic inputs for completeness of the information that is required to perform different types of analysis. They include magnitude, intensity, peak ground acceleration/velocity/displacement, duration, predominant ground frequency, and so on **(Datta 2010)**.

The seismic action is characterised by ground vibration under the base of structures its acceleration, velocity and displacement imposed to the base of the structure vary with the duration of the motion. It is thus clear that the seismic action has dynamic properties that interact with those of the stricken building. Indeed, due to ground vibrations, deformations occur in the structure, which in turn generate restoring forces (due to the structure’s stiffness), damping forces (due to internal friction, external dissipation and essentially by the damage of the dissipative zones) and inertial forces (due to the mass of the structure) **(Gardner 2011).**

3.6 SEISMIC RESPONSES OF A STRUCTURE

The assessment of the seismic response of buildings should provide load distributions and displacements resulting from design seismic action. In particular, the analysis should evaluate seismic demand in terms of strength and ductility within each structural element, taking part in the absorption of seismic forces. The response of the structure may be measured in many ways. The response of steel structures, including multi-storied buildings, to loading is influenced by two non-linear effects, the non-linearity concerning material behaviour and the geometric non-linearity. The first is due to large strains the second due to large displacements. Furthermore, real structures deviate from the ideal ones in respect to both geometric properties and stresses in the unloaded condition **(Vayas 2019).**

The current design philosophy is to design the structures for forces that are much lower than the expected design earthquake forces, so that the structures can undergo inelastic deformations and damages under severe earthquakes without collapse.

In design practice, in some cases the structure will be required to remain elastic in the design earthquake, but more commonly some degree of inelastic behaviour will be assumed **(Dowrick 2003).** This capability to deform sufficiently in the inelastic range can be done by the structural system by giving it sufficient ductility.

3.7 SEISMIC RELIABILITY ANALYSIS

Seismic reliability or risk analysis of structures can be performed with different degrees of complexity as outlined previously. Uncertainties that could be considered in the analysis are shown in the excellent flow chart **(Datta 2010)** in Figure 3.6.

It is practically impossible to consider all uncertainties in one analysis. Further, the estimated probability of failure obtained by any reliability analysis technique cannot be accurate because of the approximations involved in each method. Thus, the calculated seismic reliability is at best a good estimate of the actual reliability of structures against the failure event.

In view of this, many simplified seismic reliability analyses have been proposed by several researchers **(Der Kiureghian 1981, 1996).** They are useful in obtaining an estimate of the seismic reliability of structures considering some (but not all) of the uncertainties at a time. Some of them are described here. They include:

(i) reliability analysis of structures considering uncertainty of ground inputs only;
(ii) reliability analysis of structures using seismic risk parameters of the site;
(iii) threshold crossing reliability analysis of structures for deterministic time history of ground motion;
(iv) first passage reliability analysis of structures for random ground motions;
(v) reliability analysis using damage probability matrix;
(vi) simplified probabilistic risk analysis of structures.

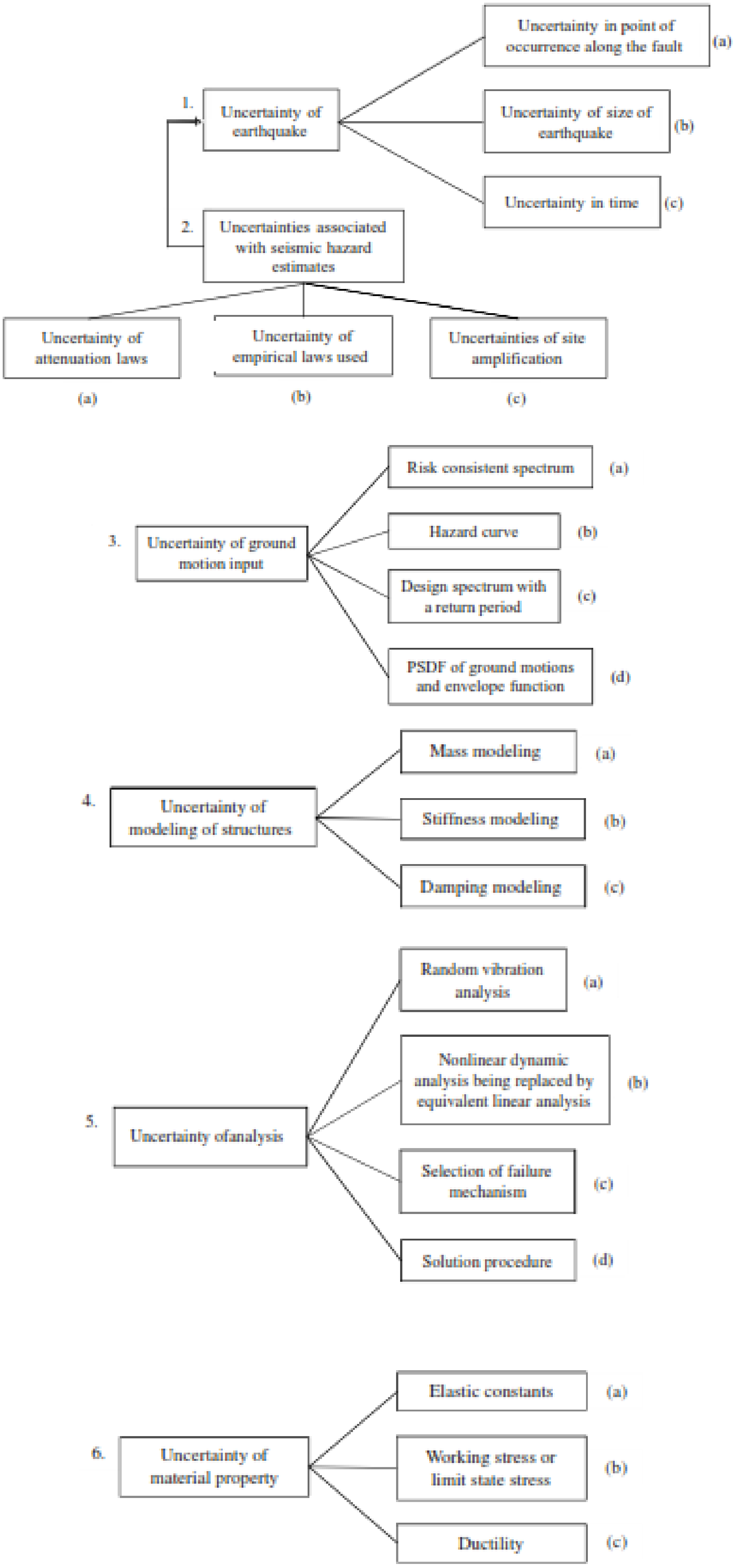


Figure 3. 6. Different types of uncertainties in seismic risk analysis **(Datta 2010).**

3.8 STRUCTURAL SEISMIC ANALYSIS

3.8.1 General

The structural behaviour requires the understanding of a series of physical phenomena related to the occurrence of ultimate limit states like flange local buckling, web local buckling, lateral torsional buckling, plastic hinge formation, or even, the crack distributions. The analysis of a structural system, as an essential step in the design of a structure to resist earthquakes, is required to determine the structural response to the applied loadings or ground excitation in terms of internal forces and moments, stresses strains, deformations. For seismic context, the performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. This can be also done by setting up appropriate numerical models implanted in computer codes.

3.8.2 Modelling

Using finite element methods (FEM) in order to represent the structure and perform the analysis at the design stage, through global model in a 3D (three dimensional) or 2D (two dimensional). Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures. Generally speaking, a structural analysis procedure requires:

- A model of the structure,
- A method of analysis for forming and solving the governing equations. In fact, it does exist a range of methods from a plastic analysis to a sophisticated nonlinear, dynamic analysis of a detailed structural model that can be used, depending on the purpose of the analysis in the design process.

3.8.3 Seismic analysis and design procedures

The problem of selecting an analysis method, of course, depends largely upon whether the materials are intended to be elastic or inelastic during the design earthquake. The usual methods of analysis for these two states are set out and well summarized in Table 3.2 (**Dowrick 2003).**

The structural analysis procedures used in earthquake-resistant design are summarized in Table 3.2 and discussed in the following section.

As already mentioned, the response of steel structures, including multi-storied buildings, to loading is influenced by two non-linear effects, the non-linearity concerning material behaviour and the geometric non-linearity. The first is due to large strains the second due to large displacements. Furthermore, real structures deviate from the ideal ones in respect to both geometric properties and stresses in the unloaded condition. This refers to both the components and the complete structure that deviate from the ideal one due to imperfections created during the manufacturing and erection process.

Table 3. 2. Seismic analysis and design procedures (**Dowrick 2003)**.

Method of analysis	Material behaviour	Design provisions	Seismic loading
Static linear	Elastic	(1a) Permissible stress or factored ultimate design	Full
		(1b) Sometimes detailed for nominal ductility	
	Inelastic	(2) Permissible stress or factored ultimate design Detailed for ductility	Reduced by factor R (Figure 5.29)
Dynamic linear	Elastic	(3a) Permissible stress or factored ultimate design	
		(3b) Sometimes detailed for nominal ductility	Full
	Inelastic	(4) Permissible stress or factored ultimate design Detailed for ductility	Reduced by factor R (Figure 5.29)
Dynamic non-linear	Inelastic	(5) Hysteresis loops required Ultimate strength design Ductility demands found from plastic deformations	Full

- **Linear and nonlinear analyses**

An important decision in a structural analysis is to assume whether the relationship between forces and displacements is linear or nonlinear. Although the seismic design for buildings is currently based on elastic analysis, nonlinear structural analysis has become increasingly important in the investigation of structural response to earthquakes.

Nonlinear structural analysis, but the existing method used for calculating the nonlinear behaviour of civil engineering structures is often by changing the structural member stiffness.

Linear analysis for static and dynamic loads has been used in structural design for decades. Nonlinear analysis methods in civil engineering are not a new topic and are widely used, because emerging performance-based guidelines require representation of nonlinear behaviour.

There are two major sources of nonlinear behaviour of a structure or an element of structure. The first is a nonlinear relationship between force and deformation resulting from material behaviour such as ductile yielding, stiffness and strength degradation or brittle fracture. The second type of nonlinear behaviour is caused by the inclusion of large displacements in the compatibility and equilibrium relationships.

In succinct manner, the main characteristics of the various methods are recalled. Further details can be found in **(Vayas 2019).**

- **Resolution procedure**

After a structural model and earthquake loading are defined, an analysis method is needed to compute the response of the structure. The governing equations are formed using equilibrium, compatibility and force-deformation relationships for the elements and the whole structure, and are expressed in terms of unknown displacements (or degrees of freedom). To elucidate the theory and provide a compact mathematical representation, the fundamental relationships are expressed using matrix algebra. Since the governing equations may have a large number of degrees of freedom, they must be solved numerically using a computer-based analysis method. Nearly all structural analyses for earthquake-resistant design are performed using software that incorporates one or more of the analysis methods presented in this chapter. Modern software generally includes graphical features for visualizing the forces and deformations computed from an analysis.

3.9 STRUCTURAL MODEL

3.9.1 Introduction

The estimation of seismic demands requires the development of a mathematical model of the building. The model should incorporate all components that influence the mass, stiffness, and strength of the building, particularly in the inelastic regime of the response. The model should properly account for gravity loads that comprise dead loads and other permanent fixtures. Consideration of live loads is also necessary if the presence of additional gravity loads is likely to create a situation resulting in adverse seismic response or the shifting of the location of the plastic hinge. A structural model also includes specification of the expected behaviour of all of the elements used to develop the building model. The development of analytical models must account for all possible aspects of behaviour while still working within the limitations of the analytical tools being used to carry out the evaluation of seismic demands. Both commercial and non-commercial software is available to assist engineers in estimating seismic demands, and the modelling requirements vary from one tool to the next.

3.9.2 Choice of structural model

This section outlines the fundamentals and basic assumptions relating to the modelling of structures and joints. It states that the chosen (calculation) model must be appropriate and must accurately reflect the structural behaviour for the limit state under consideration. For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. The choice between a first- and a second-order analysis should be based upon the flexibility of the structure; in particular, the extent to which ignoring second-order effects might lead to an unsafe approach due to underestimation of some of the internal forces and moments. A linear elastic analysis requires only the estimation of the effective stiffness of each element, whereas a nonlinear analysis demands a more concerted effort to establish the expected local behaviour of every element in the overall structural model.

3.9.3 Modelling the parameters

The structural model for seismic analysis must be capable of simulating (**Landolfo 2017)**:

(1) Masses, (2) damping and (3) the distribution of structural mechanical properties in the structure. The level of refinement and complexity for the modelling of these features basically depends on the two aspects:
1) The regularity and complexity of the structural configuration;
2) The method of structural analysis.

- **Modelling of masses**

Generally, the mass of a body can be concentrated in its centre of gravity and it is characterized by three translational and three rotational components and is distributed in its whole volume.

There are two ways of forming the mass matrix of the structure: lumped mass and consistent mass. The simplest method for considering the inertial properties for a dynamic system is to assume that the mass of the structure is lumped at the nodal coordinates where translational displacements are defined, hence the name lumped mass method (**Paz and Kim 2019).** The dynamic analysis using the lumped mass matrix requires considerably less computational effort than the analysis using the consistent mass method for the following reasons. The lumped mass matrix for the system results in a diagonal mass matrix whereas the consistent mass matrix has many off diagonal terms which are called mass coupling. Also, the lumped mass matrix contains zeros in its main diagonal due to assumed zero rotational inertial forces. (**Paz and Kim 2019).** The consistent mass matrix is fully populated whereas lumped mass matrix is diagonal, even though consistent mass is more accurate, lumped mass gives better results because both stiffness and mass are over-estimated, thus resulting in the correct answer **(Rajasekaran 2009)** and the excellent publication by **(Deshpande 2016)**. However, to simplify the structural analysis, in most cases, the mass can be considered lumped in the nodes of the structural model.

The correct modelling of the structural mass is crucial in seismic analysis, because the seismic forces are inertial actions that appear due to accelerations induced to the structural masses. The structural analyst should clearly distinguish between mass and weight, which must be never confused. Indeed, mass refers loosely to the amount of “matter” in a body, whereas weight refers to the force experienced by an object due to acceleration of gravity (e.g. a body with a mass equal to 1.0 kilogram has a weight approximately equal to 9.81 Newton, being 9.81 m/s^2 the acceleration of gravity).

- **Modelling of damping**

Damping is the general the term used to characterize energy dissipation in a building frame, irrespective of whether the energy is dissipated by hysteretic behaviour or by viscous damping **(ATC-19 1995).** The damping reduction factors are used in a variety of building codes in order to estimate the elastic response spectrum with higher or lower damping ratios (ξ) from 5% critical damping.

The modelling of damping is not required for lateral force method, modal response spectrum analysis and nonlinear static analysis. Indeed, in these types of analysis, the damping properties of the system are directly included in response spectra by calculating the damping correction factor that modifies the ordinates of the elastic spectrum, and characterises the seismic demand. However, the explicit modelling of damping is necessary for linear and nonlinear dynamic analyses **(Rajasekaran 2009).**

- **Modelling of structural mechanical properties**

The assumptions and the level of refinement required for structural models differ with the type of analysis. In EC8, for elastic analyses (i.e. lateral force method, modal response spectrum and linear dynamic analysis) the structural model must adequately represent the distribution of stiffness of members into the building (see clause 3.3.1(1)P). In addition, the elastic structural model of a steel building should include the connections, the web of the columns (for moment resisting frames) and the stiffness of foundation joints (e.g. soil to-footings interaction) if those components are characterized by significant deformability that potentially modifies the overall dynamic behaviour and the internal distribution of forces. Moreover, the structural model shall include non-structural elements that may significantly influence the response of the main seismic structure. The modelling of bracing is dependent on the slenderness ratio (l/r).

3.10 OVERVIEW OF THE MENU FOR EARTHQUAKE-RESISTANT STRUCTURES ANALYSIS METHODS IN SEISMIC CURRENT CODES

3.10.1 General considerations

Significant changes have occurred in the approach to seismic design of steel structures over the last twenty years. These changes have been brought about by a more general understanding of the nature of the problem and the development of the digital computer.

It is difficult to give clear general advice on selecting the means of analysis, as each structure will have its own requirements, technical, statutory, economic and sometimes political. However, the larger and/or more complex the structure, the more sophisticated the dynamic analysis used. In this section, a brief account of the reviewing and discussing concerning the current seismic design codes for steel building located in seismic zones is presented.

In general, the methodology used to design buildings located on seismic zones are mostly based on capacity design criteria using the force-based design as fully discussed in the previous chapter. Various simplifications are often adopted in order to simplify the structural analysis. A brief recall of the aforementioned methods of analyses are summarised in terms of structure kinds in Table 3.3.

In addition to what it has been already said on the seismic analysis of structures in previous sections, the seismic excitation may be described in terms of displacement, velocity, or acceleration varying with time. When this excitation is applied to the base of a structure it produces a time-dependent response in each element of the structure, which may be described in terms of motions or forces. The dynamic characteristics of such a system are simply described by its natural period of vibration T (or frequency ω) and its damping ξ.

Table 3. 3. Methods of seismic analysis and types of structure **(Bozorgnia 2006).**

Type of structure	Method of analysis (two-dimensional or three-dimensional)
Small simple structures	(1) Equivalent static forces
↓	(2) Response spectra
Progressively more demanding structures	(3) Modal analysis
↓	(4) Direct integration
Large complex structures	(5) Non-linear soil-structure

3.10.2 Code's approaches

This section presents the seismic design of structures in accordance with several seismic codes. The seismic design of structures requires a clear load path for both vertical and lateral actions.

Practically, instead of solving the equations of motion, different methods for structural seismic analysis can be adopted. The current methods that have been employed to design buildings are described herein. The plastic analysis only requires the equilibrium relationships, and is useful for capacity design procedures (**Paulay 1992**), see chapter 3 for details. For a given load distribution and flexural strength of members, plastic analysis gives the collapse load and the location of plastic hinges in members.

The analysis procedures in Table 3.4 are in order of increasingly rigorous representation of structural behaviour, but also increasing requirements for modelling and complexity of the analysis. The linear static procedure has been a traditional structural analysis method for earthquake-resistant design Uniform Building Code **(UBC 1997)**, but it does not represent the nonlinear behaviour or the dynamic response of a structure caused by an earthquake ground motion. The simplest dynamic analysis method is based on a linear model of the structure, which permits use of vibration properties (frequencies and mode shapes) and simplification of the solution with a modal representation of the dynamic response. An estimate of the maximum structural response can be obtained with response spectrum analysis, or the maximum can be computed by response history analysis with specific earthquake ground motion records. Linear dynamic analysis methods are covered in depth by **Chopra (2001)**.

For the earthquake analysis of many types of structures, it is reasonable to assume that the foundations and soil are rigid compared to the structure itself and that the supports of the structure move in phase during an earthquake ground motion. Soil-structure interaction modifies the input motion to a structure because of wave propagation and energy dissipation in the soil.

Table 3. 4. Structural Analysis Procedures for Seismic-Resistant Design **(Filippou 2004)**.

Category	Analysis Procedure	Force Deformation Relationship	Displacements	Earthquake Load	Analysis Method
Equilibrium	Plastic analysis procedure	Rigid-plastic	Small	Equivalent lateral load	Equilibrium analysis
	Linear Static procedure	Linear	Small	Equivalent lateral load	Linear static analysis
Linear	Linear dynamic procedure 1	Linear	Small	Response spectrum	Response spectrum analysis
	Linear dynamic procedure 2	Linear	Small	Ground motion history	Linear response History analysis
Nonlinear	Nonlinear static procedure	Nonlinear	Small or large	Equivalent lateral Load	Nonlinear static analysis pushover
	Nonlinear dynamic procedure	Nonlinear	Small or large	Ground motion history	Nonlinear response history analysis

3.10.3 Code's seismic procedures

According to several international seismic codes, including US and European ones, the following analysis options for the design of buildings and for the evaluation of their seismic performance can be used:

- Linear static analysis: equivalent static analysis (termed the 'lateral force' method of analysis in EN 1998-1);
- Modal response spectrum analysis (also termed in practice 'linear dynamic' analysis. Do not to be confused with linear time-history analysis;
- Non-linear static analysis, commonly known as Pushover Analysis, not included yet in RPA99 version 2003;
- Non-linear dynamic analysis or time-history or response-history analysis, not included in RPA99.

Linear time-history analysis is not explicitly mentioned as an alternative to linear modal response spectrum analysis.

Unlike US and Algerian codes, which consider the linear static analysis as the reference method for the seismic design of buildings, Eurocode 8 gives this status to the modal response spectrum method.

The linear methods of analysis use the design response spectrum, which is essentially the elastic response spectrum with 5% damping divided by the behaviour factor R (q in EC8). Internal forces due to the seismic action are taken to be equal to those estimated from the linear analysis; however, and consistent with the equal displacement rule and the concept and use of the behaviour factor R or q, displacements due to the seismic action are taken as equal to those derived from the linear analysis, multiplied by the behaviour factor R or q. In contrast, when a non-linear analysis method is used, both internal forces and displacements due to the seismic action are taken to be equal to those derived from the non-linear analysis.

The use of a linear method of analysis does not imply that the seismic response will be linear elastic; it is simply a device for the simplification of practical design within the framework of force-based seismic design with the elastic spectrum divided by the behaviour factor. The demand spectrum is reduced, as appropriate, to account for the inelastic deformation of the structure.

3.11 LINEAR SEISMIC ANALYSIS TO SEISMIC CODES

3.11.1 General

For large or complex structures, static methods of seismic analysis are often deemed to be not accurate enough and many authorities demand dynamic analyses for certain types a size of structure (**Dowrick 2003)**.

Various methods of differing complexity have been developed for the dynamic seismic analysis of structures. They all have in common the solution of the equations of motion, as well as the usual statically relationships of equilibrium and stiffness. The three main techniques used for dynamic analysis are:

(1) Direct integration of the equations of motion by step-by-step procedures.
(2) Normal mode analysis.
(3) Response spectrum techniques.

- **Direct integration**

Direct integration provides the most powerful and informative analysis for any given earthquake motion. A time-dependent forcing function (earthquake accelerogram) is applied and the corresponding response-history of the structure during the earthquake is computed. That is, the moment and force diagrams at each of a series of prescribed intervals throughout the applied motion can be found.

Linear behaviour is seldom analysed by direct integration, unless mode coupling is involved, as normal mode techniques are easier, cheaper, and nearly as accurate.

- **Normal mode analysis**

Normal mode analysis is a more limited technique than direct integration, as it depends on artificially separating the normal modes of vibration and combining the forces and displacements associated with a chosen number of them by superposition. As with direct integration techniques, actual earthquake accelerograms can be applied to the structure and a stress-history determined, but because of the use of superposition the technique is limited to linear material behaviour. Although modal analysis can provide any desired order of accuracy for linear behaviour by incorporating all the modal responses, some approximation is usually made by using only the first few modes in order to save computation time. Problems are encountered in dealing with systems where the modes cannot be validly separated, i.e. where mode coupling occurs.

- **The response spectrum technique**

The response spectrum technique the word spectrum in seismic engineering conveys the idea that the response of buildings having a broad range of periods is summarized in a single graph. For a given earthquake motion and a percentage of critical damping, a typical response spectrum gives a plot of earthquake-related responses such as acceleration, velocity, and deflection for a complete range, or spectrum, of building periods is really a simplified special case of modal analysis. The modes of vibration are determined in period and shape in the usual way, and the maximum response magnitudes corresponding to each mode are found by reference to a response spectrum. An arbitrary rule is then used for superposition of the responses in the various modes. The two most used methods of summing modal responses are the Square Root of the Sum of the Squares (SRSS) and the Complete Quadratic Combination (CQC) of **Wilson et al (1981).**

The resultant moments and forces in the structure correspond to the envelopes of maximum values, rather than a set of simultaneously existing values. The response spectrum method has the great virtues of speed and cheapness.

Although both the lateral force and the modal response spectrum method are conventionally adopted in seismic design, these types of analysis have two major limitations because of uncertainties. The first uncertainty relates to the nature of the seismic action, being a dynamic action that varies in time. The results obtained using these two methods of analysis represent the envelope of response features (e.g. internal forces, displacements, etc.), without providing any information about their time variation. The second major uncertainty is that both the lateral force and the modal response spectrum methods are elastic analyses. In reality, structures designed using the dissipative behaviour principle should guarantee plastic ductile response under the seismic action. However, the structural ductility is not directly checked, being fictitiously considered in a simplified manner through behaviour factors q (EC8), R in RPA99.

3.11.2 Modal Analysis

Modal analysis is used for calculating the linear response of multi-degree-of freedom systems. It is based on the idea that the response of a building is the superposition of the responses of individual modes of vibration, each mode responding with its own particular deformed shape, its own frequency, and with its own modal damping. Modal analysis, which is a simple alternate method to time-history analysis, can be employed in order to calculate the displacements of a building due to lateral forces and torsional moments underground excitation **(Rajasekaran 2007).** A complete modal analysis provides the history of response-forces, displacements, and deformations of a structure to a specified ground acceleration history **(Duggal 2013)**.

In a modal analysis, an attempt is used to capture the multimodal response of a building by statistically combining its individual modal responses. Therefore, accelerations corresponding to an entire range of building periods are typically required in performing the dynamic analysis **(Taranath 2005)**.

In modal analysis, it is necessary to assume linear material behaviour and viscous damping. Also, a modal analysis is commonly used because of the familiarity of modal superposition to earthquake engineers. The response of the structure is therefore determined from the responses of a number of single-degree-of freedom systems with properties chosen to be representative of the modes and the degree to which the modes are excited by the earthquake motion (**Taranath 2005, Dowrick 2003)**. Although modal analysis can provide any desired order of accuracy for linear behaviour by incorporating all the modal responses, some approximation is usually made by using only the first few modes in order to save computation time **(Dowrick 2003).**

It is worth to underline that for moderate- to-high-rise buildings, the effects of higher modes may be significant. For a fairly uniform building, the dynamic characteristics can be approximated using the general modal relationship **(Taranath 2005)**.

3.11.3 Lateral force method of analysis

The equivalent lateral force for an earthquake is a unique concept used in earthquake engineering. The concept is attractive because it converts a dynamic analysis into partly dynamic and partly static analyses for finding the maximum displacement (or stresses) induced in the structure due to earthquake excitation **(Datta 2010)**.

The equivalent lateral force for an earthquake is defined as a set of lateral static forces which will produce the same peak response of the structure as that obtained by the dynamic analysis of the structure under the same earthquake. This equivalence is restricted only to a single mode of vibration of the structure, that is, there a set of lateral force exist for each mode of vibration.

The lateral force method is the easiest structural analysis method preferred and used by structural engineers to assess the structural response because of the familiarity with the basic calculation approaches used in structural mechanics, allowing also for hand computation for simple and low-redundant structural schemes. As it is well-known, this method can be applied only to structures whose dynamic response is basically dominated by the fundamental mode of vibration. This type of analysis cannot be applied in case of complex structures that are not vertically regular because of the non-uniform stiffness, strength or mass distribution. The method includes several uncertainties detailed in **(Datta 2010).**

In fact, the structural seismic analysis commonly adopted for calculating the seismic-induced effects on structures are the lateral force method and modal response spectrum analysis, because of their computational ease and their high efficiency. It should be noted that in both methods of analysis an elastic structural response is assumed and the seismic actions are combined with gravitational loads by the use of d'Alambert's principle of superposition of effects.

The equivalent (static) lateral force for an earthquake is obtained by carrying out a modal analysis of structures, and then a static analysis of the structure with equivalent (static) lateral force in each mode of vibration is performed to obtain the desired responses. The entire procedure is known as the response spectrum method of analysis and is developed using the following steps.

1. A modal analysis of the structure is carried out to obtain the mode shapes, frequencies, and mode participation factors for the structure.
2. An equivalent static load is derived to get the same response as the maximum response obtained in each mode vibration, using the acceleration response spectrum of the earthquake.
3. The maximum modal responses are combined to find the total maximum response of the structure.

3.11.4 Linear modal response spectrum analysis

According to (**Chen 2006)**, a response spectrum is a plot of maximum amplitudes (acceleration, velocity, or displacement) of a single-degree-of-freedom (SDOF) oscillator as the natural period of the SDOF is varied across a spectrum of engineering interest (typically, for natural periods from 0.03 to 3 or more seconds or frequencies of 0.3 to 30+ Hz).

The method is primarily developed for single-point excitation with a single-component earthquake. However, the method could be extended to multi-point–multi-component earthquake excitations with certain additional assumptions. Furthermore, response spectrum method of analysis is derived for classically damped structures **(Datta 2010).**

The response spectrum technique is really a simplified special case of modal analysis. The modes of vibration are determined in period and shape in the usual way, and the maximum response magnitudes corresponding to each mode are found by reference to a response spectrum. An arbitrary rule is then used for superposition of the responses in the various modes **(Dowrick 2003).**

The two most used methods of summing modal responses are the Square Root of the Sum of the Squares (SRSS) and the Complete Quadratic Combination (CQC) **(Wilson 1981).**

Although the response spectrum provides the basis for the specification of design ground motions in all current design guidelines and code provisions, there is a recognition that the response spectrum is not capable of adequately describing the seismic demands presented by brief impulsive near-fault ground motions. This indicates the need to use time histories to represent near-fault ground motions. Since the strike of the controlling fault is usually known, the differences between ground motions in the directions normal to and parallel to the fault strike can be readily taken into account.

There are no approximations involved in the first two steps. Only the third one involves approximations. As a result, the response spectrum method of analysis is called an approximate method of analysis. The approximation introduces some errors into the computed response. The magnitude of the error depends upon the problem (both the type of structure and the nature of earthquake excitation). However, seismic response analysis of a number of structures have shown that for most practical problems, the response spectrum method of analysis estimates reasonably good responses for use in design **(Datta 2010)**. On the other hand, modal response spectrum analysis is generally considered as the reference method to calculate design forces for building structures as it accounts for the dynamic properties of the structure. Indeed, if a sufficient number of modes of vibration is considered, the calculated elastic seismic structural response can closely represent the real elastic behaviour of the structures.

Standardized response spectrums are provided each seismic code (RPA99, EC8, UBC 1997, etc.). The spectrum is a smoothed average of a normalized 5% damped spectrum obtained from actual ground motion records grouped by subsurface soil conditions at the location of the recording instrument, and are applicable for earthquakes characteristic of those that occur in California **(SEAOC 1988).**

- **Commentary**

In reality, structures designed using the dissipative behaviour principle should guarantee plastic ductile response under the seismic action. However, the structural ductility is not directly checked, being fictitiously considered in a simplified manner through behaviour factors q (EC8), R in RPA99.

3.11.5 Comparison of the provisions of the lateral force method versus modal response spectrum analysis according to RPA99 and other seismic codes

Although both the lateral force and the modal response spectrum method are conventionally adopted in seismic design, these types of analysis have two major limitations because of uncertainties. The first uncertainty relates to the nature of the seismic action, being a dynamic action that varies in time. The results obtained using these two methods of analysis represent the envelope of response features (e.g. internal forces, displacements, etc.), without providing any information about their time variation. The second major uncertainty is that both the lateral force and the modal response spectrum methods are elastic analyses.

Owing to the familiarity and experience of structural engineers with elastic analysis for static loads (due to gravity, wind or other static actions), the equivalent static method has long been and still is the preferred method for practical seismic design. In the equivalent static method, a linear static analysis of the structure is performed under a set of lateral forces applied separately in two orthogonal horizontal directions, X and Y. The intent is to simulate through these forces the peak inertia loads induced by the horizontal component of the seismic action in the two directions: X or Y.

Both, the equivalent lateral force procedure and the response spectrum analysis procedure, are based on the same basic assumptions and are applicable to buildings that exhibit a dynamic response behaviour in reasonable conformity with the implications of the assumptions made in the analysis. The main difference between the two procedures lies in the magnitude of the base shear and distribution of the lateral forces. Although in the modal method the force calculations are based on compound periods and mode shapes of several modes of vibration, in the equivalent lateral force method, they are based on an estimate of the fundamental period and simple formulae for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height **(Duggal 2013)**.

The version of the method in RPA 99 gives slightly higher values for storey shear forces, which are considered as fundamental seismic effects, as those predicted by the modal response design spectrum analysis at least for the type of structures to which the lateral force method is considered applicable, therefore in the safety side. However, the lateral forces method results in Eurocode 8 have been tuned to give similar results for storey shears as those from modal response elastic spectrum analysis (which is the reference method) for structures where the conditions of applicability are present.

For the type of structures where both the lateral force method and modal response spectrum analysis is applicable, the latter gives, on average, a slightly more even distribution of peak internal forces in different critical sections, such as the two ends of the same beam or column. These effects are translated to some savings in materials. Despite such savings, the overall inelastic performance of a structure is normally better if its members are dimensioned for the results of a modal response spectrum analysis, instead of the lateral force method. The better performance is attributed to closer agreement of the distribution of peak inelastic deformations in the non-linear response to the predictions of the elastic modal response spectrum analysis than to those of the lateral force approach.

In both RPA99 and EC8, the use of modal response spectrum analysis is not subject to any constraints of applicability, it can be adopted by a designer who wishes to use the method as the single analysis tool for seismic design of structures in 3D. In addition to this advantage, modal response spectrum analysis is more rigorous as it gives, unlike the lateral force method, results independent of the choice of the two orthogonal directions, *X* and *Y*, of application of the horizontal components of the seismic action), and hence, offers a better overall balance of economy and safety. Consequently, with today’s availability of reliable and efficient computer programs for modal response spectrum analysis of structures in 3D, and with the gradual establishment of structural dynamics as a core subject in structural engineering curricula and continuing education programmes in seismic regions of the world, it is expected that modal response spectrum analysis will grow in application and prevail in the long run. Even then, though, the lateral force method of analysis will still be relevant, due to its intuitive appeal and conceptual simplicity.

Same considerations are required for application of the equivalent static method in RPA99, EC8, and many American seismic codes (UBC97, AISC2004 etc.).

3.11.6 Nonlinear static pushover analysis

Almost for any type of nonlinear seismic analysis, initial modal analysis should be done; particularly when the structural system is classified as irregular, the modal analysis is one of the ground rules in any modern and state-of-the-art seismic codes. Correct determination of natural frequencies is extremely important and forms the basis of any further nonlinear analyses **(Senjanović 2014)**. Practical dynamic analysis of large complicated multiple-degrees-of-freedom (MDOF) is generally accomplished by computer-implemented numerical analysis techniques such as the finite element method **(Rajasekaran 2007).**

Due to these uncertainties related to the above analyses, there are situations where more advanced methods are required, such as static or dynamic non-linear analyses. These situations are manifold and include the design of special structures (large spans or heights, or with complex appearance), or the evaluation of the seismic performance of existing buildings that do not comply with the seismic demands imposed by modern design provisions.

3.12 PUSHOVER ANALYSIS – AN OVERVIEW

3.12.1 Introduction

Pushover analysis is popular in earthquake engineering, as is the response spectrum method of analysis. As the latter is a good equivalent static analysis (substitute) for the elastic dynamic analysis of structures to a given earthquake, likewise pushover analysis is a good equivalent non-linear static analysis (substitute) for the inelastic dynamic analysis of structures for the earthquake **(Datta 2010)**.

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last two decades years.

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is the given.

The pushover analysis is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads, which should approximately represent the earthquake induced forces. This method of analysis was developed as a simpler and alternative method of analysis when compared to nonlinear dynamic analysis, but still accounting for the plastic behaviour of the structural elements and geometrical nonlinearity occurring under seismic action obtained by this analysis that would indicate any premature failure or weakness. Pushover analysis provides a load versus deflection curve of the structure starting from the state of rest to the ultimate failure of the structure.

The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (EC8) but unfortunately uncovered by RPA99 version 2003 provisions.

3.12.2 Objectives of Push Over Analysis (POA)

The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformations demands for elements that have to form inelastically in order to dissipate the energy imparted to the structure.
- Consequences of the strength deterioration of individual elements on behaviour of structural system.
- Consequences of the strength deterioration of the individual elements on the behaviour of the structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.
- Estimates of the inter-storey drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff non-structural elements of significant strength, and the foundation system.

3.12.3 Pushover procedures

The ATC-40 and FEMA-273 documents have developed modelling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 3.7, five points labelled A, B, C, D, and E are used to define the force deflection behaviour of the hinge and three points labelled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 (1996) and FEMA-273 documents.

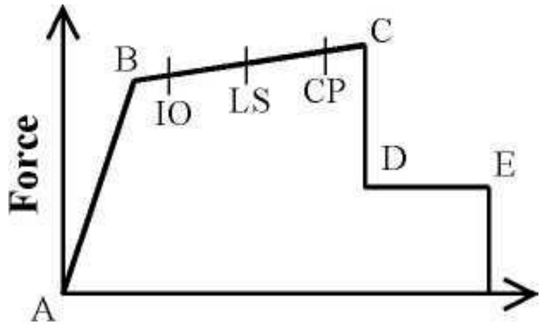


Figure 3. 7. Levels of performance in terms of Force-Deformation for a Pushover analysis.

3.12.4 Pushover principles

This method of analysis was developed as a simpler and alternative method of analysis when compared to nonlinear dynamic analysis, but still accounting for the plastic behaviour of the structural elements (see Figure 3.8) and geometrical nonlinearity occurring under seismic action.

The nonlinear static analysis is based on two hypotheses:
(1) The dynamic structural response is governed by only one mode of vibration, which (2) is represented by a distribution of the lateral forces applied to the floor masses, as for the lateral force method, and kept constant during the seismic action.

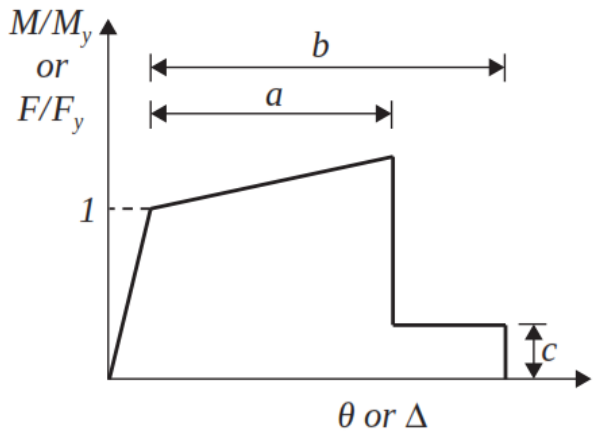


Figure 3. 8. Generalized force-deformation representation for the nonlinear behaviour of steel elements or components in case of nonlinear static analysis **(Landolfo 2017).**

The nonlinear static analysis principle is illustrated in Figure 3.9. The step "0" consists of applying the entire value of the gravitational loads in the seismic condition, which will remain constant for all the subsequent steps. With this regard, it should be noted that structural analysis soft-wares allow to control the application of forces to the numerical model of the structure by means of two different strategies:

1) Force control: Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load and should be adopted when the magnitude of the load distribution that will be applied is known a-priori and the structure is expected to resist elastically these forces.

2) Displacement control: Displacement controlled procedure should be used when specified drifts are sought where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

In the first case under force control, as shown in Figure 3.9.all loads patterns are applied incrementally from zero to their maximum specified magnitude Figure 3.9 (a). This is the case to be adopted for the step "0" of the pushover analysis.

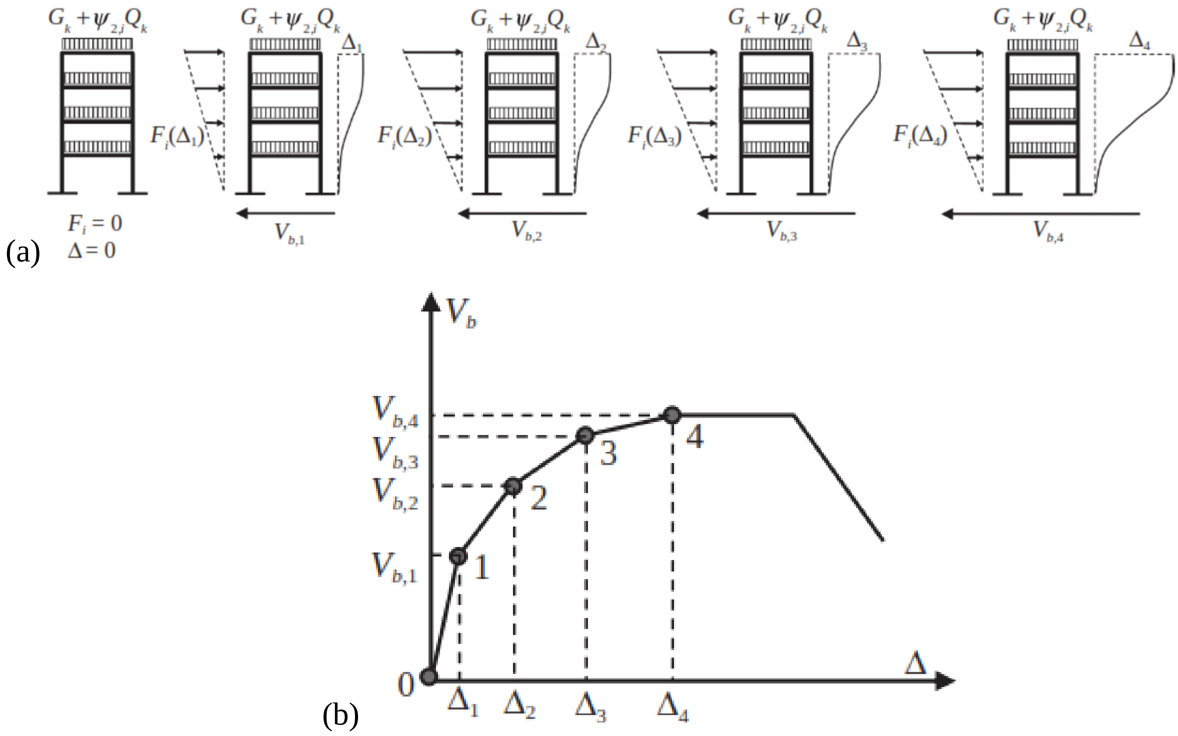


Figure 3. 9. Nonlinear static analysis principle **(Landolfo 2017).**

The graphical representation of the relationship between the base shear force "V" and the control displacement "Δ" (top lateral displacement of the structure) represents the capacity curve of the structure in Figure 3.9 (b). When running a first order global analysis, this curve linearly increases until the first plastic hinge is formed (point 1), when the structural lateral stiffness decreases. Once the lateral forces increase, more and more plastic hinges are formed, until a plastic mechanism is reached.

It is important to highlight that structural stiffness changes as plastic hinges are formed. Therefore, the fundamental mode of vibration changes and, consequently, the relevant distribution of lateral forces also changes. Figure 3.10 illustrates the variation of a lateral force's distribution corresponding to the evolution of plastic hinges pattern.

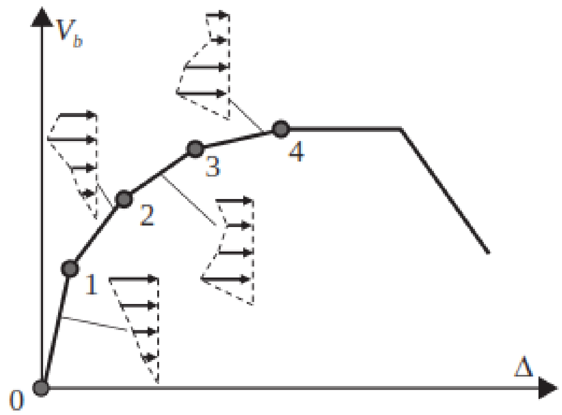


Figure 3. 10. Lateral forces distribution variation function of structural plastic Deformations **(Landolfo 2017).**

- **Loading patterns according to EC8**

– a "uniform" distribution, based on mass proportional lateral forces, regardless of height (uniform response acceleration), see Figure 3.11(a).

– a "modal" distribution, where the lateral forces are proportional to the fundamental mode of vibration weighted with the masses at each storey. This distribution corresponds to lateral forces determined as in the lateral force method.

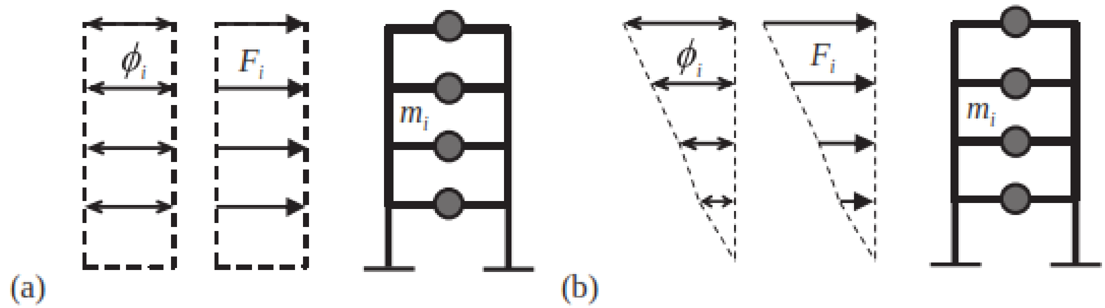


Figure 3. 11. (a) Uniform and modal distribution (b) distribution of lateral forces for the nonlinear static analysis **(Landolfo 2017).**

3.12.5 Target displacement

The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed.

The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the centre of mass of the structure distributions of lateral forces. The target displacement is determined referring to the centre of mass of the last storey of the structure.

In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor.

This assumption, which is always an approximation, can only be accepted within limitations and only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to higher mode effects.

The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure (Figure 3.12).

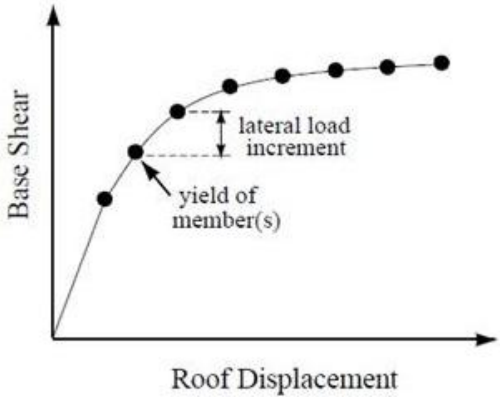


Figure 3. 12. Global Capacity (Pushover) Curve of Structure

It should be noticed that EC8 part 1 opens the possibility to use displacement-based approaches as alternative design methods, for which it presents an Informative Annex with operational rules to compute the target displacements for Nonlinear Static (Pushover) Analysis. Annex B of EC8 part 1 describes a procedure for the determination of the target displacement from the elastic response spectrum. Once the target displacement is estimated, the structure can be analysed using a nonlinear static analysis for this displacement and the structural performance is evaluated on the basis of comparison between displacement demand and corresponding plastic capacity provided by the structure. This procedure was developed by **Fajfar (1999, 2000)** and it is called the N2 method.

3.12.6 Effect of the behaviour factor (R)

The behaviour factor (R) is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. In other words, it is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. It is found through Push over analysis, Figure 3.13.

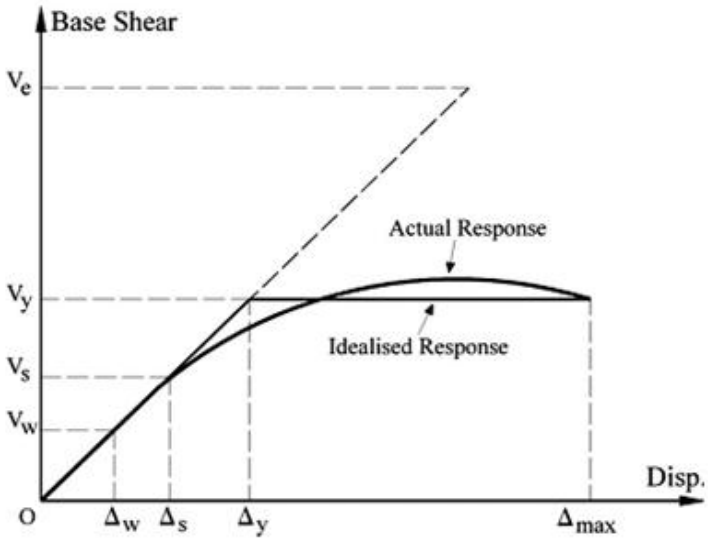


Figure 3. 13. Typical Pushover response curve for evaluation of behaviour factor, R.

The behaviour factor, R, accounts for the inherent ductility, over strength of a structure and difference in the level of stresses considered in its design. FEMA (2007), UBC (1997) suggests the R factor in force-based seismic design procedures.

3.12.7 Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, pushover predictions are accuracy and limitations of current pushover procedures must be identified. Selection of lateral load patterns and identification of failure mechanisms for estimating of target displacement due to higher modes of vibration are important issues that affect the accuracy of pushover results. In a design earthquake target displacement are global displacement are expected. The mass centre of roof displacement structure is used as target displacement. The estimation of target displacement accurate associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis. Target displacement is the global displacement expected in a design earthquake. The estimate of target displacement, identification of failure mechanisms due to higher modes of vibration are important issues that affect, selection of lateral load patterns the accuracy of pushover results.

3.12.8 Steps of performing POA

Elaborate mathematical/physical models can only be built once a structural system has been created. Such models are needed to evaluate seismic performance of an existing system and to modify component behaviour characteristics (strength, stiffness, deformation capacity) to better suit the specified performance criteria.

The second step consists of the design process that involves demand/capacity evaluation at all important capacity parameters, as well as the prediction of demands imposed by ground motions. Suitable capacity parameters and their acceptable values, as well as suitable methods for demand prediction will depend on the performance level to be evaluated.

The analysis accounts for material inelasticity, geometrical nonlinearity and the redistribution of internal forces. Response characteristics that can be obtained from the pushover analysis are summarized as follows:

- Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.
- Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.
- Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the earthquake ground motion considered.
- Sequences of the failure of elements and the consequent effect on the overall structural stability.
- Identification of the critical regions, when the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building. Pushover analysis delivers all these benefits for an additional computational effort (modelling nonlinearity and change in analysis algorithm) over the linear static analysis.

CHAPTER 4:

CASES STUDIED

LINEAR AND NONLINEAR GLOBAL BEHAVIOURS OF MULTI-STOREYED STEEL STRUCTURES

CHAPTER 4: CASES STUDIED. LINEAR AND NONLINEAR GLOBAL BEHAVIOURS OF MULTI-STOREYED STEEL STRUCTURES

In this chapter, seismic methods of analysis described in Chapter 3 have been used through study cases of multi-storeys steel frames performed by the author, starting from basic cases of multi-storeys MRFs frame, to investigate the effect of bracing systems when associated to MRFs, then the behaviour of MRFs-EBF structures (not covered by RPA99) in an elastic behaviour to more complicated ones dealing with the nonlinear analyses of regular and irregular steel structures.

4.1 INTRODUCTION

The major concern in the design of the multi-storey buildings is the structure to have enough lateral stability to resist wind and seismic forces. In the design practice, and for long time, it was generally recognised that steel is an excellent material for seismic-resistant structures, due to its performances in terms of strength and ductility and also as is capable of withstanding substantial inelastic deformations. However, as buildings and materials do not present homogeneous properties, applied loads change during the structure life cycle, structural elements connection varies with fatigue, and each structural component geometry is usually different from what is on design drawings. Consequently, uncertainty must be taken into account associated with structural safety which is the basis for the design or assessment of any structure be considered.

4.2 STRUCTURAL TOPOLOGY CONSIDERED

The structural typologies for steel buildings are classified according to the behaviour of their primary seismic resisting systems. Some basic definitions of type of frames used in this study will be provided.

4.2.1 Moment resisting frames (MRF)

The horizontal forces are mainly resisted by members acting in an essentially flexural manner. In these structures, the dissipative zones should be located in plastic hinges located at ends of beams or in the connections of the beams to the columns, so that energy is dissipated by means of cyclic bending. Steel moment resisting frames (MRFs) are typically drift sensitive structures owing to the lateral flexibility of this type of structural scheme. Hence, the stability criterion typically influences the design of MRFs. As a direct consequence, for MRFs the need to reduce inter-storey drift ratios and P-Δ effects could lead to oversize the structural members in order to increase the lateral stiffness **(Tenchini 2014)**. On the contrary braced structures are generally stiffer and less influenced by the stability criterion.

4.2.2 Dual structures MRFs- CBFs and MRFs-EBFs

In most design cases, the building structure combines MRF with braced frames (either CBF or EBF), resulting in Dual Frame systems, whereby both components are able to dissipate seismic energy. EC8-1 gives roughly some requirements for dual frames made of MRFs with X-CBFs, but it does not provide recommendations for other cases.

MRFs: Moment resisting frames (MRF) are designed based on strong column-weak beam design approach. As a result of this design methodology, plastic hinges occur in beam ends, near the beam column joints during a major earthquake and then the inelastic action distributed in the structure at beam ends can dissipate large amount of energy. Designing structures is carried out according to the recommendations of seismic design codes with the concept of SCWB (Chapter Seismic Design) and frames assumed to behave in elastic material manner dominated by shear resistance, and eventually including geometric nonlinear effects of second order (P- Δ).

CBFs: Conversely, CBFs provide the best solution regarding the limitation of the inter-storey drift demand under seismic event having a return period comparable with the lifetime of the structure **(Mazzolani 1995)**. As expected, capacity design procedures are more directly associated with the ultimate limit state, (chapter Seismic Design), but a number of checks are included to ensure compliance with serviceability conditions (**Elghazouli 2010).** The factor used to reduce the elastic seismic forces to obtain the design seismic forces is known as the structural response modification factor, which is defined explicitly in the code for each particular system, i.e. R = 4 for MFR and CBFIV, and R = 5 for CBFX. It becomes clear that by using RPA99 for steel braced structures some major imprecision occurs.

EBFs: The structural solutions ranging between the CBF, or EBF and the MRF are called dual structures, because horizontal loading being resisted in part by moment resisting frames and in part by bracing systems acting in the same plane. A similar evolution can be achieved starting from a classical one bay CBF system, with e=O, which is transformed in an MRF passing through EBF systems, characterised by different dimensions of the links, from short to long. The generation of such structural systems offers the possibility to obtain a wide range of structures, laying between a very rigid CBF up to a very ductile MRF, which can be characterised by a given combination of stiffness and ductility, for the same strength requirement.

Soft or weak-storey steel structures: The soft first storey condition is recognized as an undesirable condition of structural irregularity for seismic design worldwide. The soft first-storey is the most common feature of soft-storey irregularity and cannot be eliminated because of its important functional requirement of almost all the urban multi-storey buildings. In earthquake resistant design, the soft storey and the weak storey irregularities are reciprocal to a significant difference between the stiffness and the resistance of one of the floors of a structure and the rest of them.

4.3 DESIGN OF MODELS

Every structure to be erected in a seismic region has to be designed and constructed in such a way to meet, with an adequate degree of reliability, specific requirements connected to the return period of seismic action. Each seismic code should define a set of return periods of seismic action and the corresponding required performances, ranging from a "damage limitation" requirement to a "no-collapse" requirement. In the first case the structure will remain in the elastic range, while in the last one it will undergo large inelastic deformation.

The Algerian seismic code RPA **(RPA99 version 2003)** specifies two basic steps for seismic design of steel structures: sizing of structural element to fulfil an inter-storey drift requirement of 1% by an elastic behaviour, checking the sections to fulfil the resistance requirements 'for critical load combinations where the seismic forces correspond to the strength limit state. The factor used to reduce the elastic seismic forces to obtain the design seismic forces is known as the structural response modification factor, which is defined explicitly in the code for each particular system, i.e. R = 4 for MFRs and R = 5 for dual steel structures. In this section the modelling issues will be presented.

The design procedure is based upon the philosophy of strong-column-weak-beam (SCWB), which is not explicitly stated in RPA99, details on this concept can be found in Chapter 3 of this thesis. To guarantee overall ductile behaviour, it is recommended, as in EC8, a general principle that was conceived taking as reference MRFs frames, namely the "strong columns-weak beams" concept. This criterion aims at enforcing a global plastic failure mode with plastic hinges formed at the ends of the beams only, while the columns should remain elastic along the building height except for their bases and their top (roof level) where plastic hinges are accepted. This concept was utilised in all investigated structures regardless their respective topologies. Also, various response parameters are used for all structures to determine what correlations can be found for inelastic structures response. As nonlinear analyses can offer greater insight into the behaviour of the structure and determine if the structures satisfy performance requirements, pushover analyse implanted in SAP2000 program has been used. Default SAP2000 hinges are used in the analysis.

4.4 METHODS OF ANALYSIS

Several methods have been widely used to estimate force and deformation demands, which are imposed on structures subjected to earthquake ground motions, more details about this particular subject in chapter 3 of this thesis. Obviously, the choice of method depends on the structure and on the objectives of the analysis.

A succinct recall details are given below. The modern seismic codes allow the designer to use different analysis methodologies, in particular:

(1) The Equivalent Static Method (Lateral Force Method in EC8) is a simplified version of the modal, and in which the response method and is a static analysis which can only be employed for regular structures which respond essentially in one single mode of vibration;
(2) Multi-modal elastic analysis;
(3) Static and Dynamic Non-linear Analysis, Pushover Analysis, which are not yet addressed in the RPA 99 version 2003.

In addition to the above analyses, a commonly used procedure to determine the dynamic response of a system is the modal analysis **(Datta 2010, Chopra 2011).** Modal analysis of a structure is the operation which consists in establishing a mathematical model which can represent the dynamic properties of the real structure such as natural periods of vibration, modal forms, etc.

4.5 PARAMETRIC INVESTIGATION OF LINEAR SEISMIC BEHAVIOUR OF MOMENTS RESISTANT MULTI-STOREYS FRAMES (MRFs)

4.5.1 General description of models

The first part of the work described herein concerns a detailed global elastic behaviour of several MRFs single bay multi-storeys structures through a 2D modelling FEA implanted in LUSAS and SAP 2000 using methods prescribed RPA 99 and EC8 seismic codes and then compare their respective results. In fact, the structures studied covered a wide range of variation in characteristics geometric to help pull some of the applicable general conclusions. The seismic behaviour of these steel multi-storeys structures, including (P-delta) effect that is, the second-order analysis, throughout numerous parameters will be presented. 4, 6, and 8 storeys have been selected.

All structures have a uniform height of 3 m, and are of single bay span varying 3, 4.5 to 6 m respectively. Once again, moment resisting frames (MRF) are designed based on strong column-weak beam design approach. As a result of this design methodology, plastic hinges occur in beam ends, near the beam column joints during a major earthquake and then the inelastic action distributed in the structure at beam ends can dissipate large amount of energy. Designing structures is carried out according to the recommendations of seismic design codes with the concept of SCWB (Chapter Seismic Design) and frames assumed to behave in elastic material manner dominated by shear resistance, and eventually including geometric nonlinear effects of second order (P-Δ).

4.5.2 Characteristics and terminology of the structures studied

MRFs frames considered in this study were modelled in 2D, were implanted in SAP 2000 NL, version 14. The loading patterns were derived from both Equivalent Static Method and the Design Spectrum for RPA99 code. While in EC8, the seismic loading comes from the Lateral force's method and the elastic spectrum.

Beams: IPE 400 for the top storey and IPE450 for the remaining storeys,

Columns: in HEB Tables 4.1 and 4.2.

R: Regular structures

F: Flexible structures

The steel grade is S235 and class 1 for all structures.

N.B. The effect soft-storey will be described in further detailed analysis in both elastic and inelastic later in this chapter.

First of all, it must be noticed that the seismic mass is established in similar manner for the whole work according to both RPA99 and EC8 provisions respectively.

In this analyse, in a structure, the mass is distributed over its entire geometry which implies a continuum system with its infinite degrees of freedom and should be kept to a minimum since the larger the mass, the greater the inertia force and greater the risk of instability due to *P*-Delta effects.

The most probable value of the total mass m or "seismic mass" of the structure, the mass of each level was determined by considering: $G_{i,}$ the gravity load; Q_i the variable and the seismic mass is given by:

$$G_i + \psi_{Ei} Q_I$$

where $\psi_{Ei} = \varphi\psi_{2i}$, the combination coefficients ψ_{Ei} take into account the probability that the charges $\psi_{2i} Q_{ki}$ will not be present on the whole of the structure during the earthquake, (RPA99 and EC8) as well as the reduced character of the participation of certain masses in the movement of the structure.

The results of design process can be summarised, details are given in Tables 4.1 and 4.2. The sections selected for beams are: IPE 400 for the terrace and IPE450 for the rest of the storeys, the sections of the posts are all in HEB. The steel grade is middle steel S235 and class 1 for all structures.

Table 4. 1. Characteristics of the structures studied.

Structures		**Storeys**							
		Level 1	2	3	4	5	6	7	8
4 storeys	R-4-1	300	260	260	240				
	R-4-2	280	240	240	240				
	F-4	280	240	240	240				
6 storeys	R-6-1	400	300	300	260	260	240		
	R-6-2	400	300	300	260	260	240		
	F-6	340	280	280	260	240	240		
8 storeys	R-8-1	450	400	400	300	300	260	260	240
	R-8-2	400	340	340	280	280	240	240	240
	F-8	400	340	340	280	280	240	240	240

Table 4. 2. Geometric, seismic and mechanical data of studied structures.

Structures	**Geometric properties**	**Seismic data**		**Mechanical properties**
4 storeys	R-4-1 L = 6 m, H = 4.5 m, L/H = 4/3	RPA 99 - Zone II - Group 2 -Category S2	EC8 - Zone 3: -Category II Class B Earthquake Type1 $M_s > 5$	S235 $f_y = 235$ MPa $f_u = 360$ MPa E = 210kN / m² $\varepsilon_y = 0.1\%$ $\varepsilon_u = 20$ %
	R-4-2 L = 6 m, H = 3 m, L/H = 2			
	F-4 L = 6 m, H Ground $_{floor}$ = 4.5m, H $_{floor}$ = 3 m F = at flexible ground floor R = regular			
6 storeys	R-6-1 L = 6 m, H = 4.5 m, W/H = 4/3	RPA 99 - Zone II - Group 2 - Category S2	EC8 - Zone 3: -Category II - Class B Type 1M earthquake > 5	Same.
	R-6-2 L = 6 m, H = 3 m, L/H = 2			
	F-6 L = 6 m, H Ground $_{floor}$ = 4.5m, H $_{storeys}$ = 3 m F = at flexible ground floor R = regular			
8 storeys	R-8-1 L = 6 m, H = 4.5 m, L/H = 4/3	RPA 99 - Zone II - Group 2 - Category S2	EC8 - Zone 3: -Category II - Class B Earthquake Type1$M_s > 5$	Same.

Seismic Data

In the tables 4.3 and 4.4 are summarised the selected related seismic data used in this study.

Table 4. 3. Parameters used in the study to RPA99.

AT	**ξ %**	**R**	**B**	**Q**	**C_T**	**T_1 (dry)**	**T_2 (dry)**	**H**
0.15	**5**	**4**	**0.2**	**1.15**	**0.085**	**0.15**	**0.4**	**1**

Table 4. 4. Parameter used in the analysis EC8.

T_c	T_B	T_D	a_{gr}	S	q	γ1	ζ	a_g
0.5	0.15	2	1.1	1.2	4	1	5	1.1

4.5.3 Results and interpretations of the results

4.5.3.1 Modal analysis

- **General**

The dynamic behaviour of the structure under particular conditions of excitation in the absence of any modelling requires only the knowledge of the parameters previously discussed, for the details of the formulation of the dynamic problem **(EC8 2004, Plumier 2010, Elghazouli 2009)**.

For all structures used in this study, the procedure used for all structures regardless their lateral resisting system, the modal analysis consists of four main steps:

1. Build the model.
2. Apply loads and obtain the solution.
3. Expand the modes.
4. Review the results including the eigen values; mode shapes, frequency, period, participation factors, storey-displacements etc.

The concept of lumped and consistent mass was very well developed in Chapter 3, is a way of modelling seismic mass is being used to extract the eigen-values from LUSAS, with lumped mass, while consistent mass has been used in SAP2000.

The modal analysis outputs are the natural frequencies and mode shapes of a structure, also required in a spectrum analysis with mode superposition harmonic or transient analysis and in nonlinear analysis (Pushover).

For four and six structures, as examples of the obtained outcomes, all the figures and in the same way, figures show:

(a) Modes vs. period.

(b) Modes vs. top floor displacement

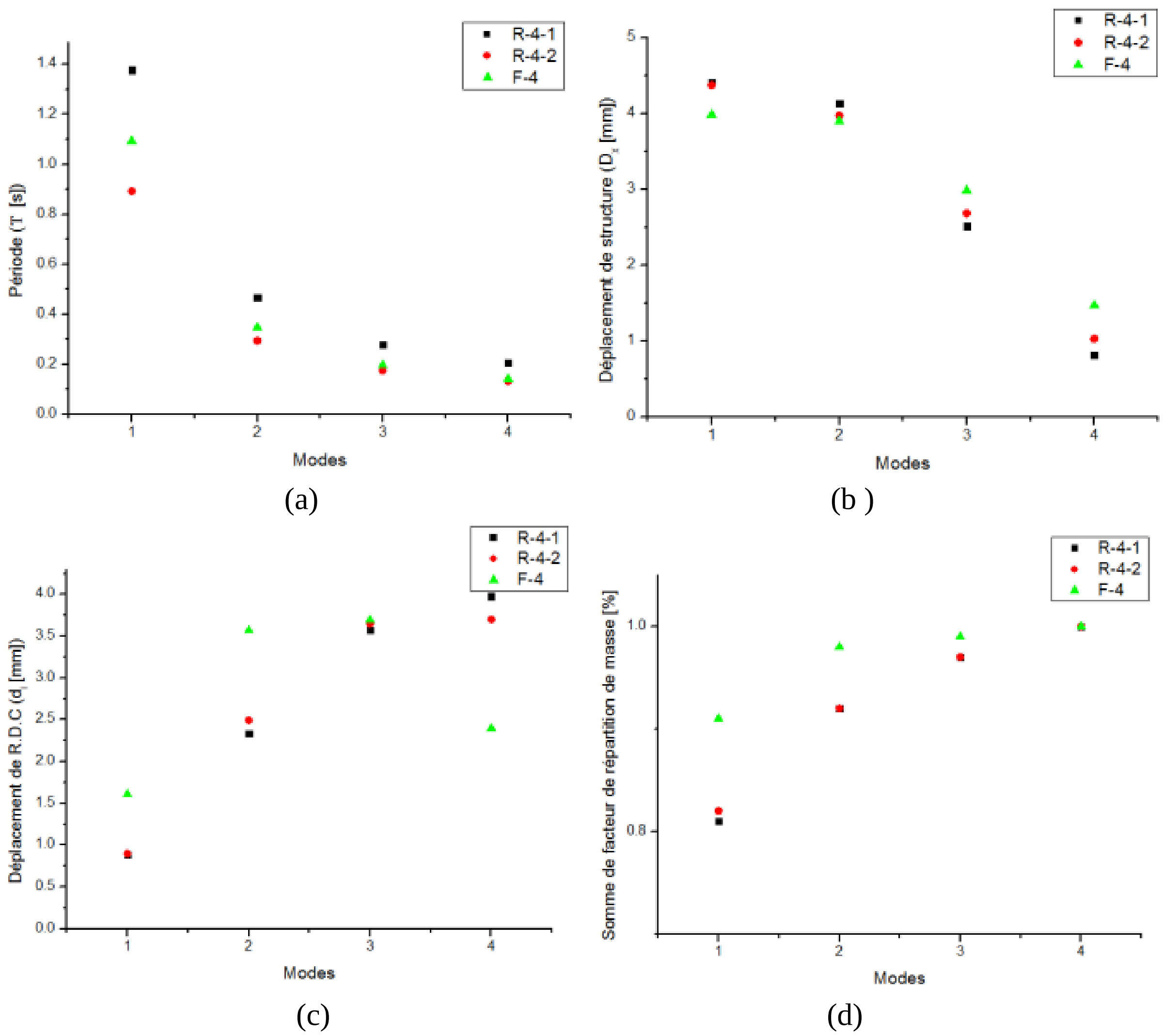
(c) Modes vs. total participation factors

- **Discussion**

An overall view of the results, and as it can be clearly seen from Figures 4.1 and 4.2, the aspect ratio (L/H) has a significant influence on the free vibration behaviour of all structures.

In all structures, and as expected, the period values decrease while going towards the higher modes of vibrations. With regard to the coefficient of participation, it can be noted that the aspect ratio (L/H) does not have a significant effect on the regular structures, and being lower compared to the flexible structures which reaches the required 90% at the first mode, with an amount of variation being generally constant. However, for flexible structures, it can be noted that the inter-storey displacement of the first floor is more important compared to regular structures only during the first modes of vibration and that the participation coefficient reaches 90%, required by the seismic codes is reached for the 3 first modes.

As far as top floor and first floor displacements are concerned, a comparison between regular and flexural structures the displacement of the structure is higher in the first floor with 79% for the first mode and becomes almost the same with the regular structure in 3 rd mode and be exceeded the fourth mode and for the first four modes, it is noticed that between R1 and 2, displacements of the top and first floors are smaller in the first floor for the ratio L/H = 2 and are larger for the top the structure. However, and contrary to the later statement, values in higher modes of vibration which one can consider as insignificant by using the prescriptions of the regulations in force. In the following, only results relating to four and six storeys will be displayed with a full discussion for all obtained will be held.



(a) (b) (c) (d)

Figure 4. 1. Model analyse outcomes for four storeys structures (LUSAS).

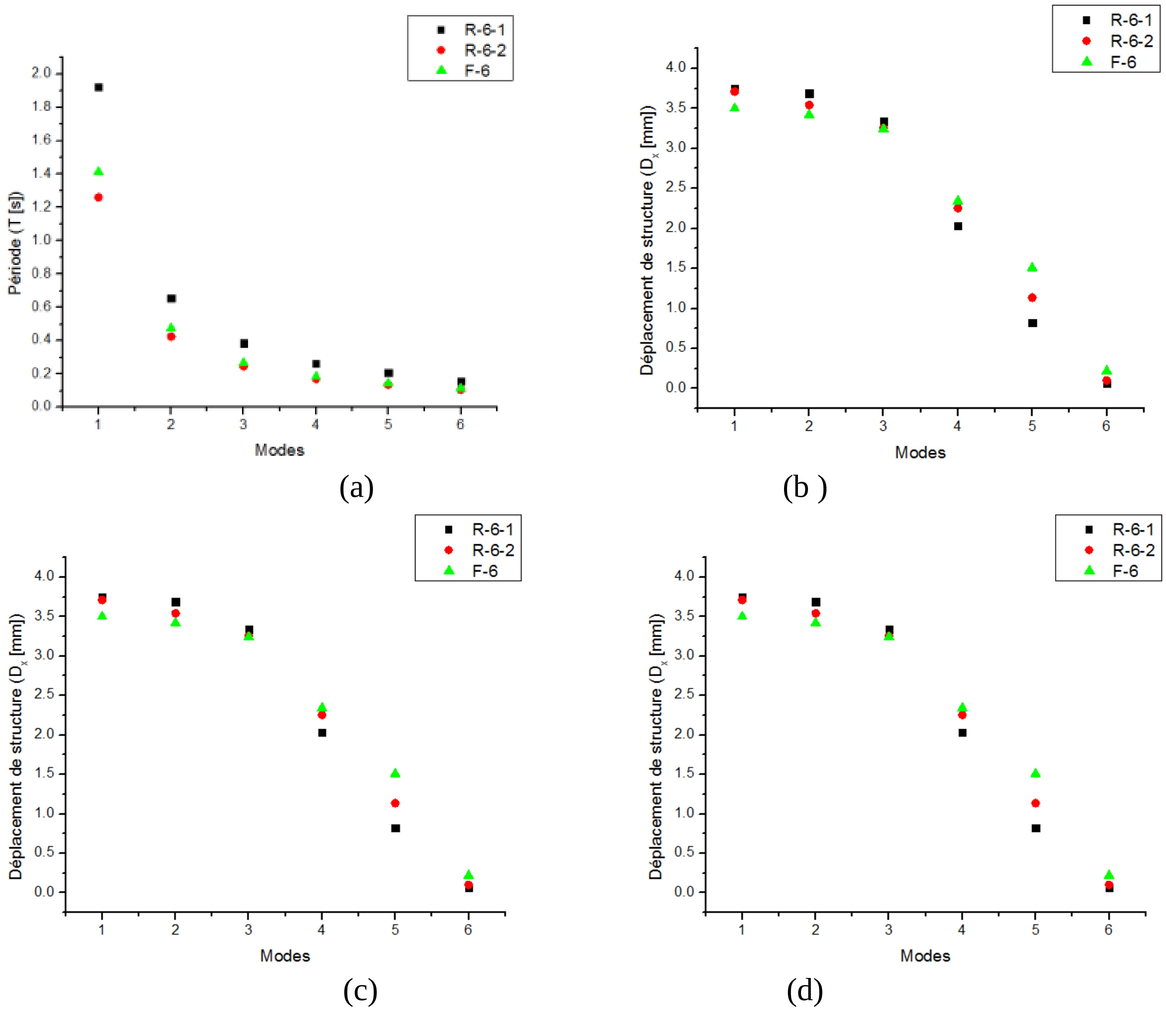


Figure 4. 2. Results of modal analysis of six-storeys structures (LUSAS).

4.5.3.2 Results of seismic analysis

- **General**

In this study, the seismic behaviour of different structures is elastic linear is investigated. The geometric non-linearity or P-Delta effects according to RPA and American codes or second order effects according to EC8 which occurs in each structure where the elements are subjected to axial loads. This effect is closely related to the value of the applied axial force (P) and the displacement (Delta).

The value of the P-delta effect depends on:

(i) The value of the applied axial force;

(ii) The rigidity of the overall structure;

(iii) The rigidity of the elements of one of two types of linear elastic analysis can be used: the "lateral force analysis method" and the "modal analysis using the response spectrum" which is applicable to all types of buildings.

- **Discussion and conclusions**

An overall observation on the obtained results of the elastic seismic behaviour of the cases studied reveals, as far as elastic linear analysis is concerned, that the values given by RPA are always higher than those given by EC8, which is true either using the equivalent static method or the spectral method that values extracted from RPA99 are higher values for both methods. This difference is about 30% in (MSE), and about 50% in (AS) for 4-storey structures, which rises to 37% (MSE) and 60% (AS) for 6-storeyed structures and drops to 35% (MSE) and 50% (AS) for 8-storeyed structures. It can be said, that RPA99 values are very conservative compared to EC8's one. In Figures 4.3 to 4.6 is plotted for four storeys structures including the results of both seismic methods in terms of storey level vs. horizontal displacements, (a) for R1; (b) for R2 and (c) for F respectively. The first three histograms display the variation of the seismic effort per storey level; the last three to represent the variation in floor displacement. It can also be noticed that the second order effects of all the structures studied are found in the upper storeys and decreases downwards, to get closer to the ground floor level. It has to be mentioned that the effect of an irregular frames will be fully detailed analysis will be performed at the last section of this chapter (4.9).in this chapter, i.e. section 4.9. It has been noticed that for six-levels structure, the seismic force at the top is not necessarily the maximum effort, this is the essentially the result of the structural slenderness. The P-delta effect decreases with the increase in the L/H ratio. However, it should be mentioned that the effect for the transition from the (L/H) ratio from 4/3 to 2 had a notable influence on seismic forces of the stage, especially for the upper stages of s structures and that this variation is almost constant, and that the number of stages of the structures does not have a direct influence on the differences.

This investigated highlighted the effect of some parameters considered in the analysis of MRFs frames. The seismic evaluation of RPA99 version 2003 compared to EC8's have shown conservative values for RPA99 provisions in both elastic analyses, which is in fact on the safe side. This difference may be attributed to R coefficient remains higher in RPA99 and is related to the period and the ductility ratio of the structure **(Aribert 2011, Plumier 2010).** The aspect ratio has a definite influence on the overall elastic seismic behaviour of the structures in terms of forces and displacements. As expected, outcomes of the LFM of all the structures studied showed greater values in the range of 30% to 60 % depending on the number of storeys considered compared to MSM. The effect of flexible floor (soft-weak storey) losses of its importance as structures are getting higher (mid to high -rise structures). However, the displacements on the ground floor for flexible structures are naturally greater than those of regular structures, but this tendency is reversed for the higher modes and this according to the number of storeys. Unlike those on the ground floor, the displacements at the top of the irregular structures are less than those of the regular structures, and also reverse in the same higher mode to become larger.

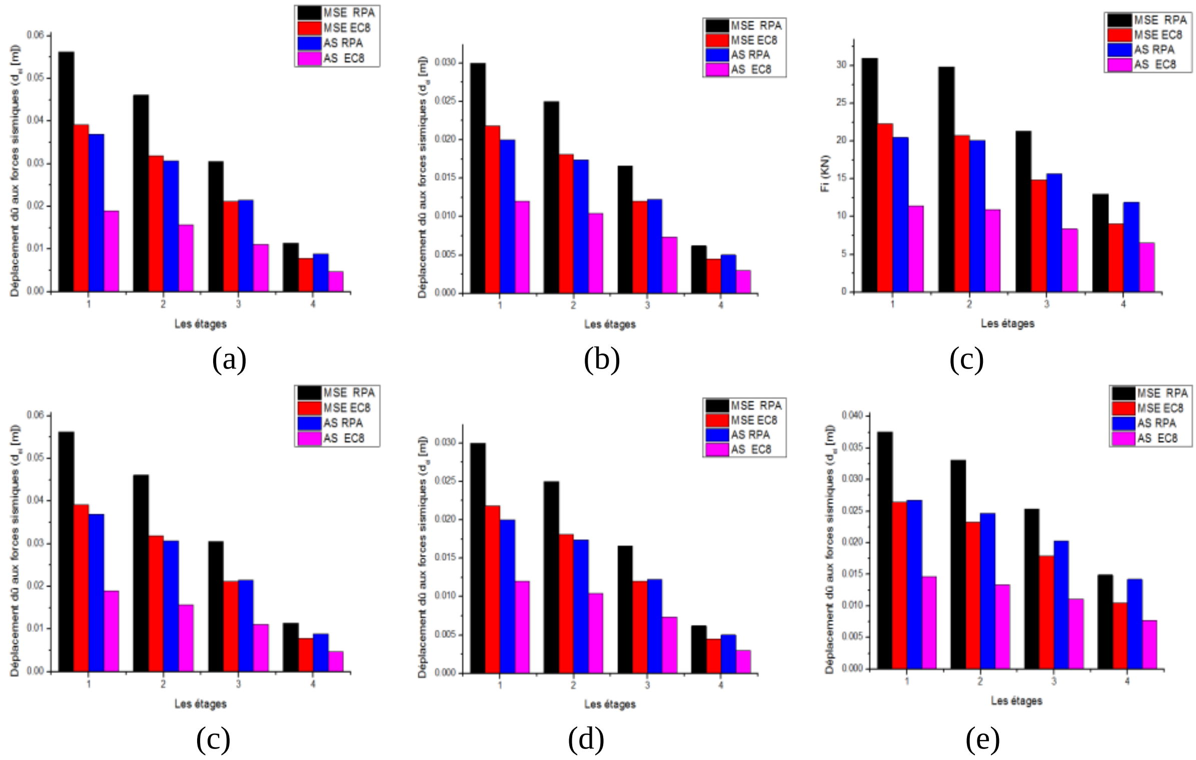


Figure 4. 3. Seismic analysis results for four storeys structures**.**

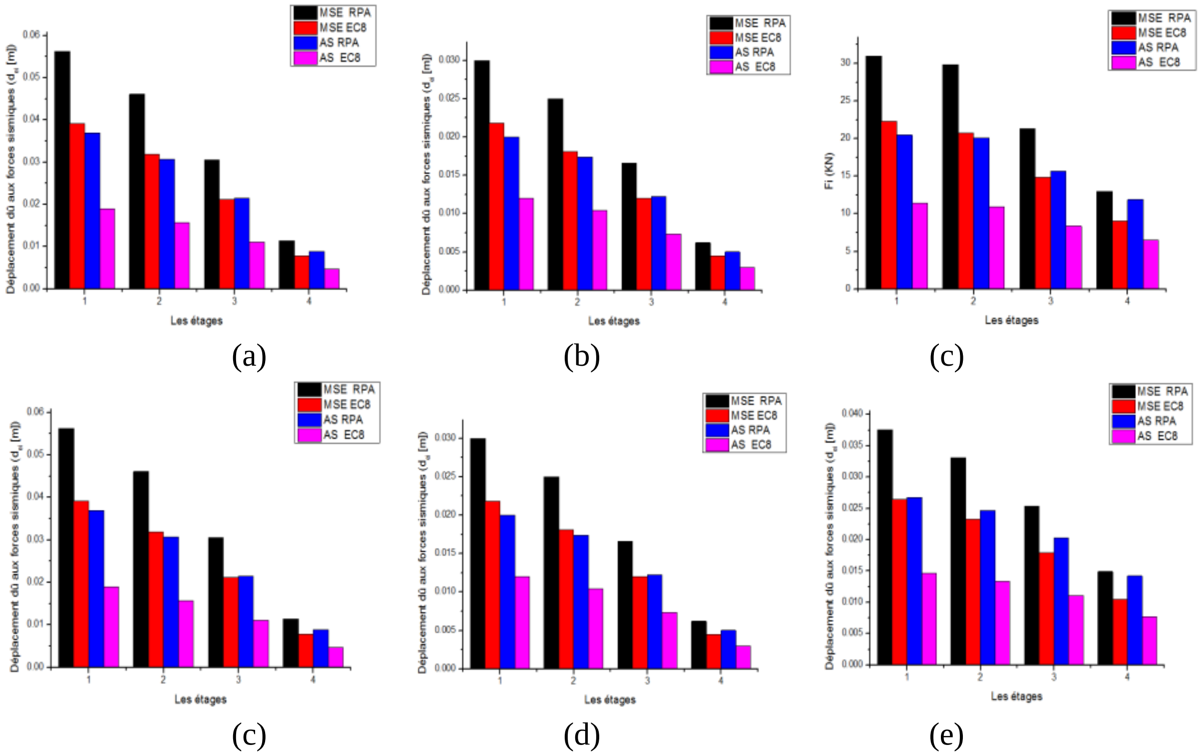


Figure 4. 4. Results of comparative seismic analysis of four storeys structures.

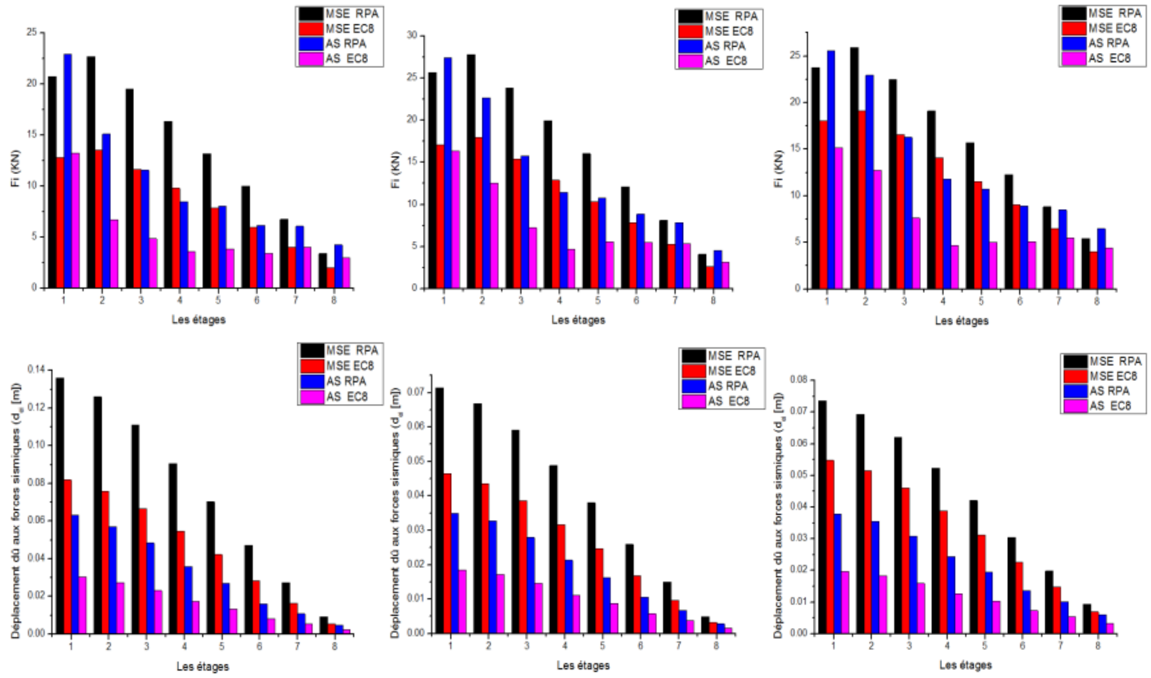


Figure 4. 5. Results of comparative seismic analysis of six storeys structures.

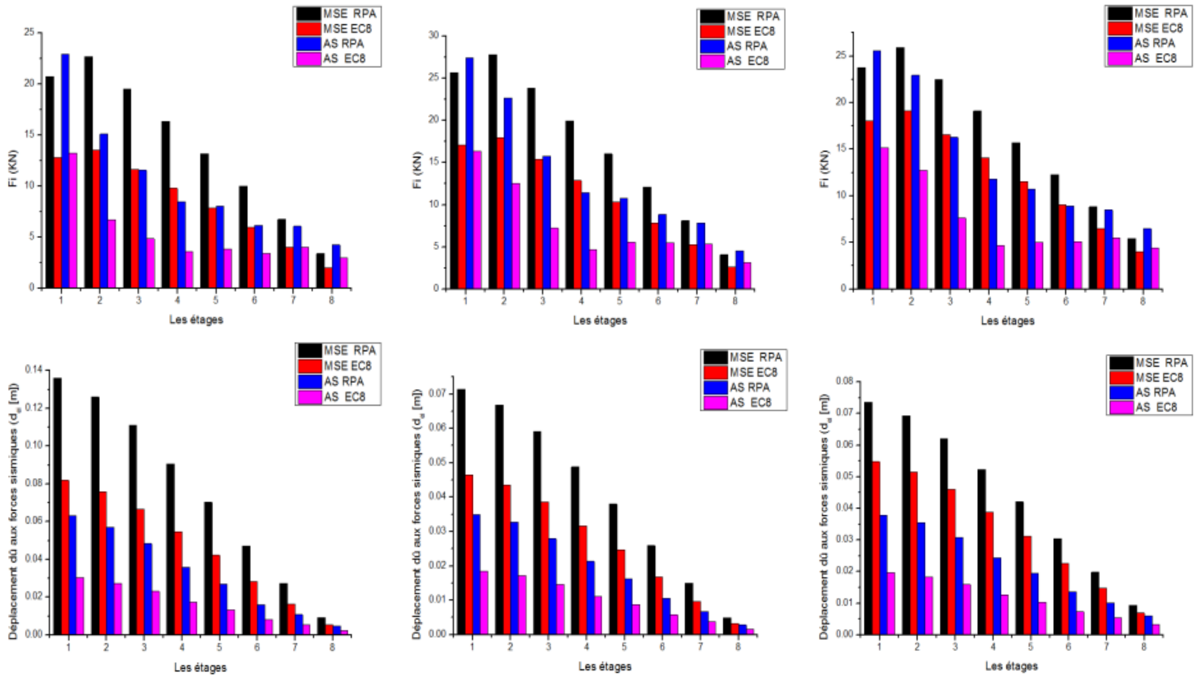


Figure 4. 6. Results of comparative seismic analysis of eight storeys structures.

It must be reminder that the case of soft/weak storeys structures will be fully in a separated section (4.9) including linear and nonlinear behaviours. When the aspect ratio (L/H) increases from 4/3 to 2, this leads to a notable influence on seismic forces at each storey level, especially for the upper floors and that the variation is almost constant. Second order effects have been noticed in the upper floors and decreases lower storeys, and depending on (L/H) ratio with a decrease of P-delta when the ratio increases. The seismic force at the top is not necessarily the maximum effort, as it is in RPA99.

4.6 PARAMETRIC INVESTIGATION OF LINEAR SEISMIC PERFORMANCE OF DUAL MRFS-CBFS MULTI-STOREYS FRAMES

4.6.1 Introduction

- **Scope**

Moment-resisting MRFs and concentrically braced frames CBFs have been widely used for earthquake resistant steel buildings. However, seismic design of steel building structures has undergone significant changes in the last decade. There have been changes in concepts of different countries seismic design codes. Therefore, it seems useful to compare some of these seismic codes. The MRF-CBF dual system constitutes a reliable alternative for designer as they combine the advantages of two basic structural topologies i.e. MRF and CBF. The different bracings were utilized as a means of reinforcing MRF with adequate lateral stiffness. This work aims to study the seismic behaviour of dual braced systems MRF-CBF with X and IV chevron in accordance with the provisions of RPA99 version 2003 and European Seismic Provisions for Structural Buildings EC8 version 2004. The study was carried out using different seismic codes under similar circumstances provisions. These models were designed in accordance with the Weak beam/ Strong columns principle for DCM class of ductility. Also, the statement of RPA99 relative to the height limitation, and consequently, the number of the storey to be allowed for the steel building concentrically braced and unbraced frames must be studied.

- **Problematic**

The work described in this section aimed at investigating and comparing, through a parametric elastic analysis, the seismic behaviour of various types of MRF-CBF dual systems according to the provisions of the seismic codes RPA and EC8. For this reason, two types of dual concentrically braced systems were considered: X-bracing, Inverted Chevron CBF with different configuration were analysed. The performance of typical braced frames concentric braced frames CBFs designed according to current requirements of the Algerians codes RPA99 and CCM97 and the European codes EC3 and EC8 are evaluated throughout a linear analysis. The effectiveness of alternate bracing as an economical solution is also examined and appears to be very intersecting solution especially for structure with reduced number of storeys. The different parameters considered are the slenderness ratio λ of structures with four different heights levels with a single bay, (L/H) ratio in which L is the span length of the beam, and H storey height with different diagonals topologies which are modelled and analysed using SAP2000 and LUSAS commercial packages. The numerical modelling using the general finite elements packages SAP 2000 and LUSAS described herein consist of elastic static and dynamic linear analyses for frames with regular single span for 4, 6, 8 and 10 floors structures associated concentrically braced in four different configurations. The comparative numerical analysis is conducted for a various range of structures with different bracing topology system. As previously done for MFRs structures, an effort is made to accurately evaluate RPA 99 provisions regarding elastic analyses by comparison with the EC8 provision in order to a arrive at conclusive remarks to improve the RPA provisions. Similarly, to the study performed in the above section, a modal eigen analysis for all structures was carried out. Furthermore, the height of building restrictions limited by the RPA, especially the statement of 17 m for MRFs and 33 m for fully centrically braced structures CBFs has been investigated and discussed.

4.6.2 Design of models

- **General considerations**

A two-dimensional model of the structure has been created to undertaken the linear analysis. Beam-to-column connections are modelled as rigid joints as per RPA99 provisions.

The Equivalent Static Method (Lateral Force Method for EC8) can be used for structures which respond essentially in one single mode of vibration and the Multi-modal Spectral elastic Analysis. It becomes clear that by using RPA99 for steel braced structures some major imprecision occurs. Conversely, CBFs provide the best solution regarding the limitation of the inter-storey drift demand under seismic event having a return period comparable with the lifetime of the structure **(Mazzolani 1994).** As expected, capacity design procedures are more directly associated with the ultimate limit state, but a number of checks are included to ensure compliance with serviceability conditions **(Ghazouli 2009).**

- **Characteristics of the structures studied**

In this study, two-dimensional single bay frames with regular storey height of 3m and variable beam span length (6 m and 4.5 m respectively). Different slenderness structure ratio λ is used. The building models with 4, 6, 8, and 10 stories used in this study correspond to MRF-CBF dual systems with X and Chevron IV diagonal having the same mass, evaluated in exactly the same manner as it was in the previous section, acting at all floors. The values of seismic masses are 13482 kg and 16436 kg for the top floor and the remaining of storeys respectively for the case of L/H = 1.5. For the case of L/H = 2, the values of masses are 22680 kg and 26820 for the top floor and the rest storeys respectively. All structures are designed, based upon SCWB principle, to meet with CCM97, RPA99 Algerian codes and EC3 and EC8 European codes provisions respectively.

The geometrical schemes of the analysed systems, which will be named: UB, FBX, ABOX, ABEX, FBIV, ABOIV and ABEIV, are detailed bellow. As example UB structures correspond to MRFs, the FBX – IV with full bracing (X and IV), the ABO and ABE are systems with an alternate bracing odd and even respectively.

- **Parameters of analysis and notations**

The parameters selected in this study are:

- The ratio l/h= 2 and 1.5 where l = span of beam and h = storey height
- The number of storeys: N = 4; 6, 8,10

- The coefficient of aspect characterizing the flexibility of the structure:
- For L/H = 2: the slenderness of the whole structure λ = 2, 3,4, 5
- or H/L = 1.5: λ = 2.66; 4; 5.33; 6.66
- The ductility of the structures: DCM
- The concentrically bracings system: X and inverted V
- Configuration of the bracing: Full bracing FB (X, V reversed chevron); alternately bracings ABE (E for even) and ABO (O for odd).

The factor used to reduce the elastic seismic forces to obtain the design seismic forces is known as the structural response modification factor, which is defined explicitly in the code for each particular system, i.e. R = 4 for MFR and CBFIV and R= 5 for CBFX an. And single value q=4 in EC8 for all structures.

- **Geometrical and mechanical data of the considered structures**

All cases considered in this study are single bay multi-storeys structures. The models were designed in accordance with the Weak beam/ Strong columns principle suggested by the modern seismic codes and for DCM class of ductility. All sections taken for members of all structures were of class 1. The grade of steel used is S235.

The design process has led: For L/H = 1.5 aspect ratio: IPE300 for top floor beam, and IPE360 for the rest., the columns are HEB200 for the top floor up to HEB300for the first floor. For the case L/H = 2, IPE 400 for top floor beam and IPE360 for the other storeys, the columns sections are HEB240 (top floor) up to HEB500 (first floor).

As far as the design of diagonals is concerned, a common design difficulty relates to the effective length that should be used when proportioning the braces for compression. For bracing members in a X configuration, the brace non-dimensional slenderness values suggested by (EN 1993-1-1) is ranging from 1.3 $\leq\lambda\leq$ 2.0 (clause 6.8.3(1)). The lower bound value is imposed in order to limit the maximum compressive axial force transmitted to the columns. Indeed, in the simplified calculation model, the relevant force distribution is not realistic both for the pre-buckling and the post-buckling stages. The concentrically braced structures with X or chevron V used in this study are designed using the non-buckling criterion with a value of relative slenderness an average of $\overline{\lambda}$ = 1.5 **(Aribert 2011)**. Thus, and for a simplification purpose, the same cross section, regardless the number of storeys of the structure has been used for diagonals in X that is L150 x150 x12 and L100 X 100 x12 for the chevron diagonals.

In this study, there are many possible topologies considered, i.e. with and without additional bracings, full and alternately bracings in both cases of X and Chevron bracings. Moreover, the sections are the common Euro sections (IPE for beams, HEB for columns and L for bracings).

- **Seismic data**

According to RPA

Seismic Zones IIa and IIb: moderate seismicity; Group 2: Current constructions or those of moderate importance; Category S2: (firm soil).; Type of structures: concentrically in X and chevron IV; Ductility DCM: Coefficient of Behaviour R = 4 (MRFs), 5 (FBX) and 4.5 for ABOX and ABEX. And R = 4 for the whole of the structures in chevron IV.

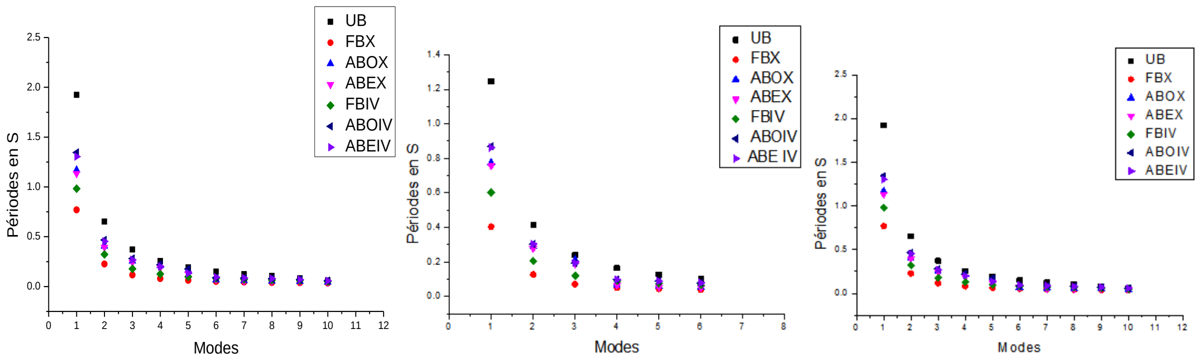
According to EC8

Seismic zone IV: medium seismicity; $a_{gr.}$ = 1.6; Ground classification B, S = 1.35 (Type 2), 1.2 (Type1); Category of importance II; Type II of spectrum of calculation T_B = 0.15; T_C = 0.5; T_D = 2. Coefficient of behaviour q = 4 (DCM).

4.6.3 Results and discussion

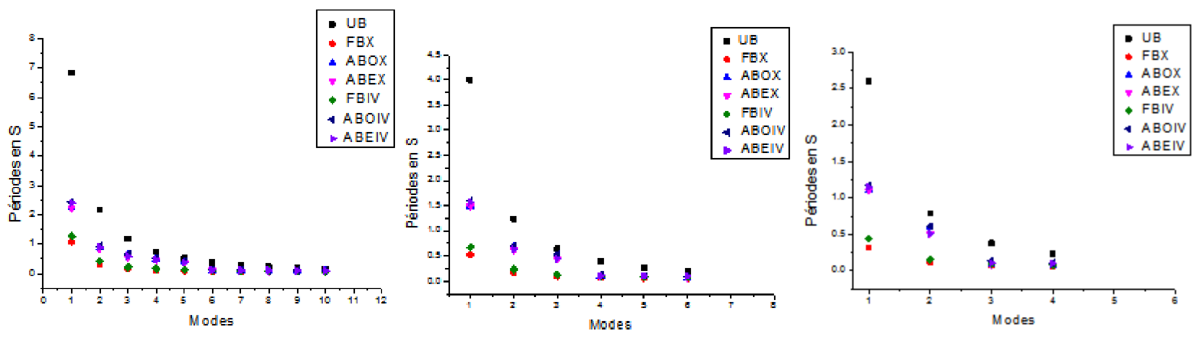
- **Modal analysis**

The complete analysis was performed on the structures with 4, 6, 8 and 10 storeys with all configurations. In the following, only some of the obtained results relating to structures 4, 6 and 10 storeys will be presented. A brief discussion of results is given in the following. Some of the obtained results are depicted in Figure 4.7 and 4.8.



(a) (b) (c)

Figure 4. 7. Period vs. Modes for 4, 6 and 10 storeys respectively (l/h = 2).



(a) (b) (c)

Figure 4. 8. Period vs. Modes for 4, 6 and 10 storeys respectively (l/h = 1.5).

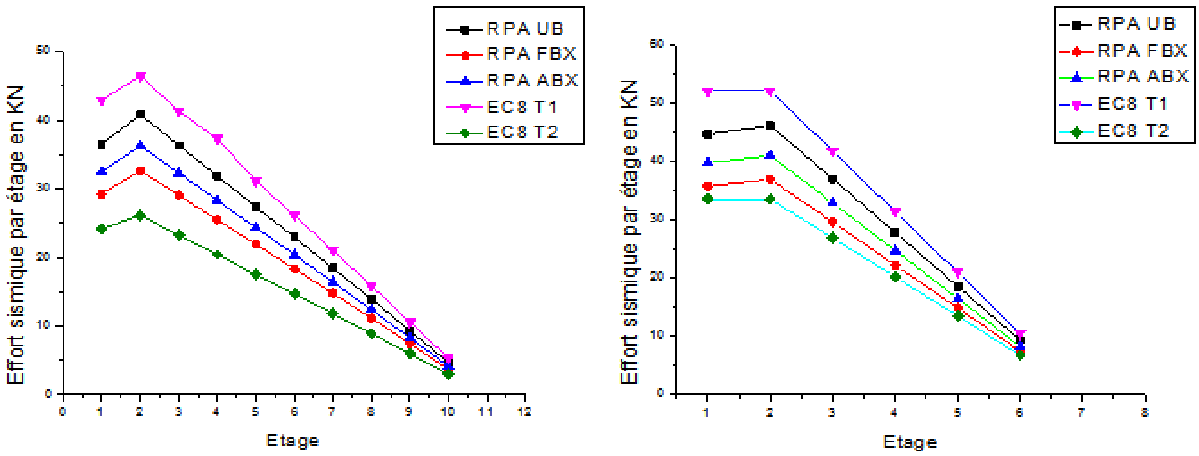
Model analysis was performed using LUSAS v.14 and SAP2000 v.14 yielded in almost similar dynamic properties although the mass modelling is quite different. As expected, modal analysis results show that introducing bracing element have very important effect on structural behaviour. In addition, other parameter i.e. the slenderness ratio (L/H) have sensibly affected the shape modal results in terms of period and displacements. The variation of mode shapes has the same geometric pace for the various structures considered in this study. Period values for all structures are higher when l/h = 1.5 compared to l/h = 2. As predictable, the structures in X bracing are more rigid in free vibration than those with chevron IV. The periods of the IV are larger and twice as much for the FBIV/FBX. The use of alternately bracings reduces considerably the period especially for low structures. It has been found and for all structures that the participating mass reaches the 90% of the total effective mass in the structure stipulated by the seismic codes (RPA and EC8).

- **Discussion of the elastic seismic analysis results**

The seismic effects associated other effects of actions present in the seismic situation of calculation can be analysed on the basis of linear elastic behaviour of the structure. According to the structural descriptions of the structures, two types of linear elastic analysis have been conducted: The Equivalent Static Method (Lateral Force Method for EC8) Multi-modal spectral Analysis using the response spectrum. Obviously, it is not possible to present all the results obtained in this study. However, the discussion will be extended to all cases studied including the case of l/h =2 and 1.5. Once again, a brief discussion of the results obtained in this analyse is given, using the LFM and SMA respectively.

- **According to LFM**

Referring to Figures 4.9 and 4.10 for the case of dual structures with X and IV respectively, an important observation on the outcomings is that the values of seismic forces by the application of the RPA provisions are always laying between the predictions of EC8 type 1 and type 2. This is probably due to the value of q(R) factor being constant in EC8 regardless the number of storeys and structures typologies. As expected, bracing systems for dual structures gives additional lateral stiffness to the structures considerably even for the cases where only alternative bracings were added, especially the low-rise structures. However, this is not the case for high rising structures where the reinforcement (X and IV) loses its influence (Figures 4.9 and 4.10).



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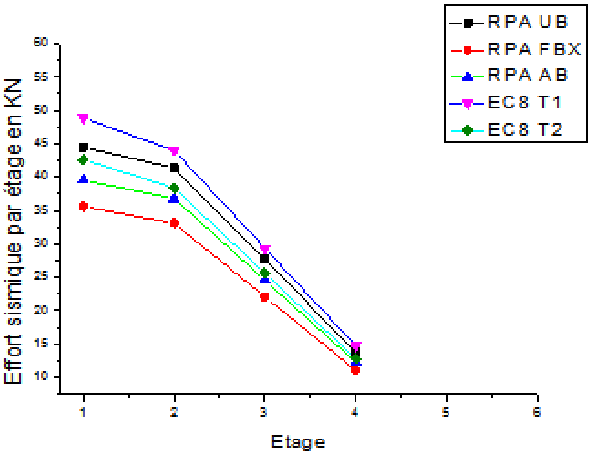

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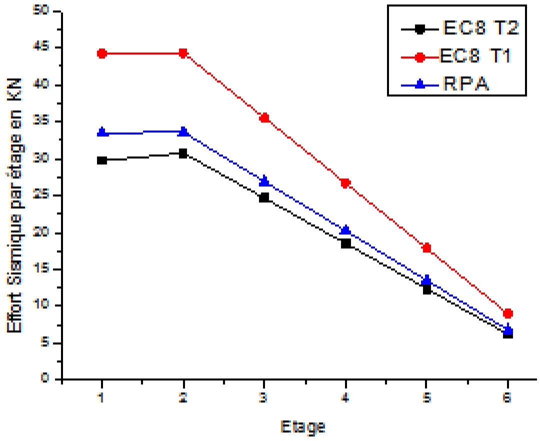
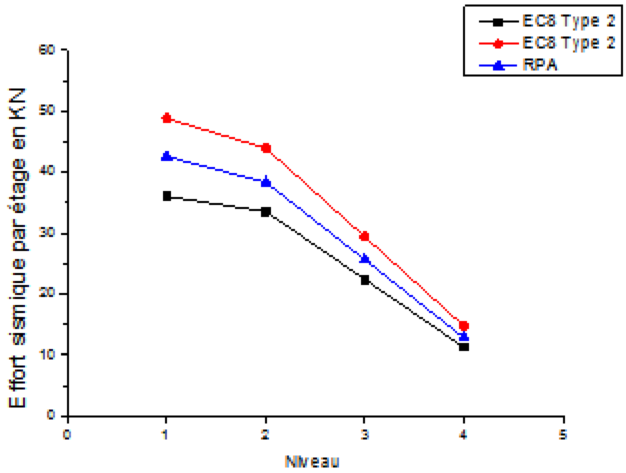
Figure 4. 9. Comparison of RPA and EC8's results in terms of storey forces obtained for structures with 4, 6 and 10 storeys with X bracing l/h=2 respectively.



(a)



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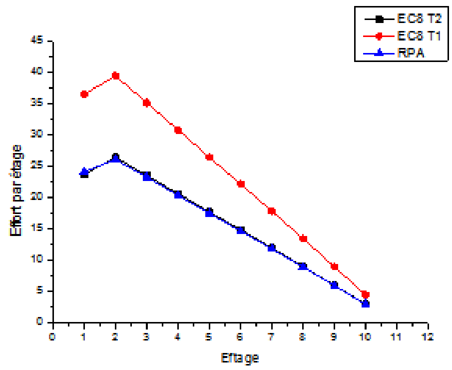


(c)

Figure 4. 10. Comparison of RPA and EC8's results in terms of storey forces obtained for structures with 4, 6 and 10 levels with IV bracing L/H=2 respectively.

Using the LFM and with regard to displacements of storey, it can be seen that the geometric displacements curves are the almost the same for all structures regardless their configurations. Exception is made for structures with alternate bracing for both X and IV bracings in which an intersection of the displacement curves occurs. The comparison of predictions of RPA99 d EC8 with X and IV bracings for (l/h) =2 are displayed in Figures 4.11 and 4.12 respectively.

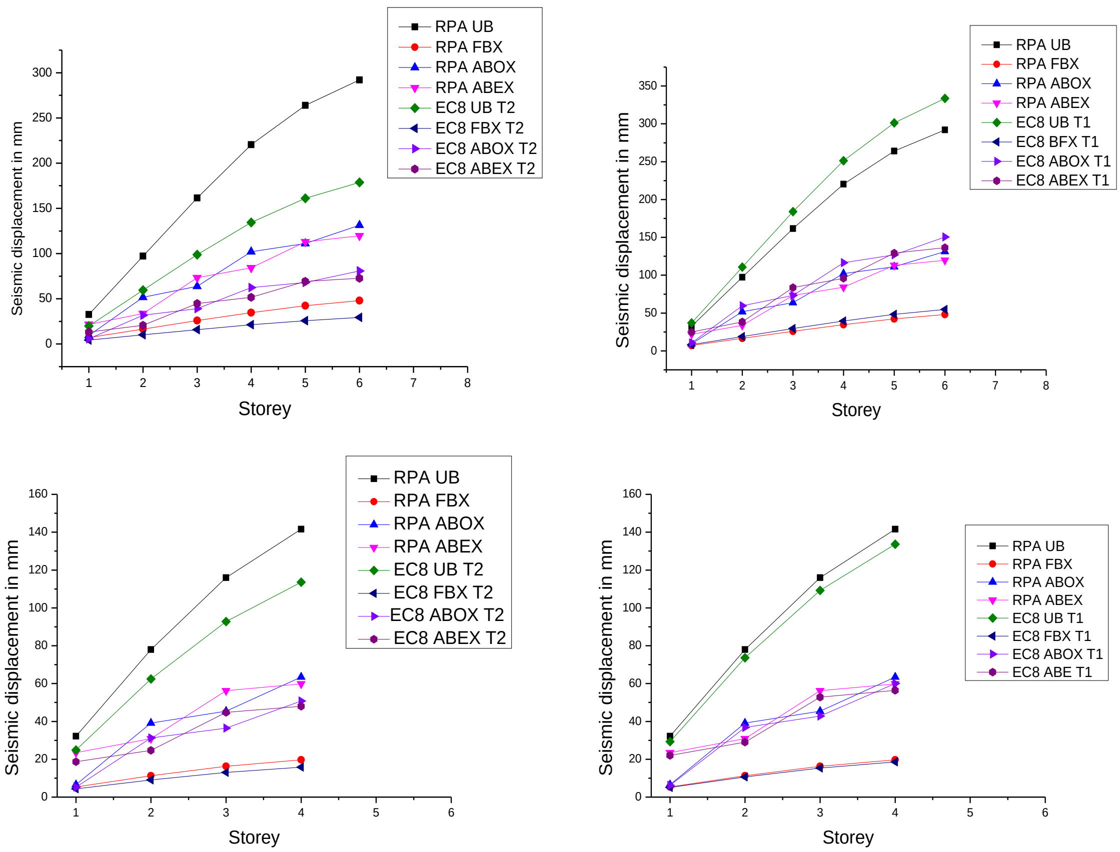
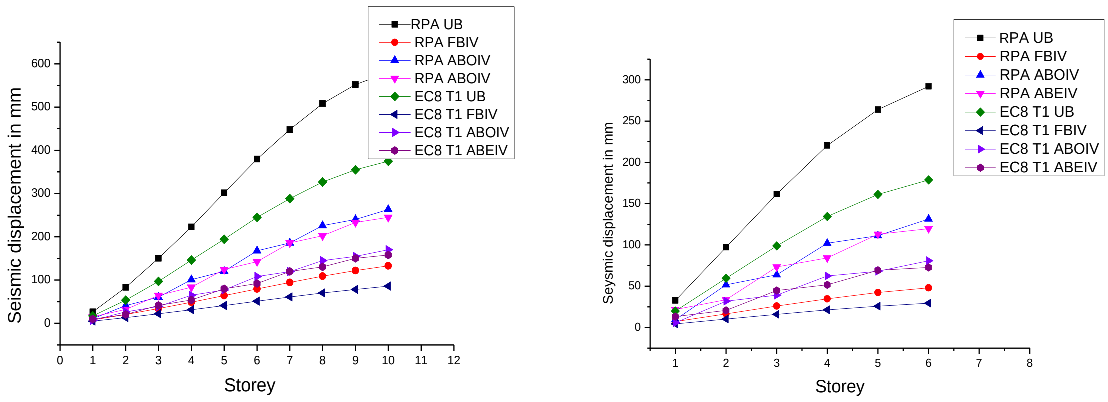


Figure 4. 11 Comparison of the variations of seismic displacements by storey for structures with 4, 6 and 10 levels with X bracing l/h=2 according to RPA and EC8.



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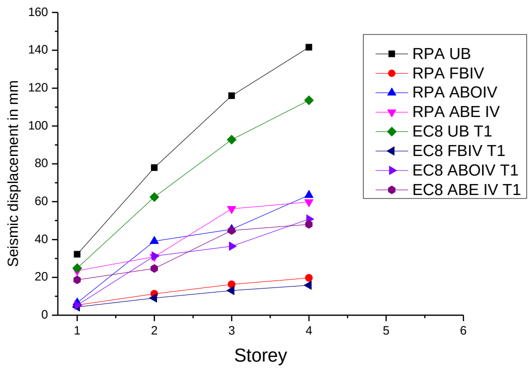


(c)

Figure 4. 12 Comparison of the variations of seismic displacements by storey for structures with 4, 6 and 10 levels with IV bracing l/h = 2 according to RPA and EC8.

Contrary to seismic resulting forces, it has been found that the displacements resulting from the application of the RPA LFM method yields in higher values than those given by EC8 (Type 1 and 2), especially for Type 2 where the gap is larger. It is also noticed that for the slender structures the two kind of bracing, X and IV, come close to each other in terms of seismic displacements.

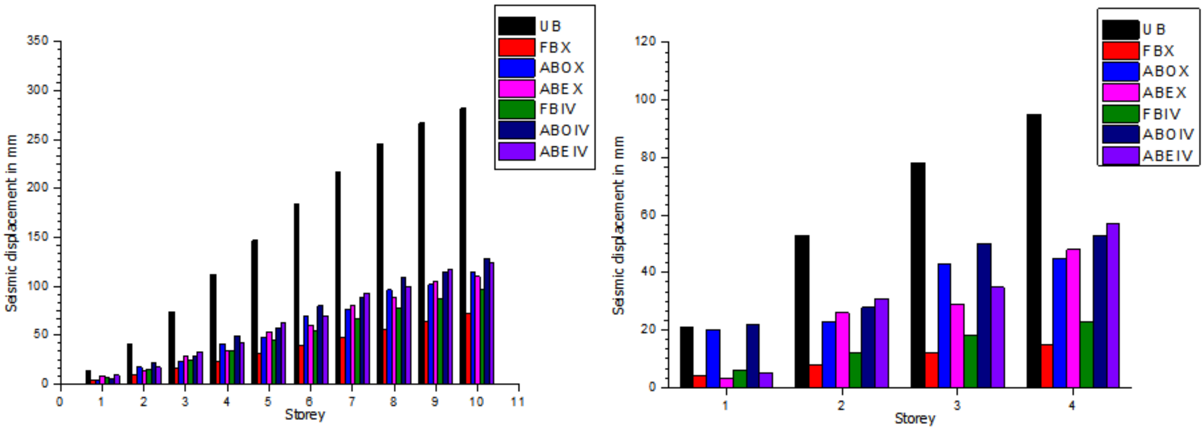
For drift displacements, the Algerian Code is very penalizing compared to the EC8 which decreases the obtained values by a coefficient $\upsilon = 0.5$ for the case of the studied structures and satisfy the relative displacement criterion, and which is not the case for RPA. Thus, RPA condition of 1% of the storey height i.e. 30 mm for the whole of the structures is largely exceeded especially for structures with l/h = 1.5 which leads to check the conditions of geometric nonlinearity, that is P-Delta effect. The implementation of an alternate bracing gives good results by reducing displacements and then the relative displacement, especially for the case of low structures (four storeys frames, for example); however, its influence is declined as the number of storeys increases.

4.6.4 Comparative of results between elastic spectral multimodal analysis and LFM

The spectral analysis was carried out by using SAP2000 NL. In Figures 4.13 to 4.15 the results are plotted in terms of storey number vs. storey lateral displacements. The results are given in following: Figure 4.13 for the variations of seismic displacements at each storey obtained by SMA (RPA) in the case of 4 and 6 structures X-braced for aspect ratio (l/h=2). Predicting results of EC8 provisions are shown in Figure 4.14 to 4.15. Figure 4.14 shows the results considering earthquake type 1 of EC8 in terms of storey displacement from SMA for X and of IV for 4, 6 and 10 respectively. While Figure 4.15 s displays a considering earthquake type 2 of EC8 in terms of storey displacement from SMA for X and of IV for 4, 6 and 10 respectively, with the comparison of the two types of earthquake being shown in Figure 4.15.

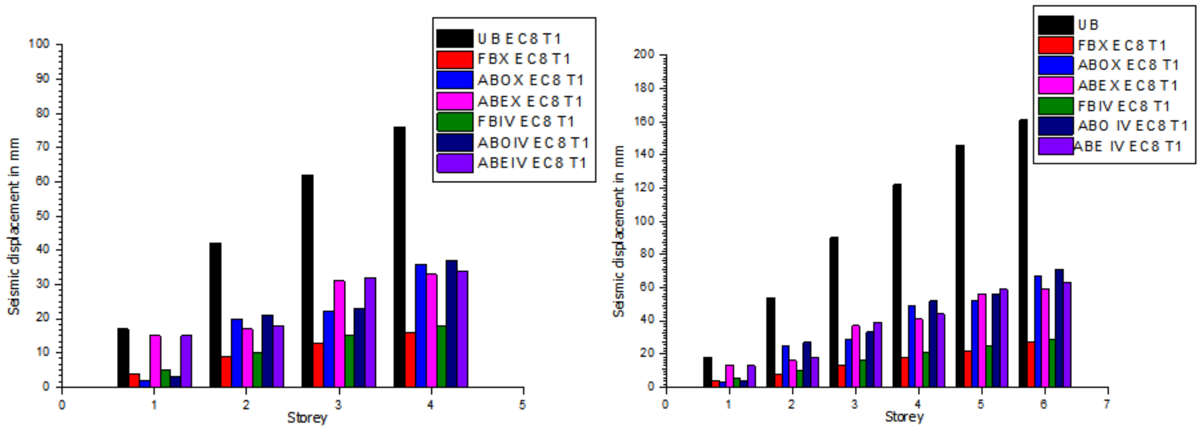
According to this spectral analysis obtained and described herein, it can be observed that the results given by the SMM analysis (RPA and EC8) give almost the same predictions without much difference. Results of the comparative study are presented in Figure 4.15.

They tend to show that the use of X bracing does not have a particular advantage for low structures. It has been founded that the dual structures with IV are slightly stiffer than the use of X bracing, especially for the ratio L/H = 1.5. This tendency starts to be reversed when the storey number increased when it can be seen a higher rigidity for the structures in X. For MRFs, the displacements are more significant for EC8 T1 than those resulting from the RPA. For the structures with IV bracing, and for some configurations the RPA always give slightly higher values of those of EC8 (T1 and T2). For the structures in X, However, for some other configurations, the values of the storey displacement RPA are slightly lower than those of the EC8 especially Type 1. When the structures become higher (8 and 10 levels), the results in terms of displacement are almost identical for type1 and 2 EC8. The deformations resulting from the free analysis give the same forms for a given mode.



(a) (b)

Figure 4. 13. Comparison of the variations of seismic displacements at each storey obtained by SMA (RPA) for structures with 4 and 6 structures with X bracing l/h = 2.



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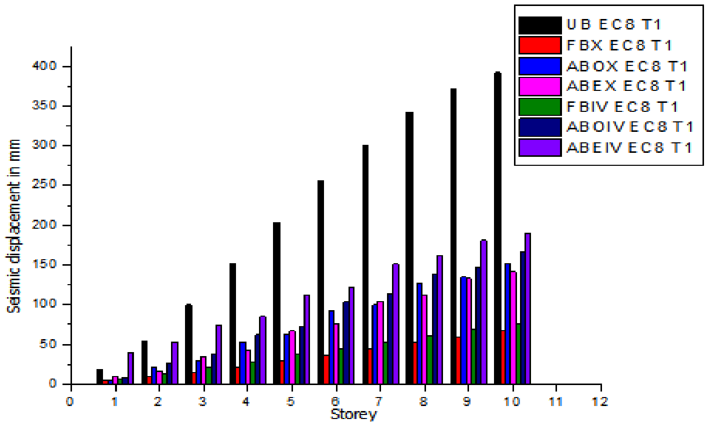
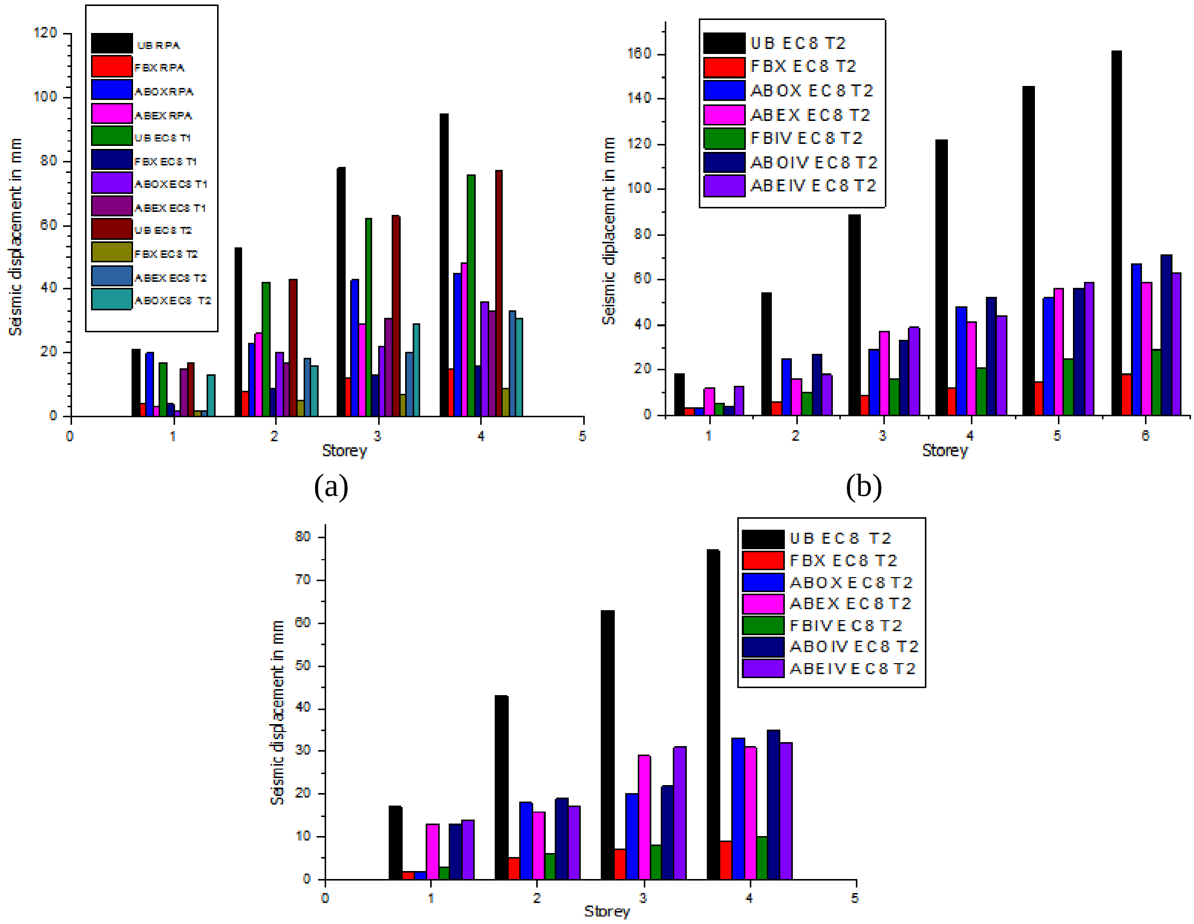


(c)

Figure 4. 14. EC8 Type1 seismic variation of storey displacement storey number from SMA for X and of IV for 4, 6 and 10 respectively.



(a) (b)

(c)

Figure 4. 15. EC8 Type2 seismic variation of storey displacement vs. storey number from SMA for X and of IV for 4, 6 and 10 respectively.

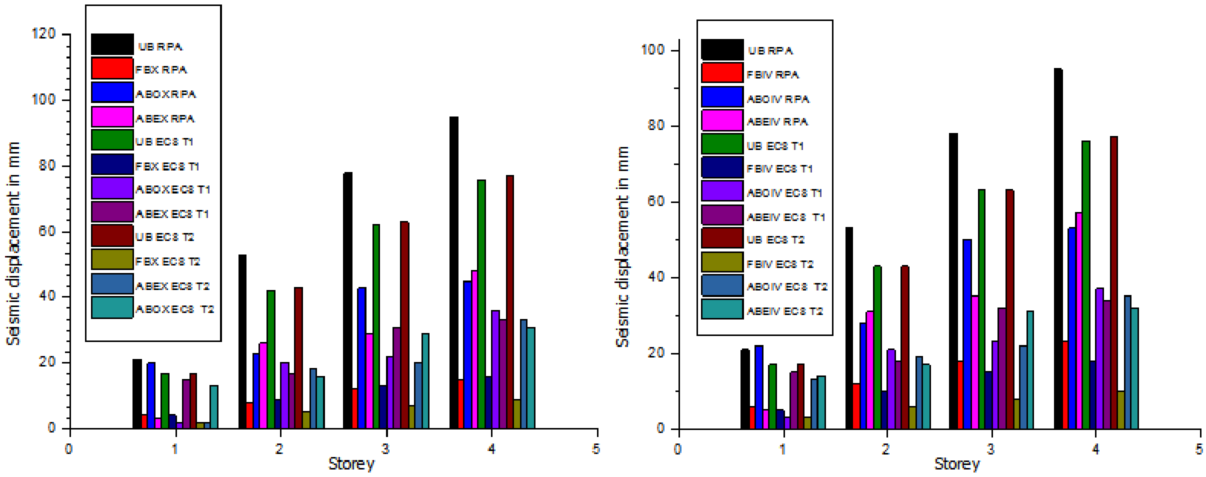


Figure 4. 16. General comparison of RPA99 and EC8 (T1 and T results for the whole of the dual structures in X and IV for 4 floors the structure).

- **Some concluding remarks**

As explicitly shown in Figure 4.16, the slenderness of the structure λ as well as the aspect ratio l/h were examined and are shown to be decisive in the elastic analysis of the structures subjected to earthquake in both seismic codes. The gap in values is not important in predicting when using EC8 Type1 and Type 2. Surprisingly enough, the use of CBFs for slender structures seems to be not necessary for the full and alternates bracings as it does not reduce the displacement and the inter-storey drift and consequently the criteria of inter-storey drift are not satisfied. Also, it has been found that, when the ratio l/h increases, this will give supplementary stiffness to the structures and the recommendations of the RPA on the preference of the bracing in X on IV seems to be correct.

The use LFM analysis has shown that, for both seismic codes, the type of concentrically bracing does not seem to be a determinant factor in the seismic response of the structures for the high-ris structures and losses its importance as structures getting slenderer. Furthermore, the obtained results in terms of forces or displacements from LFM are generally higher than those given by SMA in both RPA and EC8. It also appears worthy to place braces (X or IV) of MRFs structures with alternate concentric bracing for low-rise which yields in a considerable reducing of the displacements and then has a significant economical repercussion. The use of one or other of the alternate concentric bracings is not particularly important. However, as the structures becomes higher their effect decreases significantly.

The seismic storey forces acting at each level given by LFM of the RPA are between the values of EC8 for type1 and 2. Using EC8 provisions, the difference between the values given by LFM for types 1 and 2 appears to be insignificant when the structures become really slender for both cases l/h = 1.5 and 2. It was also noticed that RPA is very conservative when dealing with the limitation of 1% relative displacement especially for l/h = 1.5 and for slender structures, where geometric nonlinearity condition (P-Δ) effects appear to be decisive, which is not the case EC8 which gives a reduction factor of 0.5 of drift- displacement which with satisfaction of the displacements the criterion. The gap is not important in predicting when using for EC8 T1 and T2 and the values are comparable. Furthermore, the seismic forces given by RPA (ELF) are situated between the predictions of the EC8; they are lower than the results of the type 1 and higher than always higher than type 2. This is probably due to the value of the behaviour coefficient R which remains relatively high, fixed and especially does not take into account the fact of analysing slender structures.

4.7 PARAMETRIC INVESTIGATION OF LINEAR SEISMIC BEHAVIOUR OF DUAL MRFs– EBF MULTI-STOREYS FRAMES

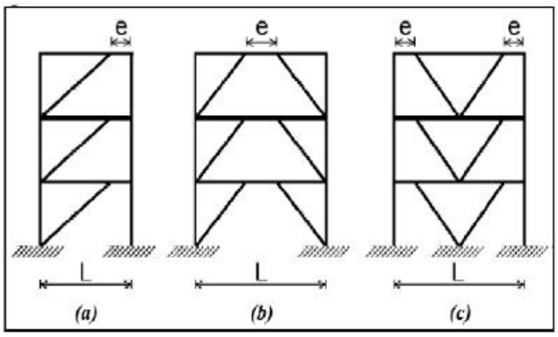
4.7.1 General

In dealing with the RPA99 it can be noticed the lack of information concerning steel structures. Once again, the bracing systems in steel constructions quoted by the RPA are: MRFs and CBFs steel structures only. However, other various types recent bracing systems, such as Eccentrically Braced Frame (EBF), does exist, for long time, in the internationally seismic codes: UBC 97 version 2005, Standard AISC 2002, EC8 version 2004. This bracing system date from the years 1970 and regarded as an optimal solution as it combines at the same time, a sufficient stiffness in order to limit horizontal displacements and has a rather high ductility which makes it possible to ensure a good cycling inelastic behaviour.

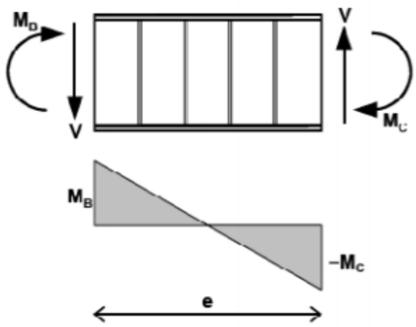
4.7.2 Overview of EBFs

When correctly designed and detailed EBFs behave in a ductile manner through shear or flexural yielding of a link element. The seismic link is designed to remain elastic under low earthquake and to make it possible the structural system to absorb a great quantity of energy during a major earthquake. According to UBC 97 version 2005, Standard AISC 2002, EC8 version 2004 and many other seismic codes, an eccentric braced frame requires at least one end of every brace to frame into a link, see the next chapter for more details. The seismic link is characterized by its length e and of the geometries of the various configurations of the EBFs is shown in Figure 4.17 (a) below.

Link is a segment of an EBF system which behaves as a short beam and on both sides apply shear forces with opposite directions along with the corresponding flexures (see Figure 4.17 (b). The link of an EBF experiences three forces: shear, axial, and flexural. Axial forces have been shown to be negligible for cases where link required axial strength, is marginal compared to nominal axial yield strength. Depending on the length of the link, either shear or flexural forces will dominate failure behaviour. The web of a link should be of single thickness without doubler plate reinforcement and without a hole or penetration.



(a)



(b)

Figure 4. 17 Different configurations of EBF sand internal forces distribution in a link

According to several seismic codes, i.e. EC8, AISC, seismic links are classified into 3 categories according to the type of plastic mechanism developed:

- Short links or shear link which dissipate energy by yielding essentially in shear:

$$e < e_s = 1.6M_{p,link}/V_{p,link}$$

- Long links or flexural link which dissipate energy by yielding essentially in bending:

$$e < e_L = 3M_{p,link}/V_{p,link}$$

- Intermediate links, in which the plastic mechanism involves bending and shear:

$$e_s < e < e_L$$

The theoretical limit between the behaviour dominated by shearing or inflection is based on the simple theory of plasticity for a link in equilibrium, shear and flexural yielding occur respectively. Details of design procedures can be found in EC8 2004, Chapter 5, UBC 1997 in chapter 16 and ANSI/AISC 2002. These statements will be fully discussed in the following chapter of this thesis.

4.7.3 Problematic and scope of the study

In this study, an effort is made to determine the effects of variation in eccentricity on the behaviour of Chevron type EBF for multi-storeys dual steel structures, not covered yet by RPA99, with different slenderness ratios and located at medium seismic region. Hence, the choice of an appropriate eccentricity is of major importance from strength, stability and economic aspects point of view. An evolution can be achieved starting from a classical single bay multi-storey CBF with e = 0 which is in fact MRF frame and passing through MRF-EBF systems, characterised by different dimensions of the link, from short to long one. As in previous studies, the design of considered frames is made by using SCWB principle, considering provisions of CCM97, EC3 and RPA99 and EC8 for a medium class of ductility (DCM). The structures considered are of regular storey height and different span lengths leading to different l/h ratio.

It is well-known that the eccentricity of EBF structures has significant influences on the stiffness, and internal forces, base shear, fundamental frequency and time period and also the member forces and moments and consequently, the mode of failure of frames. Hence, by a suitable choice of eccentricity, a sufficient amount of stiffness from the brace can be retained taking into account the specifications of the Algerian Seismic Code RPA99 and EC8 based on the philosophy of weak beam/strong column in order to obtain highly ductile seismic response. In modern seismic codes (ASCE; UBC97; EC8), a specification of rules is given to ensure that yielding in the bending/shear links of the beam will take place prior to yielding or failure in other members. In this study, elastic seismic analyses were carried out by using SAP2000 NL.

The influence of various parameters on the behaviour of EBFs structures such as the number of stories, the slenderness ratio of structures, the effect of the fraction (l/h) span length to storey height has been also considered. A comparison of the performance of EBFs Chevron structures with the CBFs is made and some concluding remarks are given with some suggestions for further works are also proposed. This study is considered as a preparation one, and give a primary insight of the behaviour of MRF-EBFs frames, for a more advanced analysis as it takes into account both linear and nonlinear behaviours of multi-storeys steel structures described in the following section (4.8).

4.7.4 Structural configuration, modelling and assumptions

- **General**

It is worth to consider that the structural characteristics and detailing of the chosen structures aim to simulate the seismic design situations. The two selected dual EBF/MRF systems herein termed EBF1 and EBF2, differ only on their length link that is, EBF1 (e= 0.1 L), representing short link and EBF2 (e = 0.4 L) for long link. As previously referred, these structures were defined in order to represent simple examples facilitating the presentation of the results and the preliminary suggestion for RPA seismic code which does not cover this kind of structures.

- **Parameters of analysis and notation**

The parameters chosen for the present study are:

- The aspect ratio l/h = 2 and 1.5 where l = span length and h = Height of storey;
- The number of storeys: N = 4; 6, 8,10 the ductility of the structures being DCM;
- The slenderness ratio $\lambda = H_t/L$, where H is the total height of the structure;
- The type structures: MRFs (e = L), CBFIV (e = 0), EBF1 (e = 0.1 L) and EBF2 (e = 0.4 L).
- - Mechanical properties are those of S235 mild steel for the whole of the elements.

There are many possible topologies, i.e. with and without additional bracings, full and alternately bracings in both cases of CBF and EBF Chevron bracings. Moreover, the sections are the common Euro sections (IPE for beams, HEB for columns and L for bracings). It must be mentioned that EBFs are not equipped with any stiffeners. The effect of stiffeners will be fully discussed in subsequent independent chapter under cyclic loading.

- **Seismic Data**

Seismic Zones IIa and IIb: moderate seismicity; Group 2: Current constructions or those of moderate importance; Category S2: (firm site).; Type of structures: CBFs and EBFs chevron IV; Ductility DCM: R = 4 (MRFs), R = 4 for the whole of the structures in chevron IV (CBF and EBF).

- **Geometrical and mechanical data of the structures**

For the development of a numerical model of the frames for linear analysis, the previously presented data is sufficient. All cases considered in this study are single bay multi-storeys structures, see figure 4.18.

In exactly the same way as it was in the above analyses described in sections 4.5 and 4.6, models were designed in accordance with the Weak beam/ Strong columns principle suggested by the modern seismic codes and for DCM class of ductility.

All sections taken for members of all structures were of class 1.

For l/h = 1.5, the top floor beam is IPE300, while IPE360 for other rest floors, the columns are HEB200 for the top floor up to HEB300 for the first floor.

For the case l/h = 2, IPE 400 for the beam of the top floor and IPE360 for the other storeys, the columns sections are HEB240 (top floor) up to HEB500 (first floor).

The dual structures used in this study are concentrically (e=0) and eccentrically with 0< e< L bracing system is IV reversed diagonals, and for a simplicity, these diagonals are dimensioned according to the stability criteria of no-buckling of the compressive diagonal, by taking a and average $1.3 < \bar{\lambda} < 2.0$ being equal to 1.5 **(Aribert 2011)**. A common design difficulty relates to the effective length that should be used when proportioning the braces for compression, what causes to have the same section of the diagonals according to the type of studied bracing some is the number of stages of the structures considered for both CBF and EBF structures. Thus, and once again for a simplification purpose, the same cross section of diagonals, regardless the type of structures, the number of storeys has been used for diagonals with equal angle sectionL100 X 100 x12 for the chevron diagonals (CBF and EBF).

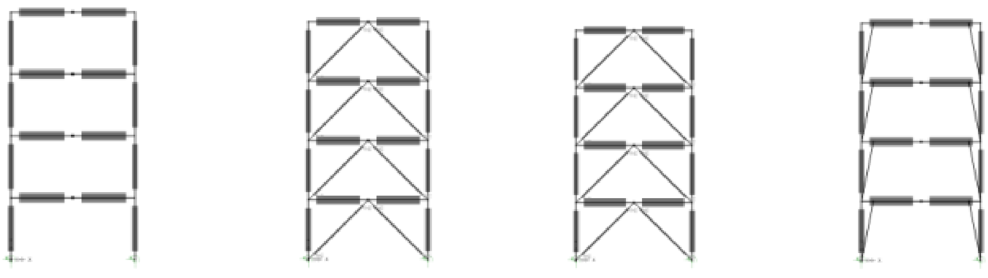


Figure 4. 18. Geometrical schemes of structures: case of four storeys.

4.7.5 Discussion of results

Considering the significant number of parameters of analysis, it is not possible to present the totality of the results obtained in this study and only samples of each part of the analysis will be presented in the following Figure. Figure 4.19 shows the seismic displacement given by SMA for the case of 4 storeys structures and for l/h = 2 and 1.5 respectively. Figure 4.20 depicted the seismic displacement given by SMA for the case of 6 storeys structures and for l/h = 2 and 1.5 respectively. Figure 4.21 present a comparison of results obtained by LFM and SMA for the cases l/h = 2, et 1.5 for the structures with 8 storeys respectively. Figure 4.22 concerns the comparison of inter-storey displacement obtained by LFM and SMA for the cases of l/h = 2, 1.5 for the structures with 6 and 8 storeys respectively. The linear seismic behaviour of several MRF-EBF steel structures is discussed herein in terms of the link eccentricity values. Nevertheless, the discussion will concern the whole of the obtained results. It is pointed out that the methods of analysis are the equivalent static method (LFM) and spectral multi- spectral SMA using the spectrum of RPA.

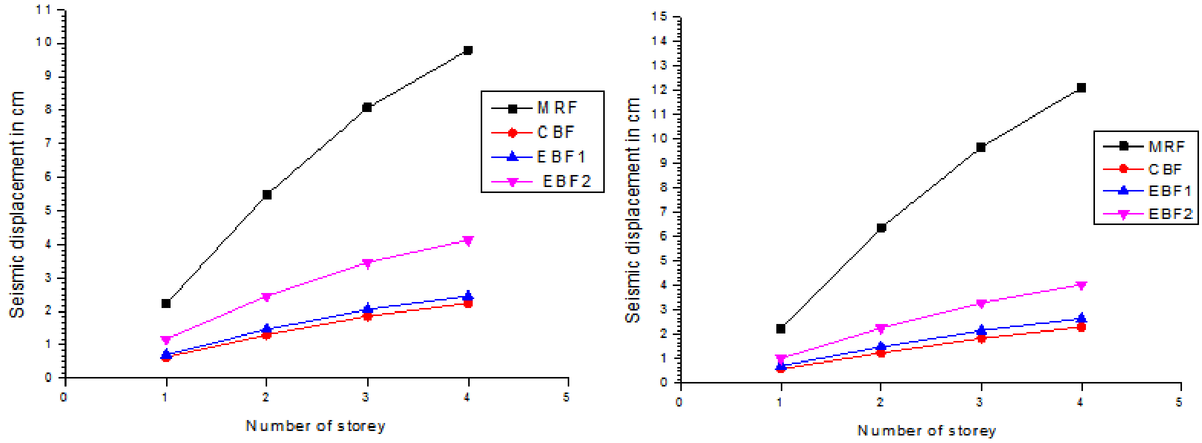


Figure 4. 19. Storey displacement given by SMA for the case of four storeys structures and for ratios (l/h = 2 and 1.5).

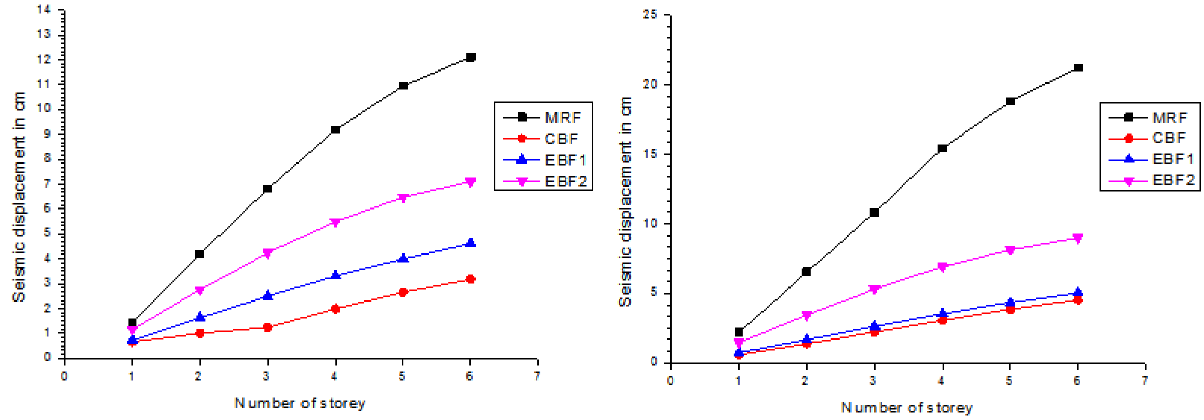


Figure 4. 20. Storey displacement given by SMA for the case of six storeys structures ratios (l/h = 2 and 1.5).

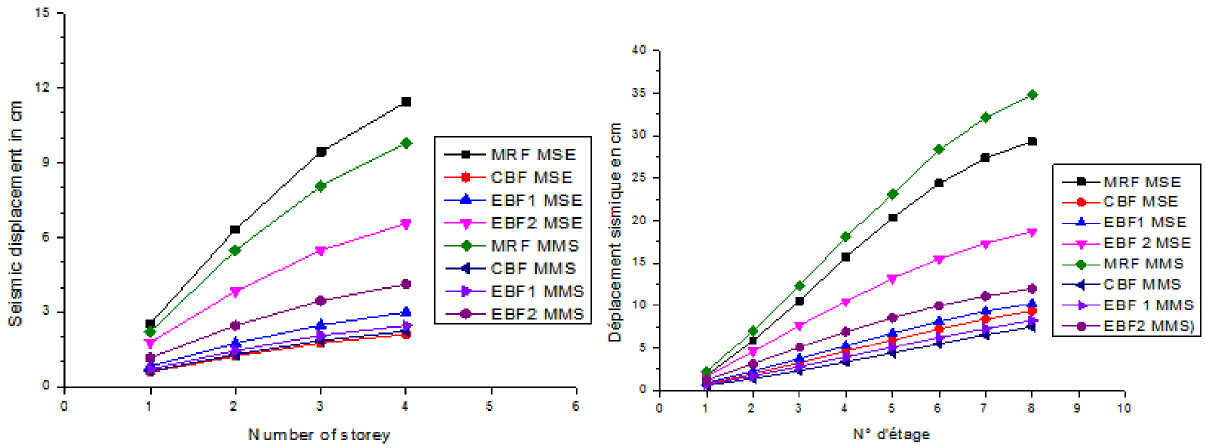


Figure 4. 21. Comparison of results obtained by LFM and SMA for the cases ratios (l/h = 2 and 1.5) for the structures with 8 storeys respectively.

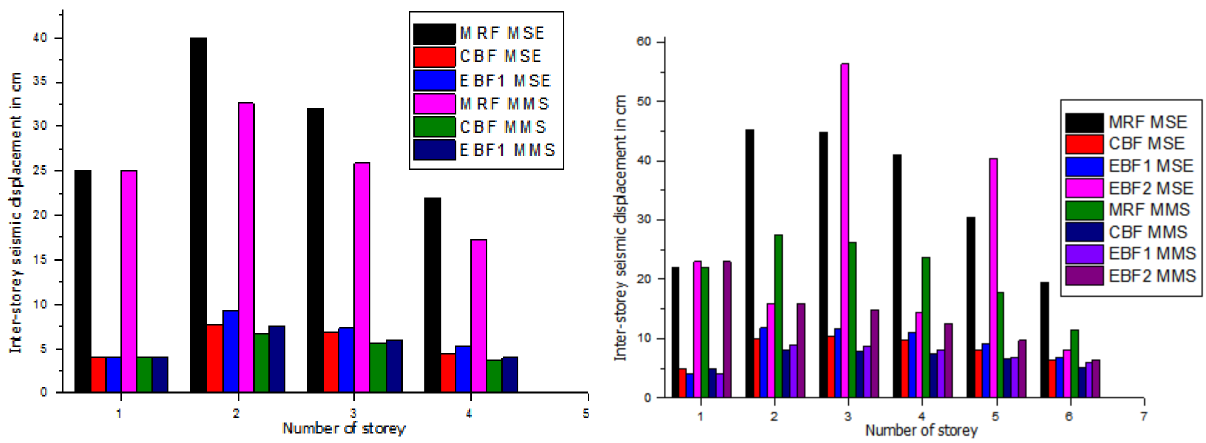


Figure 4. 22. Comparison of inter-storey displacement obtained by LFM and SMA for the cases of ratios (l/h = 2 and 1.5) for structures with 6 and 8 storeys.

The length effect of the seismic links by means of a comparative study on the basis of variation in seismic displacements and drift-displacements is made between MRFs and dual-CBF/EBFs structures taking into account two aspect ratios, i.e. 1.5 and 2.0, and several slenderness ratios λ according to the number of number of storeys.

In fact, the present study showed an incontestable influence of the seismic length for dual MRFs-EBFs on the behaviour, even linear, of the multi-storey structures in terms of displacement, forces acting on the different levels of the structures. The slenderness ratio of structures (L/H) influences considerably the results given by the two methods of analysis. It is also noticed that the predictions of method LFM are always higher than those of SMA and deviate from each other as the number of stage increases.

Referring to Figures 4.19 to 4.22 it can be seen that for structures in EBF1 and EBF2 with l/h = 2, in terms of seismic displacements results given by both methods LFM and SMA are close for EBF1 with short link (e = 0.1L) with 4 storeys, but as the structures gets more storeys number the variation is more significant and becomes between 20% up to 40 %, i.e. for EBF2 (e= 0.4L), the variation reaches 40%. As expected, storey displacements of EBF1 and EBF2 are ranging between those of the structures in CBF and MRF, with drift-storey condition is generally satisfied and does not need any P-Delta effect.

An important remark can be made from Figure 4.19 to 4.22 is that, for the structures with l/h=1.5 and EBF1 and EBF2 topology, values of displacements in all frames are higher than those of the first case i.e. L/h = 2. With regard to drift-storey and for low-rise structures, the restrictions of the RPA seem to be satisfied for the report/ratio L/h = 2 and of a lesser amount for structures with L/h = 1.5. For the EBF1, EBF2, the results of this study show that the RPA99's condition of drift-storey is satisfied for mid-rise structures, which is not the case of EBF2 with long link which resembles to the behaviour of MRF, particularly when the slenderness ration L/H decreases for the structures with 4 and 8 storeys respectively. It is important to mention that it seems that all the links segments (EBF1 or EBF2) are responding in an elastic manner.

4.7.6 Concluding remarks

Behaviour of the eccentrically braced frames can change depending on the length of the link length and other parameters of the structure. According to first results obtained in this study which is, of course preliminary and limited, showed that the use of the dual structures in MRFs-EBFs can be defensible when the seismic link remains elastic under pure shearing, i.e. the EBF1. The results given by LFM and SMA confirm that and at least for the structures small slenderness ratio, the criteria of the RPA concerning the drift-displacement are satisfied. Therefore, the behaviour of EBF1 is acceptable and can be compared to CBF with more ductility and using the same value of R coefficient = 4. This unique value does not take into account the degree of ductility of structures (MRF and CBFIV or even EBF).

For detailed study of linear and nonlinear behaviours of MRF-EBFs compared with MRFs and CBFs frames, will be provided in the following section using both seismic codes, i.e. RPA99 and EC8 respectively.

4.8 COMPARATIVE NONLINEAR ANALYSIS OF DUAL EBFs AND CBFs STEEL STRUCTURES

4.8.1 Scope and objectives of the study

It is well-known that RPA99 methodology and provisions are mainly based upon the knowledge of the seventies and the eighties of the 20th century. As mentioned earlier and for several times, that RPA 99 has not clearly address the way of performing a nonlinear analysis on structures, except the fact that R coefficient takes into account, in some way, the inelastic behaviour of a structure. This was acceptable in many seismic international codes during the seventies and eighties of the last century which is not tolerable at the present time. This is certainly not enough for the time being and with the advance in recent decades in understanding of the linear and nonlinear behaviour of steel structures.

The adoption of inelastic behaviour of structure in many advanced seismic codes throughout the world, it is, no more, acceptable that this concept will not be included in RPA99, which is clearly well behind concerning the of new methods of seismic analysis which have shown a safe use many developed countries. Another aspect of the unjustifiable delay in RPA99 code compared to others in many emerging countries like turkey and Iran in which research works are always publishing results as well as developed countries, is that the EBFs structures are not yet covered in RPA99, which is not acceptable, while they are commonly used and adopted for resisting structures against earthquakes and are proved to be very efficient as they effectively combine the ductility property of moment frames MRFs and the higher stiffness associated with concentrically braced frames CBFs.

In this work, which is represent a preliminary investigation, an attempt is made to assess the nonlinear performance of a series of medium, high rise multi-storeys dual MRF- Chevron EBFs, MRFs and CBFs Chevron steel structures, with medium ductility level DCM, located in moderate seismic regions, through a number of parameters that are thought to affect the seismic response of the dual structures. Also, an humble effort is made to study steel structures designed to RPA99 and to check up its provisions concerning the lateral load and spectrum analyses and it, in fact, a natural continuation to the work described in section 4.5 with an inelastic analysis by a pushover analysis with two lateral loading patterns to comply with RPA99 provisions with comparison of all results with those provided by EC8 recommendations. For the sake of comparison, these structures with regular storey height and single span and different storey number lying from mid to high-rise structures are designed to comply with CCM97, RPA99 (version 2003) and EC specifications and are modelled in SAP2000 NL code. It's worth to mention that all structures were analysed in both elastic with spectrum multi-modal analysis and pushover static nonlinear analysis using both RPA99 and EC8 provisions. The results obtained in this study of linear and with special emphasis on the pushover analysis in terms of capacity curves (RPA99 and EC8) which illustrate the change in stiffness as well as in lateral load-carrying capacity. An analysis of the internal forces in links with different lengths behaviours reveals the real contribution of the latter in taking the seismic loads. Finally, a comparison with the response of MRF-CBFs structures is being given in order to evaluate the performance of MRF-EBFs.

4.8.2 Non-linear Static analysis

Recent advent of performance-based design has pushed the nonlinear static pushover analysis procedure to the forefront. It has gained wide acceptance and is considered a valid alternative (of course with limitations) to dynamic nonlinear analysis of building frames to evaluate likely structural performance of existing and new buildings under seismic conditions. The pushover analysis is a simplified methodology to obtain the structural response to seismic actions through a non-linear static analysis. It is one of the easiest non-linear analyses that can be conducted, while still assessing the non-linear stress-strain relationships for all elements.

Although this method does not have a rigorous theoretical foundation **(Krawinkler 1996),** it is recognised as a useful tool in seismic engineering to obtain nonlinear force–displacement relationship of a structure and thus determine the load-carrying capacity of an inelastic system. Admittedly such a procedure would be an approximate one, but it serves the purpose because it can capture the essential features of inelastic behaviour of a structure and thus has an important bearing in predicting satisfactory performance of the system. This analysis evaluates the performance of the structures through control of its displacements (at local and global levels), still giving information about the ductility and the resistant strength capacity. For analysis, a mathematical model is displaced by monotonically increasing lateral force or displacement, in gradual discrete increments (pushing the structure), until either a target displacement is exceeded or the collapse condition of a building is reached. Pushover analysis is applicable to structures whose response is dominated by its first mode of vibration. In recent years, several methods have been proposed, with limited success, to enhance the capability of this method so that it can adequately account for higher mode effects.

The vertical distribution of the lateral force at each storey level should be in proportion to the fundamental mode shape **(FEMA 2005, 2009)**, which is also one of the two suggested load-patterns in EN 1998-1 **(EC8 2005)**. The other method in EN-1998-1 is an equal load pattern, and both patterns have to be applied. The lateral force is applied until a specified target displacement is reached or, the lateral forces are applied until a loss of 20% of the maximum base shear capacity is achieved **(FEMA 2009)**, at this point the ultimate displacement is reached, this is directly linked to the displacement, the structure were to exhibit due to an earthquake. However, there are some disadvantages with the method, especially since the analysis is static, damping is not included, and neither can other time-dependent dynamic effects be.

When the structure deforms and inelastic behaviour occur, the modal properties change. This will not be included in the SPO, which is unfortunate since the forcing on the building in a SPO-analysis is dependent on the mode shape. For structures dominated by the first mode, POA has satisfying accuracy, but for structures that depends highly on multiple modes, the method is not good enough **(Cemalovic 2015)**. The most complex analysis method is the non-linear dynamic analysis (NDA), which is used mainly in research and for analysis of some important structures. It requires several analyses with different accelerograms and additional data on the mathematical model (hysteretic and damping models). For practical application, pushover-based methods, e.g. the N2 method **(Fajfar 2000)**, implemented in Eurocode 8 – Part 1 **(EC8 part 1, 2004a),** are better suited.

4.8.3 Description and design of the selected structures

- **General**

Although the height of a structure and the structural system are key parameters which can affect the inelastic behaviour and response of the structure, nevertheless these parameters have not been taken into consideration in the current design codes for designing EBFs.

This section provides a summary of the design of the building that is to be analysed in this thesis, and calculations relevant to the seismic design are shown here. It is worth mentioning that structures studied are multi-storeys buildings, made with S235 mild steel with characteristics taken from EC3, and are located in a medium seismic region. The strong-column weak-beam (SC /W B) concept is a global frame concern w here in meeting the SC/W B requirements the columns should be strong enough to force yielding in the beams at multiple levels of the frame before yielding occurs in the columns. The intent is to prevent column failure before beam failure because column failure could produce an unfavourable storey wide mechanism. The mid to high -rise frame set used in this study contains four - to ten-storeys frames having single bay. The ratio of height to least lateral dimension of a building is termed as its slenderness ratio and that of its length divided by width (both in plan) as its aspect ratio. From seismic considerations, slenderness ratio of a building is more important than its absolute height. The larger the slenderness ratio, the greater is the need to check stability against overturning, impact of *P*-4effects, extent of storey drifts and consequence of large axial tensile and compressive forces prevailing in supporting columns.

- **Analysis parameters**

The ratio L/h = 2, with h = 3m regular storey height of the floor and the span length L=6m. e = length of the seismic link.

Types of structures:
Identifications of models to the bracing system: UB; CBF; EBF1 and EBF2.
UB: MRF structure e = L
CBF: Concentrically structure with bracing IV e = 0
EBF1: Eccentrically structure with intermediate link, e = 0.1 L
EBF2: Eccentrically structure with long e = 0.4 L
Number of floors: 4, 6, 8 and 10.
Aspect ratio: the ratio between the total height of the structure, H, and L= length of the span:
λ = H / L = 2, 3, 4 and 5.
Ductility Level: (DCM).
Sections: hot-rolled first-class section (IPE and HEB) (CCM97 and EC3).

Table 4. 5. Sections adopted for the analysed structures**.**

Type of frames	Storey level	Column section	Beam section
Four Storeys	4^{th}	HE 240 B	IPE 400
	3^{rd}	HE 260 B	IPE 450
	2sd	HE 260 B	IPE 450
	1^{st}	HE 280 B	IPE 450
Six storeys	6^{th}	HE 240 B	IPE 400
	5^{th}	HE 260 B	IPE 450

	4th	HE 260 B	IPE 450
	3rd	HE 280 B	IPE 450
	2sd	HE 280 B	IPE 450
	1st	HE 340 B	IPE 450
Eight Storeys	8th	HE 240 B	IPE 400
	7th	HE 260 B	IPE 450
	6th	HE 260 B	IPE 450
	5th	HE 280 B	IPE 450
	4th	HE 280 B	IPE 450
	3rd	HE 340 B	IPE 450
	2sd	HE 340 B	IPE 450
	1st	HE 450 B	IPE 450
Ten storeys	10th	HE 240 B	IPE 400
	9th	HE 260 B	IPE 450
	8th	HE 260 B	IPE 450
	7th	HE 280 B	IPE 450
	6th	HE 280 B	IPE 450
	5th	HE 340 B	IPE 450
	4th	HE 340 B	IPE 450
	3rd	HE 450 B	IPE 450
	2sd	HE 450 B	IPE 450
	1st	HE 500 B	IPE 450

N.B. Braces are double-angle L100 x 100 x12 with A = 55.08 cm² using, $\overline{\lambda}$ = 1.6

$1.3 < \bar{\lambda} \leq 2.0$ where $\bar{\lambda} = \sqrt{\frac{N_{pl}}{N_{cr}}}$

- **Types of analysis**

As will be described alternately in the foregoing sections, three types of analyses were conducted. Several methods of analyses have been used, namely: modal analysis; seismic analysis including: Elastic linear analysis represented by the dynamic model method spectral (according to EC8, and RPA99) and non-linear analysis which is the PUSHOVER (according to EC8, and RPA99). In order to both assess and verify the nonlinear response of structures located in moderate seismic area and having DCM level of ductility; designed to meet the RPA99 provisions, a parameter study is performed using SAP2000 NL.

- **Results and discussion of elastic analyses**

- **General**

In of the purpose of evaluating the capacity of handling the nonlinear analysis, POA, by RPA99 recommendations, the seismic data was adopted from RPA99 and EC8 (2004) respectively for a comparison purpose. Firstly, an elastic study using a spectrum analysis MDOF systems was conducted to compute the seismic demands, namely maximum roof displacement, maximum inter-storey drift ratio and maximum base shear. Secondly an emphasis on the nonlinear static analysis (pushover analysis) was utilised to estimate the load-deformation response of the structures. Using the elastic first mode lateral load pattern dependent capacity curves were obtained. Two different invariant lateral load patterns were applied on the two-dimensional models to reflect the global behaviour of the frames. Two loading patterns are used namely Push 1 for the presentation of the response spectrum in the acceleration vs. displacement and the lateral force/mass vs. displacement from POA as Push 2 enable the plotting of these graphs of capacity curves and for every single structure. However, to highlight that, POA could not predict the cyclic hysteresis demand and fundamentally ignored the dynamical nature of the building response during an earthquake.

- **Modal analysis Interpretation**

In order to obtain dynamic characteristics, mode shapes the modal properties and corresponding periods of the 2D analytical models of structures. Eigenvalue analyses were carried out. Figures 4.19 gives the variation of periods against modes and the number of modes selected for each structure is related to the storey number. Due to the regularities, no torsion is effective for all frames. The results of the first three mode shapes are what would be expected for the symmetric structure. The second and third mode is also expected for the symmetric structure. It can be noticed from Figures 4.19 the fact that higher modes.

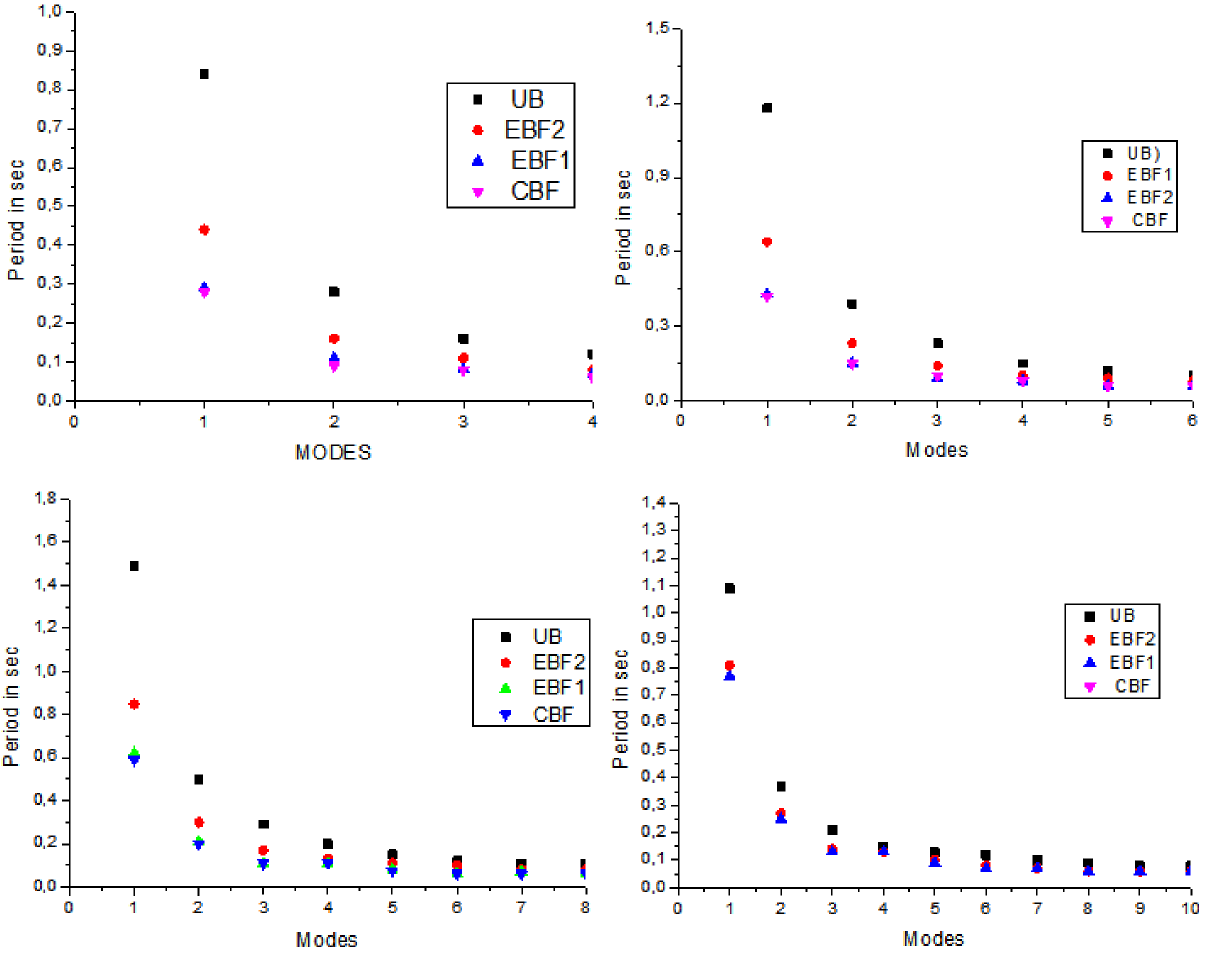


Figure 4. 23. Modal analysis results for four, six, eight storeys frames respectively**.**

Generally speaking, the modal analysis results show a definite effect of the frame's topology with the significant effect of the bracing on the fundamental mode and the storeys number in all studied structures. For higher modes, this influence seems to weakened as periods tend to have comparable values in all structures regardless the storeys number and the bracing systems. Regarding the participation coefficients, the structures arrive to 90% requirements in most cases in the first mode or in second or third mode and then meet the RPA and EC8 provisions. As can be seen in Figure 4.23, the variation pace seems, generally, the same for all studied structures despite their topologies. As expected, the more flexible the structure (UB) has the greater values of the periods in mode function compared to other structures. The bracing topology decreases the values of periods with a higher difference for CBFs then EBF1 and EBF2. The gap between the values of the periods for the different structures considered begins to decrease for the so-called higher modes and to asymptotic values for very high modes which remain hypothetic as the number of storeys increases. This is valid for the all 4, 6, 8 and 10 storey structures.

- **Spectral multimodal method (MMS)**

The obtained results of MMS show that the seismic displacements of floors resulting from the two regulations according to the multi-modal spectral methods are appreciably the same for the structures with 4 floors and this whatever the type of structure analysed (UB, CBF, EBF1 and EBF2). The general shape of the displacement curve is remarkably the same. Besides, the two regulations give roughly the same results, with slightly higher values for RPA99, with higher values for UB structures and displacements decrease with less values for the other frames and displacements in EBF structures are depending on the length of the link and are close to the case of CBF structures for EBF1 and to UB for EBF2, where the minimum values are found. As excepted, values of seismic displacements observed on structures in EBF are laying between those of UB and CBF for all structures studied (4, 6, 8 and 10 floors).

- **Results and discussion for nonlinear analysis**

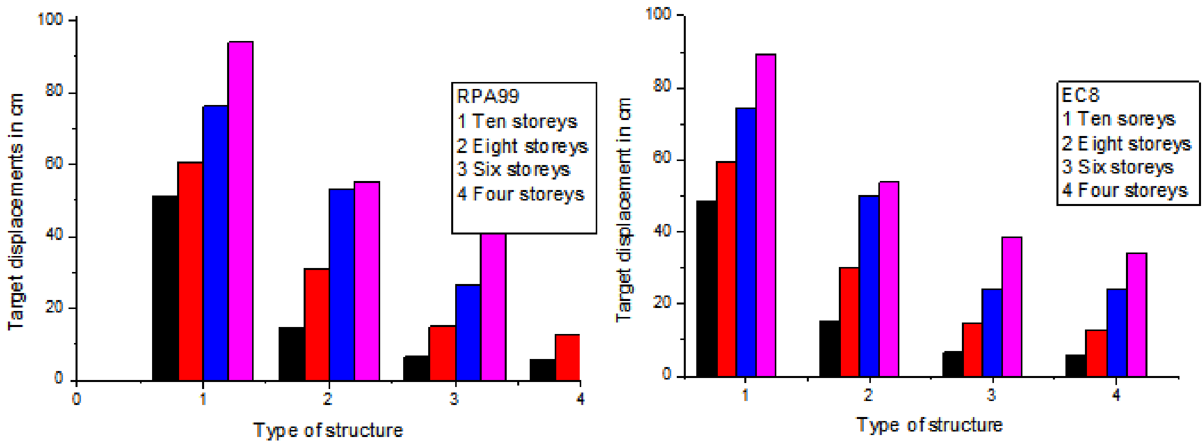
- **General considerations of Pushover analysis**

As stated in FEMA P695, a pushover analysis is a reasonable approach for first m ode dominated structures but is not representative of the dynamic response of structures in which higher modes dominant. The nonlinear static analysis reflects the global behaviour of a structure through a capacity curve representing the lateral force-deformation characteristics of the structure and forms the basis of approximate procedures which use the capacity curve to determine the seismic demands imposed on the structure.

The basic steps required to predict the seismic behaviour of a structure using pushover analysis consist of modelling nonlinear member behaviour of structure, performing a pushover analysis on the structure with an appropriate lateral load pattern using a software, predicting the maximum displacement demand of the structure by an approximate procedure and estimating important response parameters at predicted maximum displacement demand reasonably close to those predicted by nonlinear time history analyses. In this study, these basic steps were studied in detail on low, mid and high-rise reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. Firstly, computational scheme and underlying principles in modelling nonlinear member behaviour of SAP2000 which is commonly utilised software to perform pushover analysis were identified. Geometric nonlinearity can be considered through P-delta effects and account for true large displacement effects.

- **Target displacement**

The target displacement, that is the expected maximum displacement demand of the structure under a ground motion, is found in the expense of accuracy by disregarding the complex hysteretic actual behaviour of the structures and the intricate characteristics of ground motions to some extent. It is later used to predict the seismic demand in terms of forces and deformations at target displacement. Therefore, the accuracy of determining the displacement demand certainly affects the accuracy of the approximate seismic demand parameters using pushover analysis results. In this study, the provisions of EC8 are used to compute target displacements. Results are summarised in Figure 4.19 and 4.20 for RPA99 and EC8 respectively. By using the specified step-by-step method in EN 1998-1 Annex B. Point B.1 to B.6, the target displacement for different design situations can be calculated.



(a) RPA 99 (b) EC8

Figure 4. 24. Target displacement for all structures according to RPA99 and EC8

- **Plastic Hinge Locations**

Lateral load patterns utilized in traditional pushover analyses give some idea about the locations where inelastic behaviour is expected but their prediction of plastic hinge locations is generally inadequate for the deformation levels considered. Location of weak points and potential failure modes that structure would experience in case of a seismic event is expected to be identified by pushover analyses.

Nonlinear member behaviour is modelled by concentrated plastic hinges but plastic hinges can be defined at any point along the span of member as well as member ends in SAP2000. The location of a hinge is specified as a relative distance along the member, chosen by the user. It is important to ensure that the location of the hinges is representative of where the moment demands are greatest so that the hinges will be activated appropriately and the structure will perform as expected and realistically. The location of plastic hinges for case study steel frames were predicted by pushover analyses performed considering the lateral load patterns used in this study at roof.

Hinge properties can be determined using the default or they can be completely user-defined. The default hinge properties in SAP2000 are based on FEMA 356 guidelines, which are material-dependent. Since the hinges are also dependent on the cross-section properties of the element it is assigned to, the properties cannot be viewed until after the hinge is applied to an element and becomes a "generated" hinge. Once generated, the hinge properties cannot be modified. For this research work, the chosen method to model nonlinearity is through discrete flexural plastic hinges because it is commonly used in academia and practical design. Modelling hinges provides an advantage with valuable insight into the structure's response to loading. However, default discrete plastic hinges were also selected in SAP2000.

- **Discussion**

- **General**

The nonlinear static analyses of the previously mentioned two-dimensional systems were also carried out using SAP20000 NL using the same models developed for spectrum analysis. To this aim the same structure has been alternatively designed with Concentric Braced Frames (CBFs), Eccentric Braced Frames (EBFs) and Moment Resisting Frames (MRFs) according to RPA99 and EC8 and then analysed. Obviously, and because of the huge information obtained in this particular study on the nonlinear performance of multi-storeys mid to high-rise structures, a full discussion of all results will take several pages. The all studied structures, that is four, six, eight and ten structures, the essential results of pushover analysis regarding the capacity pushover curves, the internal forces in the links and the effective contribution of diagonal the point of performance of each structure will presented and discussed.

Result analysis and interpretation

- **General on SAP2000 NL outputs**

In this study, inelastic response of the selected buildings was predicted using the Nonlinear Static Procedure. The lateral loads are increased until a target displacement or onset of a structural mechanism. The structural response parameters, such as nodal displacements, element deformations, and strains, are recorded for each load step. Pushover analysis results provide an overview of the basic parameters that account for the seismic response of the structure. Meanwhile, during the analysis, the sequence of yielding, plastic hinges formation, displacement, and failure of various structural components can be recorded based on the objectives of the undertaken analyses. At each discrete interval, the effect of existing gravity loads is taken into account. In SAP2000 NL, the base shear at each incremental loading is plotted against corresponding lateral roof displacement. Also, at each stage, the resulting internal deformations and forces are determined and recorded, i.e. in the pushover curve, sequential elastic analysis is inbuilt. The resulting load displacement relationship is termed the capacity curve.

The assessment of seismic capacity of several frames designed to RPA99 and EC8 has been carried out under moderate earthquake, having medium ductility DCM in order to quantify their additional- resistance as respect to that strictly required by design earthquake. The numerical outcomes show that in all the examined structures could display a global linear behaviour except some parts of the frames: diagonals, links for which some local plastic behaviour has been noted. Globally, all frames could exhibit a sufficient additional capacity as respect to the design demand with values of overstrength ratio ranging between 2 and 4. Almost all frames however, each structural typology showed a particular seismic response. CBFs showed larger base shear forces, and slightly similar values for EBF1, which correspond to higher forces transmitted to foundations than those exhibited by EBF2s and MRFs. As it can be seen from Figures 4.21 and 4.22, the number of steps performed by the pushover analysis in SAP2000NL is about 10 for Push 1 and 30 for Push 2. However, and surprising enough, the final results from the two load patterns are to some extent different. Furthermore, astonishing finding is that the obtained values from RPA99 and EC8 Type 1 are similar in most cases with and slightly higher values for RPA99 regarding the seismic lateral force at each storey level, which can be seen as favourable argument. When varying the ratio of the length of link with respect to the span length i.e. e/L, between 0 and 1, the configuration of the structure varies between CBF and MRF, the shape of the capacity curves changes from almost linear to two branches representing linear and nonlinear behaviours. In case e/L = 1, the structure acts as MRF, but if e / L reduced to 0, structure can be identified as CBF. The calculated target displacements for both codes are represented in Figure 4.20.

- **Performance point**

The initial step of the detailed assessment of the existing or new buildings is the prediction of inelastic response (performance point, target displacement) against the earthquake intensity expected at the site.

To understand the actual behaviour of a structure, the inelastic range will provide valuable information about the response of the structure by identifying actual failure modes, which usually refers to an ultimate force (F_{ult}) and displacement (Δ_{ult}).

To determine the inelastic deformation demand, two widely used approximate procedures, implanted in SAP2000 NL that rely on pushover curve of the structure exist. These procedures, namely the Capacity Spectrum Method of ATC-40 and the Displacement Coefficient Method of FEMA 356 generally use a bi-linear representation of the original pushover curve to compute the approximate inelastic displacement demand. The capacity spectrum method (CSM) was originally developed by Freeman et al. (1975), as part of a rapid evaluation procedure. Idealisation is held on the capacity spectrum of a structure which is used in conjunction with the Acceleration Displacement-Response-Spectra (ADRS) to compute the performance point (target displacement) of the structure. According to ATC method, a bi-linear representation of the capacity spectrum is obtained by using the initial stiffness of the capacity curve and by satisfying the equal energy rule, as it is in EC8. A key point in this method is related with the trial performance point. Thus, the bilinearization of the capacity spectrum should be repeated for every trial to match the areas under each curve obtained. The original pushover curve is typically obtained by applying a load pattern that represents the first mode response of the structure.

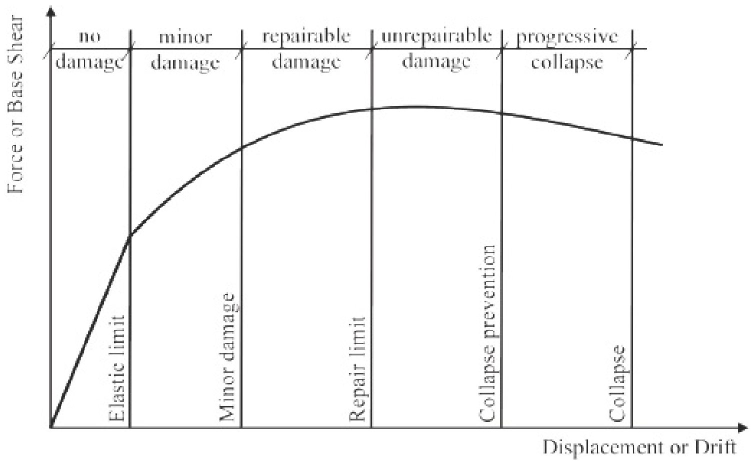


Figure 4. 25. Performance and levels of damage of a structure **(Ghobarah 1997).**

Table 4. 6. Performance point according to Push 1 by these procedures, namely the Capacity Spectrum Method of ATC-40.

Performance Point	RPA99				EC8			
	UB	**EBF2**	**EBF1**	**CBF**	**UB**	**EBF2**	**EBF1**	**CBF**
	V (kN)	V (kN)	V (kN)	V(kN)	V (kN)	V (kN)	V (kN)	V(kN)
	D (m)	D (m)	D (m)	D(m)	D (m)	D (m)	D (m)	D(m)
Four storeys	360.249	540.610	513.586	505.997	367.872	545.23	532.419	523.240
	0.106	0.05	0.027	0.023	0.108	0.053	0.028	0.025
Six storeys	364.648	536.933	585.090	622.943	372.636	546.325	584.710	633.984
	0.153	0.075	0.050	0.051	0.156	0.077	0.050	0.052
Eight storeys	373.455	540.617	604.434	622.237	381.759	549.243	613.757	631.541
	0.195	0.102	0.074	0.071	0.199	0.104	0.076	0.073
Ten storeys	380.794	538.367	596.097	610.972	389.324	547.023	606.329	622.295
	0.240	0.132	0.099	0.095	0.245	0.135	0.101	0.097

As can be easily noticed from Tables 4.8 and 4.8, it seems that the two seismic codes, namely RPA99 and EC8 give almost the same values for the performance point, indicating that structures are behaving in elastic manner. Surprisingly enough, it can be also noticed that for push 2 for eight storeys structures the same values are recorded. The UB structures have the low values for the acting force with higher displacement, while CBF frames have the lower displacement values with higher acting force, meanwhile, structures in EBF are ranging between the latter topologies with EBF2 approaching UB and EBF1 close to CBFs. All four storeys studied structures have been observed to behave in elastic manner, but as the number of storeys increase, some inelastic behaviour can be observed, see tables below.

Table 4. 7. Performance point according to Push 2 these procedures, namely the Capacity Spectrum Method of ATC-40.

Performance Point	RPA99				EC8			
	UB	**EBF2**	**EBF1**	**CBF**	**UB**	**EBF2**	**EBF1**	**CBF**
	V (kN)	V (kN)	V (kN)	V(kN)	V (kN)	V (kN)	V (kN)	V(kN)
	D (m)	D (m)	D (m)	D(m)	D (m)	D (m)	D (m)	D(m)
Four storeys	404	616.85	505.276	543.15	401.0	607.17	550.274	541.175
	0.095	0.046	0.022	0.019	0.910	0.046	0.022	0.019
Six storeys	417.23	622.253	678.635	726.545	419.2	622.979	678.748	726.540
	0.130	0.067	0.045	0.0450	0.129	0.067	0.045	0.045
Eight storeys	434.956	642.578	717.417	738.848	434.956	6.42.213	714.312	738.212
	0.170	0.071	0.065	0.063	0.170	0.0910	0.065	0.063
Ten storeys	447.227	641.741	715.205	734.71	447.277	641.617	713.941	734.033
	0.208	0.116	0.102	0.083	0.208	0.116	0.0860	0.083

- **Capacity curves**

General

Although pushover analyses give an insight about nonlinear behaviour imposed on structure by seismic action, any design and seismic evaluation process should be performed by keeping in mind that some amount of variation always exists in seismic demand prediction of pushover analysis for low, mid and high-rise frames. A force-displacement curve is created by applying a monotonically increasing horizontal force to the structure, while still subject to gravitational loads, until collapse, or non-convergence. An important non-linear feature that are to be included in the SPO-analysis, is (P-δ) effects. Since the lateral loading is applied in combination with vertical gravitational loads, (P -δ) effects can be of great significance in determination of capacity.

In the conventional method of nonlinear static analysis, the lateral storey inertial invariant load pattern is maintained invariant with time although in a vibrating structure, the distribution of storey inertial forces varies with time. During the analyses, certain responses such as element forces, nodal displacements and storey drifts can be recorded and then, processed to obtain the maximum base shear 'V'.

A non-linear static pushover analysis is used to quantify the maximum base shear and the ultimate displacement, which are in turn used to compute overstrength and ductility. In this study, it must be recalled that the push1 is representing the lateral forces pattern was carried out in 10 steps. As well-known, the capacity (pushover) curve is computed as the relationship between base shear force and lateral displacement of the "control" node that is assigned as the centre of mass at the roof. For the lateral load patterns, at least two vertical distributions must be used.

It must be noted that results of Pushover analysis coming from depend on the used software due to its limitations and element library. i.e. SAP2000 NL for instance. SAP2000 NL produced almost same pushover curves for both RPA and EC8 data input and yielded almost same sequence of plastic hinging and plastic hinge pattern. The Push1 load patterns yielded almost same predictions of global capacity curve, storey displacement, inter-storey drift ratio, storey shear and plastic hinge location for low to mid rise frames, since the variation in height-wise distribution of triangular lateral load patterns is negligible for low to mid-rise frames. An overview of the obtained results demonstrates a general good agreement between RPA99 and EC8 as are depicted in Figures 4.24 and 4.25 for RPA99 and EC8 respectively. The maximum base shear is however close in the two seismic codes.

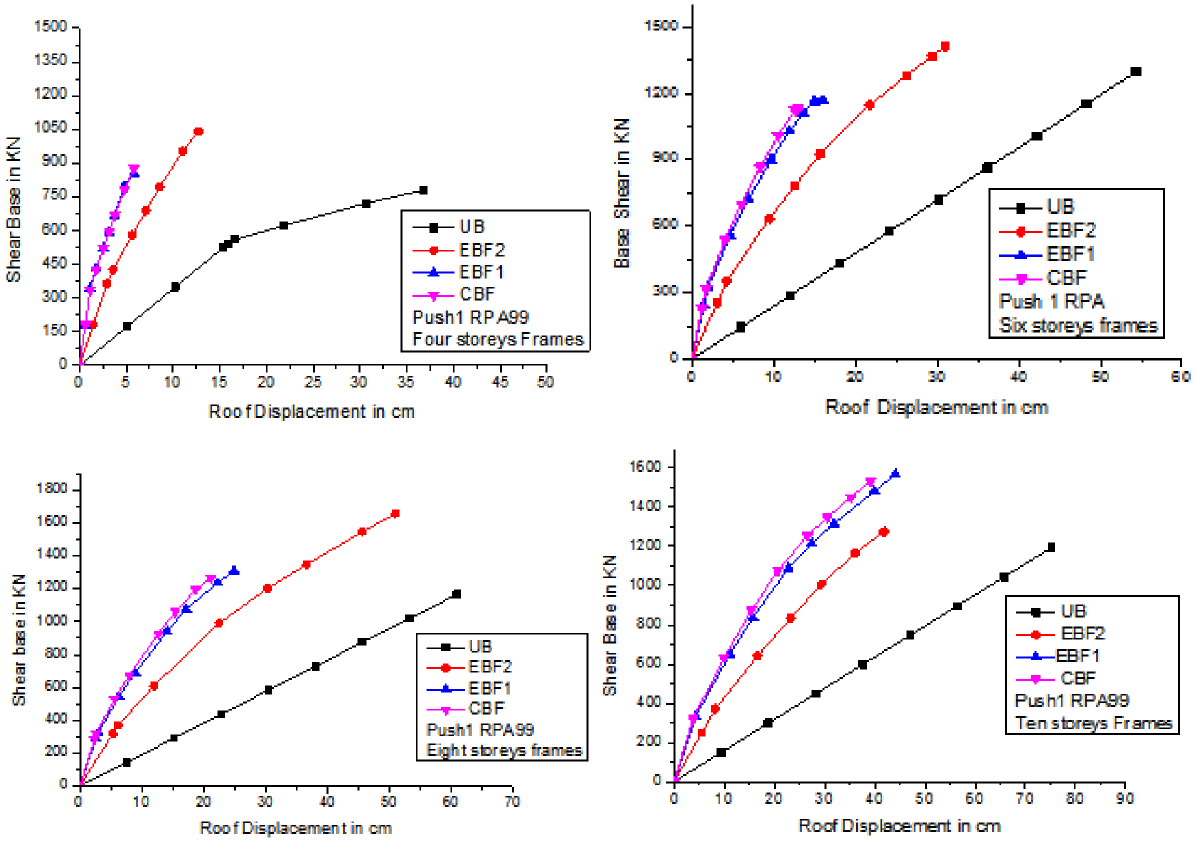
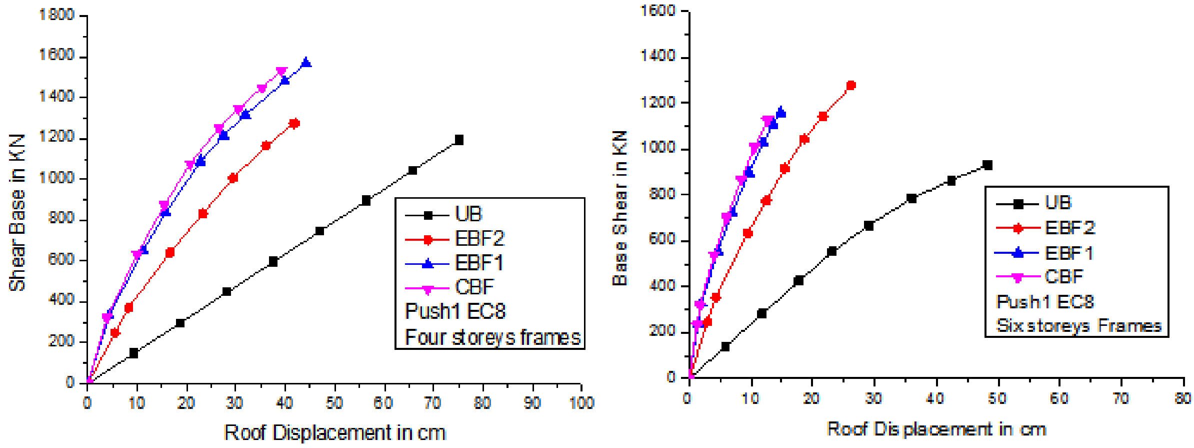


Figure 4. 26 Capacity curves Push 1 for the studied frames according to RPA99 for four, six, eight and ten storeys structures



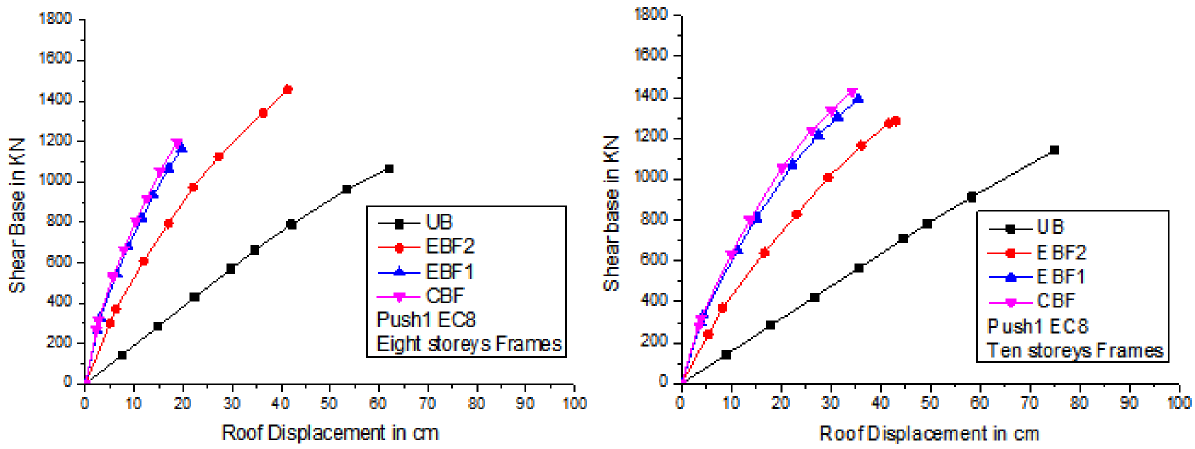


Figure 4. 27 Capacity curves Push 1 for the studied frames according to EC8 for four, six, eight and ten storeys structures

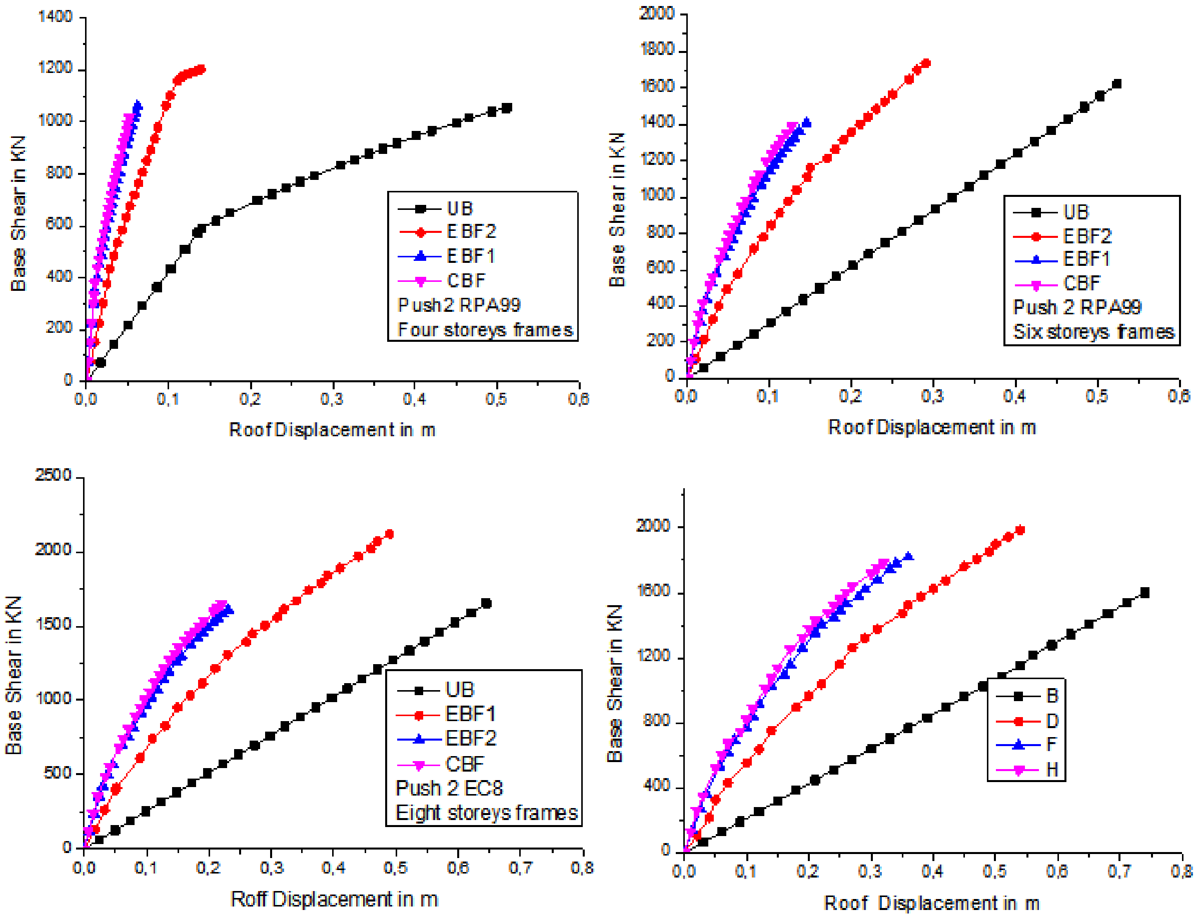


Figure 4. 28 Capacity curves Push 2 for the studied frames according to RPA99 for four, six storeys structures and to EC8 for eight and ten storeys structures

The first part of the curves, which is the linear elastic part of the response, is also correlating well. The whole analysis is conducted in moderate seismic area with a moderate ductility for all structures considered in this work. Comparing results in Figures 4.24 to 4.28, it can be noticeable that curves obtained from this analysis range from elastic behaviour to reparable state. UB structures behave in elastic manner in most cases, this can also be said for EBF2 frames. While CBF frames and EBF1 frames, which is eccentrically braced with short link, show a close behaviour. Comparing the pushover capacity curves of all examined structural typologies. Figures 4.25 and 4.26 it is possible to recognize that both EBF2s and MRFs exhibit large roof displacement ductility as respect to CBFs and EBF1. Indeed, displacement capacity of the latter structures is impaired by the ductility capacity of the braces in compression. In addition, the maximum base shear forces of CBFs are larger than those exhibited by EBFs and MRFs. his implies higher forces transmitted to foundations in the former case, which correspond to higher costs of construction.

- **Overstrength**

Overstrength came into focus in the 1970s with the introduction in New Zealand of a capacity-based approach to design. Overstrength is a measure of the amount by which the prevailing strength in a system or its elements exceeds the specified strength and can be represented as

$$\text{Overstrength} = \frac{\text{Maximum lateral strength of the structure}}{\text{Unfactored design strength as per code}}$$

The advantages of overstrength are obvious, but on the flip side, if a beam possesses large flexural overstrength, it could lead to brittle shear failure under high seismic displacements. Secondly, since column failure has serious consequences, it is clear that the strength of a column, at its junction with frame beams, must exceed that of the beams inclusive of their overstrength. In seismic design both overstrength and understrength can prove to be detrimental to structural safety. To prevent shear failure in beams, designers should provide for an overstrength in shear.

- **Links**

Understanding strength, stiffness and ductility properties of EBFs requires studying the nonlinear behaviour of links. Research studies show that behaviour of shear links is fairly complicated and affected by various parameters, and as a result significant amount of research interest has been directed towards both experimental and numerical determination of the nonlinear behaviour and cyclic energy dissipation characteristics of shear-links. As mentioned earlier, the active link must be designed in order to obtain that its bending and shear limit strength precedes the attainment of the tension and compression limit strength of other elements. The length of the active link is responsible of the mode of collapse mechanism which dissipates energy.

As the objective of this part of research work aimed at investing the assessment of differences of the linear and nonlinear behaviour of EBFs structures in the same seismic conditions with other more conventional steel frames that is UB and CBFs covered in RPA99. Furthermore, an insight of the global inelastic performance of links will be introduced.

The structure is characterized by a first-mode dominated response developing an overall mechanism, with plastic engagement of links well distributed along the height of the building.

Obviously, and because of the huge information obtained in this particular study on the nonlinear performance of multi-storeys mid to high-rise structures concerning the behaviour of links, a summary of results will be discussed with some drawn conclusions.

- **Internal forces distribution**

The internal forces distribution in EBFs under the action of horizontal forces are shown and discussed in previous Chapter. Generally speaking, the obtained results regarding the behaviour of links were, as found in earlier works provide in literature, and were satisfactory and no special remark can be mentioned. As the axial forces in links is concerned, and despite its length, it has been remarked that its value is always feeble and uniform and increases as with storey level but permanently less the ultimate value and it can therefore be neglected without risk. For the other internal forces, that is shear and bending moments values, as expected, it has been observed that the behaviour of seismic segment of EBFs changes depending on its length along with other parameters of the structure. in which shear force is being constant along the link while bending moment has similar values at end of the segment whereas the axial force. Indeed, shear is generally observed along a seismic link constant, moments at different ends in value accompanied by a normal force weak along a short seismic (shear) link, EBF1 is very small as point out **(Hjelmstad 1984, Okazi 2006),** see figure 4.15, the condition of EC8 N_{Ed}/Npl, Rd $\leq$ 0.15 it is verified. The moments developed at the ends in the beam have different values depending on the storey level and the lateral forces. It has been also noticed that the maximum values of the shearing forces were generally located at the first floor of all of structure having four and six storeys and at storey level 2 for eight and ten storeys structures. The same remarks can be made as far as values of bending moment are concerned.

All remarks made in the following section are also valid for both codes RPA99 and EC8 and obviously with different values of internal forces, with a general trend of slightly higher values for RPA99. This is also true for both lateral patterns: Push 1 and Push 2.

Shear forces in the links with long link: EBF2 During loading history, it has been noticed from the obtained results that the shear force is constant along the seismic section and remains below V_{pl}. full plastic capacity of the link, with the maximum arithmetic value located at the level of the bottom storey for the 4^{th} and 6^{th} frames. For EBF1, similarly to EBF2, the shear force is constant on the along the seismic section and remains less than V_{pl}. the maximum arithmetic value is at the first storey and the condition of the EC8 $V_{Ed} \leq V_{pl}$, link is verified in all four storeys structures. As expected, the shear force in the shorter link is higher than in the other sections because it dissipates the energy by plasticization mainly in shear link. The location of maximum shear force is moved to upper storey levels, that is the second and third level storeys for 8^{th} and 10^{th} structures as the frames getting higher, but is being almost constant and always under the limit it is for mid-rise structures.

Bending moments

And obviously with different values of moment resistance plastic capacity of all eccentrically frame structures (4, 6, 8 and 10 floors frames), which can be explained by the absence of the necessary stiffeners in the seismic section. This Observation is valid for push1 and push 2 patterns by RPA99 and EC8 codes. The bending moment in the long link, EBF 2, has higher value as the long link section dissipates energy by plasticization mainly in bending. In lower stories, it has been noticed that the obtained bending moment values are bigger that the M_{pl} of the link section, and behaves thereby in plastic manner in EBF1 and EBF2 frames. These exceeding values can be explained by the absence of stiffeners required by seismic codes adopting EBFs structures. This effect will be evaluated in details in the next chapter with an international original paper. It was observed that for mid-rise frames: 4^{th} and 6^{th} storeys, the maximum bending moment was observed in the first level, while for more slender structures the maximum bending moment move up to the second and third level storey respectively.

- **Axial forces in braces**

As far as bracing system is concerned, once again, values from both seismic codes were similar and the same pattern of failure has been observed during the loading histories. As well-known, the rupture of diagonals occurs well beyond the tension one because of buckling phenomenon, namely Euler's bucking. It has also been noticed that the bracing system losses its importance as the structures considered were higher. However, for the structures having four storeys a significant improve of the behaviour by a significant reduction of general Load-displacement curve, especially for CBFs and EBF1, with less reduction. for EBF2 as its behaviour resembles to UB in many ways. This is also true, to some extent, to six storeys frames. Also, it is worth to mention that bracings placed in lower storeys are more effective as they received more internal forces and then to form a plastic hinge more rapidly than those located in the upper storeys. For slender structures, eight and ten storeys, both CBFs and EBFs bracing systems do not help in improving in the global behaviour of the frames.

- **Summary and concluding remarks**

To highlight the results obtained in this particular study, more detailed results will be provided in an international paper, which at the present time in progress and be finished in very near future.

At the light of the results of the parametrical analysis, several conclusions may be deducted, a summary of which will be given in the following, namely:

According to results obtained through this particular study, the most important conclusion which can be deducted is that structures designed to meet RPA99 provisions with eccentric EBFs bracing system, not covered yet by RPA99, can be justified and therefore applicable for different lengths of seismic sections: short, medium and long, by using the EC8 provisions, based upon works, and the nonlinear pushover analysis can be safely used as it gives very encouraging results compared to EC8 ones and with an overstrength ratio exceeding the value limited by EC8. Results of this study have shown that Pushover can be safely applied to steel as results from two loads patterns are similar whether considering RPA99 or EC8 codes. Also, the results of this particular study seem to indicate that this use of structures bracing can be justified. As mentioned in many research publications, the inelastic behaviour of EBFs seems to be very close to CBFs for shorter links, and nearer to UB for long links. For EBFs structures, the shear force may exceed limit values indicated in EC8 because of absence of transverse stiffeners necessary to withstand the links highly charged, namely in case of a major earthquake regardless the number of storeys. As expected, the values of shear forces increase or decrease depending on the length of the link. This topic will be fully studied and discussed in the following chapter and has been published in international original paper. For slender structures, of 8 and 10 storeys, the presence of a diagonal of bracing in CBF or EBFs does not seem to be necessary since the efforts in these diagonals always remain very low, which is not the case for lower structures where diagonals are highly stressed and a plastic hinge in the compressed diagonals of the lower floors. is commonly formed in lower levels of the frames.

The following specific conclusions can be drawn:

The pushover analysis results have shown that the most stressed parts, that is beams, of a frame are the first to form a plastic hinge after diagonals and the bottom column.

For the elastic spectral analysis, the predictions of RPA99 and EC8 in terms of displacement spectral lateral are substantially the same for all structures studied. Pushover analysis showed that the principle of strong columns/weak beams used in dimensioning structures determining for structures mid and high-rise structures is useful as it seems to be correct since the performance point of all structures is at a ratio (V/V_b) and overstrength is observed in all structures remain in elastic behaviour. Analysis of the braced structures showed that the incursion into the elasto-plastic domain primarily, as expected, affects the compressed diagonals of the first levels, even before the column base, with a negligible contribution of bracing which needs more attention during the dimensioning process, however, in upper floors despite their nature bracing of the is negligible. Then; it's the turn to beams in first floor to be plasticized. This study employed a few numbers of steel frames. An extensive study containing a larger number of frames covering a broad range of fundamental periods and a set of representative ground motion records would enhance the results obtained in the accuracy seismic demand prediction of pushover procedures and in the accuracy of maximum displacement demand prediction of approximate procedures. Finally, all the issues mentioned could be extended into three dimensional structures.

The aim of this chapter is to define the parametrical study in order to investigate and evaluate the elastic and inelastic behaviours and performance of regular and irregular multi-storeys steel structures. The seismic performance of dual-steel building frames composed of mild steel grade. The seismic design based on RPA 99 provisions. In recent years, pushover analysis is gaining popularity for predicting the likely inelastic performance of a structures with important merits and limitations as it can determine the load-carrying capacity of an inelastic system and the likely failure mechanism under progressive yielding along with an insight into the nonlinear behaviour and can help identify the locations of potential weakness prevailing in a structure where failure could occur. It provides a good estimate of local and global inelastic demands for inter-storey.

Every structure to be erected in a seismic region has to be designed and constructed in such a way to meet, with an adequate degree of reliability, specific requirements connected to the return period of seismic action. Each seismic code should define a set of return periods of seismic action and the corresponding required performances, ranging from a “damage limitation” requirement to a “no-collapse” requirement. In the first case the structure will remain in the elastic range, while in the last one it will undergo large inelastic deformation.

In order to check the structural performance, it is necessary:

- to define a geometrical and mechanical model of the building, which may include only the structural elements or also the so-called non-structural elements;
- To evaluate the seismic response of the building in the elastic range; and seismic response of the building in the inelastic range.

4.9 ELASTIC AND INELASTIC BEHAVIOURS OF IRREGULAR MULTI-STOREYS STEEL DUAL-STRUCTURES

4.9.1 INTRODUCTION

- **Outline**

Notwithstanding the significant improvements occurred in the approach to seismic design of steel structures to prevent the collapse subjected to design earthquakes, design structural deficiencies can still be observed namely in: weak/soft-storey. Indeed, several partial or total collapses are due to soft and weak storeys, insufficient lateral bracing, and inadequate ties and connections between the components of the building **(Elnashai 2008).** Actually, these upturns have been brought about by a more general understanding of the nature of the problems and the extensive development of the digital computer. The role played in the seismic analysis by the triad of capacities (stiffness, strength and ductility) plus robustness is recognised **(Gioncu 2011).** The soft first storey condition is recognized as an undesirable condition of structural irregularity for seismic design worldwide. The soft first-storey is the most common feature of soft-storey irregularity and cannot be eliminated because of its important functional requirement of almost all the urban multi-storey buildings. The problem has to be tackled with a holistic approach **(Tena-Colunga 1999).** The codes set a somewhat arbitrary threshold to identify the parameters that are involved in such situations. It is worth to remark that Algerian Seismic Code RPA99 **(RPA 99 2003)** does not address clear provisions about the concept of soft/weak-storey design methodologies for steel structures. In the United States, the **(UBC 1997)** also included the soft storey condition since 1988 as one of the recognized vertical structural irregularities to account for design, and most recent regulations such as **(ASCE7-05 2010)** and **(IBC-06 2006)** endorse also these recommendations.

- **Problematic**

Open ground storey is a typical feature in the modern multi-storey constructions in urban zones. Many of these buildings do not have masonry infill at ground floor level to increase the flexibility of the space for recreational use, parking or commercial use. Such features are highly undesirable in buildings built in seismically active areas; this has been verified in numerous experiences of strong shaking during the past earthquakes around the world. These buildings which possess open ground storey are significantly weaker or more flexible than adjacent storeys are known as soft- storey buildings because of the absence of walls makes them much flexible than upper storeys. It is commonly generate unconscientiously due to the elimination or reduction in number of rigid non-structural walls in one of the floors of a building, or for not considering on the structural design and analysis, the restriction to free deformation that enforces on the rest of the floors, the attachment of rigid elements to structural components that were not originally taken into consideration.

The inadequately-braced soft-storey level is relatively less stiff than the upper floors to lateral earthquake motion, subject to disproportionate lateral stress, and less able to withstand the stress, the floor becomes a weak point that usually suffer structural damage or complete failure, which in turn results in the collapse of the entire building under earthquakes of high magnitude. While weak-storey is an irregularity referring to the existence of a building floor presenting a lower lateral structural resistance than the immediate superior floor or the rest of the floors of the building. The building's weakest part would suffer severe damages due to its inability to withstand the different types of loads (lateral, vertical and moments) produced by the ground motion.

- **Scope and objective of the study**

The understanding of the elastic and inelastic behaviours in earthquake condition of steel multi-storeys building structures having vertical irregularity in stiffness and masses distribution is quite important. In other words, as already mentioned in chapter 2, in earthquake resistant design, the soft storey and the weak storey irregularities are reciprocal to a significant difference between the stiffness and the resistance of one of the floors of a structure and the rest of them. A soft-storey could occur at any floor. This creates a sudden and significant reduction in lateral stiffness resulting in formation of a soft-storey. Unfortunately, this occurs at a level where gravity and inertia load from superstructure are highest. As a result, large deformations can occur causing substantial P-Δ effects which can lead to undesirable plastic hinge formation in columns.

This section of the chapter focuses on some kind of types of irregularities in vertical stiffness with their implications and methods to minimize their negative impact on the linear and non-linear behaviour of such structures are discussed as shown in Figure 4.29. Merely possessing inherent structural strength is not adequate for a building to successfully withstand a major seismic event. In addition to strength, the framework should meet codes limitations regarding permissible drift being 1% in most seismic codes.

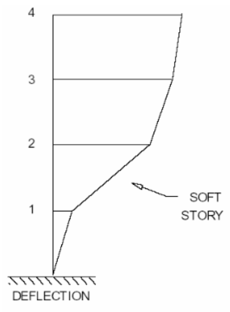


Figure 4. 29 Soft storey behaviour of a building structure under lateral loading according to FEMA-310

- **Methodology**

The aim of this work is an attempt to assess the impact in elastic and inelastic behaviours of irregularity on structural behaviour of low and mid- rise bare steel structures having soft-storey equipped with various type of bracing devices arrangement, designed to exclusively to meet only RPA99 and CCM97 provisions, contrary to previous studies described in this chapter, is done using strong columns vs. weak beams criteria (SCWB) which encouraged potential primary plastic regions to be in the beams except at the column bases, this concept is explained in details in Chapter 3.

As the present study is dealing with a parametric analysis of the linear and nonlinear seismic performance assessment of steel structures having irregularity in vertical stiffness, which may be a soft/weak-storey in low and mid- rise bare steel structures limited to a single soft storey located at the first-storey, and the effect of retrofitting the first floor with bracing devices, several two- dimensional analytical models with single bay and various number of stories having soft-storey at the first level are investigated. Clearly it is important that the model assumed in the analysis is indeed representative of the way in which the proposed structure would behave. In order determine whether the structure is satisfactory, the structural response is compared with the appropriate codified limits especially the drift-storey and local and overall deflection response.

Critical buckling modes detect soft storey mechanisms and unlike EN 1993-1-1 lead to different angles for each storey, despite the fact that the mean angle from the foundation to the top of the building is the same. The severity of the soft/weak storey is studied by varying the increasing the height of the first- storey. The design is accordingly safer, as soft stories are assigned higher sway imperfections.

In this section, the main focus is to evaluate the seismic performance of soft or weak storey structures retrofitted with braces at the ground storey. Structural deficiencies of potentially soft or weak storey buildings have been considered by increasing the storey height from 3 m to 4.5

m and 6 m while keeping the same span length, l = 6 m, to demonstrate how their performance can be improved under seismic action. Four and six structure models located in moderate seismic zone are studied through elastic and inelastic pushover analysis using SAP 2000.

4.9.2 Design of analytical models

- **Introduction**

In earthquake resistant design, the soft storey and the weak storey irregularities are reciprocal to a significant difference between the stiffness and the resistance of one of the floors of a building and the rest of them. Configuration defines the size and shape of members of a building, their layout in plan and elevation as well as nature and location of the lateral force resisting elements. From a structural engineer's standpoint, symmetrical structures with uniform mass and stiffness distribution are among the key requirements for satisfactory seismic performance. If the building, and its structure, has been conceived respecting the general principles of a good conceptual design (structural simplicity, uniformity, symmetry and redundancy, bi-directional resistance and stiffness, torsional resistance and stiffness, diaphragmatic behaviour at storey level, adequate foundation) it is possible to use standard modelling while checking the structural performance. The above-mentioned principles grant adequate reliability of the numerical analysis and, at the same time, promote a good behaviour under seismic actions more severe than the design ones.

- **Model general considerations**

It can then be ascertained whether the responses meet global performance objective, e.g. stability against lateral loads, inter-storey drifts, plastic hinge rotations floor displacements, strength, etc. Thereafter, the check can be at component level, viz.: beams, columns and other structural elements. This portion of the design process needs to be done with utmost care supported by sound engineering judgment. During the development of the analytical models, several issues were explored. A significant topic at this stage was to evaluate easily the existence of the soft storey behaviour in the structure by increasing the height of the first floor from 3 m to 4.5 and 6 m respectively. Another important point is the representation of the steel building typical in Algeria by the models that are selected. Taking into account these considerations and for simplification purposes, 4 and 6 storey frames with single bay are modelled since most of the framed structures in Algeria have number of stories ranging between two to eight storeys. The weakness is imposed by increasing the height of the first floor without increasing the cross-section of the columns, nor changing the reinforcement, resulting in a softer building, and thus a higher first natural period.

According to EN 1998, moment resisting frames should be designed such that plastic hinges are formed in the beams or in the beam to column connections and not in the columns. This requirement could not be respected at column bases and at the top storey of multi storey buildings. Following EN 1998, plastic mechanisms in the connections are possible only when special connections are introduced and their capability to develop plastic deformations is studied experimentally and analytically. Otherwise, when the usual types of connections are employed, it should be ensured that plastic hinges develop only in the beams. The beams should have sufficient resistance against lateral-torsional buckling, assuming a plastic hinge at its most loaded end, under the seismic design situation.

The design procedure is based on the philosophy of strong-column-weak-beam (SCWB), details can be found in Chapter 3 of this thesis. Various response parameters are used for all structures to determine what correlations can be found for inelastic structures response, see Figure 4.30.

The model is pushed to a target displacement determined automatically by SAP2000 using ATC-40 recommendations. For modelling of the members in SAP 2000, all frame members, beams and columns are considered as rigid-ended. Also, the nonlinear behaviour of members is suggested by FEMA-356 and ATC40. The P-Delta effect is captured by applying the floor gravity loads on ‘gravity columns’ which can be lumped into one.

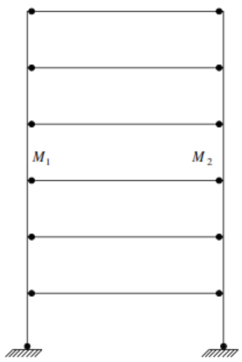


Figure 4. 30 Weak beam–strong column frame with potential sections for hinge formation (**Krawinkler 2009**)

- **Definition of the structural typologies**
  - **Geometric**

The analytical models are two-dimensional and beam and column elements, are represented as one dimensional frame elements. The members are prismatic and linear. The span length for the bays is chosen as 6m in all the models considered. The storey height in the models is fixed to 3 m for all the floors except the first. In order to carry out a parametric study to examine the soft storey irregularity, the height of the first storey of the models is kept as the selected variable. Three different heights that are considered for the first floor are: 3 m, 4.5m and 6m. With these three different heights for the first floor the total number of analysed models reaches to 36. Beams and columns of the building are modelled as one-dimensional members, mutually connected in points named nodes.

- **Material**

Beams, columns and braces are made with hot-rolled first- class sections to CCM97 and EC3 classification. The mild ductile S235 grade steel was used for all frame members including brace elements. The latter are being designed to satisfy only the buckling condition. Material nonlinearity is represented by concentrated plastic hinges as previously explain in section 4.8.

N.B. As it is clearly represented in Tables 4.8. 4.9 and 4.10, each model in this study is named according to the type of structure, and the height of the first storey. For example, the model referred to as UB3 is representing the regular unbraced structure (i.e. without additional bracing at the ground story), 3 indicates the height of the first storey in m, while BCBX3 is for a regular structure with concentric X bracing at the bottom of the structure.

Table 4. 8. Frame Member sections for four storeys structures.

Type of frames				**Storey number**	**Column section**	**Beam section**
UB3	**BCBX3**	**BCBV3**	**BEBFS3**	**1**	HEB160	IPE360
			BEBFM3	2	HEB220	IPE360
				3	HEB260	IPE360
			BEBFL3	4	HEB320	IPE360
UB4.5	**BCBX4.5**	**BCBV4.5**	**BEBFS4.5**	**1**	HEB160	IPE360
			BEBFM4.5	2	HEB220	IPE360
				3	HEB260	IPE360
			BEBFL4.5	4	HEB340	IPE360
UB6	**BCBX6**	**BCBV6**	**BEBFS6**	**1**	HEB160	IPE360
			BEBFM6	2	HEB220	IPE360
				3	HEB260	IPE360
			BEBFL6	4	HEB400	IPE360

Table 4. 9. Frame Member sections for six 6 storeys structures.

Type of frames				**Storey number**	**Column section**	**Beam section**
UB3	**BCBX3**	**BCBV3**	**BEBFS3**	1	HEB160	IPE360
				2	HEB220	IPE360
			BEBFM3	3	HEB260	IPE360
				4	HEB300	IPE360
			BEBFL3	5	HEB360	IPE360
				6	HEB450	IPE360
UB4.5	**BCBX4.5**	**BCBV4.5**	**BEBFS4.5**	1	HEB160	IPE360
				2	HEB220	IPE360
			BEBFM4.5	3	HEB260	IPE360
				4	HEB300	IPE360
			BEBFL4.5	5	HEB360	IPE360
				6	HEB500	IPE360
UB6	**BCBX6**	**BCBV6**	**BEBFS6**	1	HEB160	IPE360
				2	HEB220	IPE360
			BEBFM6	3	HEB260	IPE360
				4	HEB300	IPE360
			BEBFL6	5	HEB360	IPE360
				6	HEB550	IPE360

Table 4. 10. Geometrical characteristics of the bracing system.

FRAME	$\bar{\lambda}$	**BRACE**	**A (cm^2)**
BCBX3	1.6	2L100.10	38.4
BCBX4.5	1.6	2L120.12	55.08
BCBX6	1.6	2L150.15	86.04

BCBV3	1.6	2L90.10	34.26
BCBV4.5	1.6	2L100.10	38.4
BCBV6	1.6	2L100.10	38.4
BEBFS3	1.6	2L50.5	9.6
BEBFL3	1.6	2L50.7	13.12
BEBFM3	1.6	2L60.4	9.34
BEBFS4.5	1.6	2L90.8	27.78
BEBFL4.5	1.6	2L80.6	18.7
BEBFM4.5	1.6	2L90.6	21.14
BEBFS6	1.6	2L100.8	31.02
BEBFL6	1.6	2L100.8	31.02
BEBFM6	1.6	2L100.8	31.02

4.9.3 Results and discussion

- **Outline**

The assessment of the linear and nonlinear behaviour through a comprehensive numerical analysis of two- dimensional analytical models of low to mid-rise steel moment resisting frames with and without first soft-storey configuration, having or not a reinforcement with different kind of bracings for single bay structural with various number of stories and single length span are investigated. In order to predict the actual structure behaviour under a given seismic loading, different elastic analyses were performed, that is: the modal analysis; lateral forces method, spectrum analysis and the pushover analysis as that the effects of nonlinearity are not proportional to the cause and small causes may have large effects. The nonlinear behaviour was assessed through static pushover detailed in section 4.8, by considering various first storey heights and deformation levels.

- **Modal Analysis**

Performing the modal analysis for all structures as it is necessary step for the next moves with and without the irregular storey using software to make sure that the Eigen values are reliable and a special emphasis on the correct values of mass participation factor are important in subsequent analysis.

Modal analysis provides a correct estimate of the elastic response of any structure in the elastic range, provided that: Correct determination of natural frequencies is extremely important and forms the basis of any further nonlinear analyses (**Senjanovi 2014)**. Practical dynamic analysis of large complicated multiple-degrees-of-freedom (MDOF) is generally accomplished by computer-implemented numerical analysis techniques such as the finite element method **(Rajasekaran 2009).**

In a building, the mass is distributed over its entire geometry which implies a continuum system with its infinite degrees of freedom. Thus, for dynamic analysis purposes, a continuum system having masses distributed over its height is replaced by a lumped mass system with masses (with no physical size) concentrated at floor levels, thus reducing a structure with infinite degrees of freedom to one with drastically reduced number of degrees of freedom A lumped mass does not undergo any rotation and its position at any time can be represented by a single kinematic displacement quantity. In this study, two approaches exist for calculating the element mass matrix were used. The consistent mass matrix, used in SAP2000 NL is the most accurate way of representing the mass and inertia of the element, but when the aim is to calculate the dynamic response of the structure the simpler lumped mass approach, used in LUSAS software, can be more effective as it avoids single element modes of vibration **(SCI 2003).** It is also important to ensure that all the relevant mass is accounted for in a modal dynamic analysis.

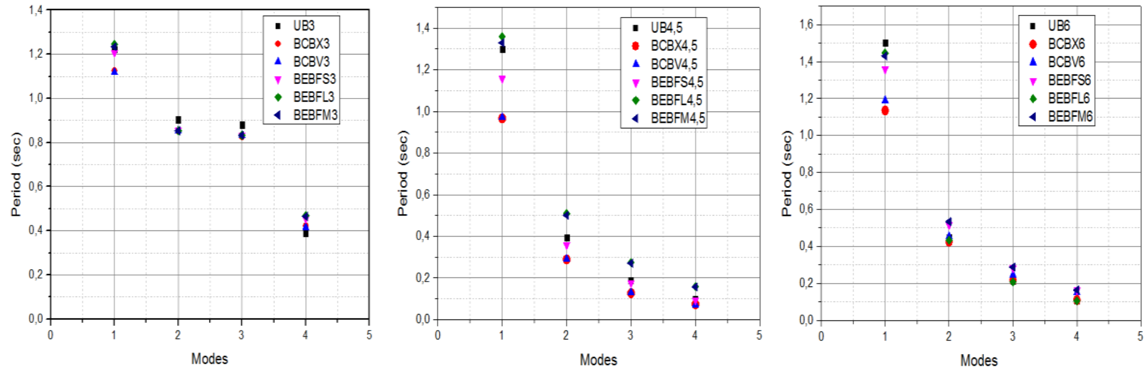


Figure 4. **31** Samples of Modes vs. Period variations results for four storeys structures

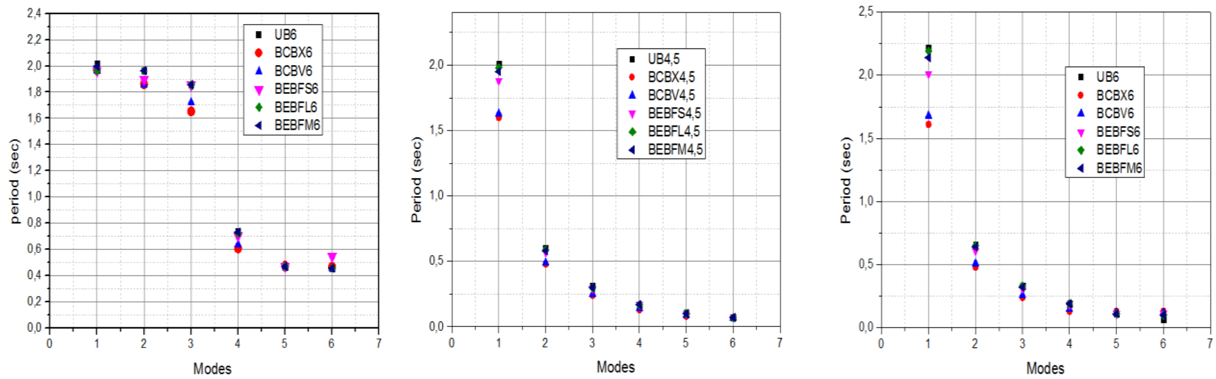


Figure 4. 32 Samples of Modes vs. Period variations results six storeys structures

Two soft-wares have been used in this study: LUSASver.14, which uses the concept of lumped mass, and SAP 2000 ver.14.2, which in turn uses the concept of consistent mass, to estimate the free vibration and dynamic properties of all model studied cases. The extract values of the modal analysis for each structure are: modal properties for each mode, that is frequency, shape, modal participation factor, effective modal mass, and the determination of the number of modes to be used in the analysis, which are enough to capture 90% of mass participation can be done. It is worth to mention that the seismic mass of each storey has been determined according to RPA99 provisions.

It must be noted that, as far as the first mode of vibration is concerned, similar values for four-storeys and six storeys frames were found by the two software. However, for higher modes a certain disparity is noticeable as LUSAS and SAP 2000 use different concept of modelling masses. Figures 4.31 and 4.32 show the general tendency which seems to be similar concerning variation of periods vs. le mode number despite their topologies et the ground storey and irregular structures. The values of the first mode in irregular are, as expected higher than those of regular frames, with maximum for H= 6 m. In both results, the of equipping the first storey with a topology of bracing systems has a positive consequence on the values of the first mode values with a decrease of about 10% for centrically bracing and 5% only for eccentrically bracing for four-storeys frames. While for six-storeys irregular frames, present roughly the same values as the regular frames indicating the decrease of the effect of braces for H= 4.5 m and 6 m respectively. The gap between the values of the periods for the different structures considered begins to decrease for the so-called higher modes.

Referring to Figures 4.31 and 4.32, the modal analysis results show a certain effect on the fundamental mode and of the reinforcement at the first floor especially for four-storeys frames depending on the used topology. However, this effect seems to be less effective as the number of storeys rises, i.e. six storey-frames. Regarding the participation coefficients, the structures arrive to 90% requirements in most cases in the first mode or in second or third mode and then meet the RPA provisions. The contribution of braces can be noticed by decreasing the period for all mode shapes from CBX to EBF.As can be seen in Figures 4.31 and 4.32, the variation pace seems, generally, the same for all studied structures. It must be as expected, regular unbraced frames UB has the greater values of the periods in mode function compared to other structures.

- **Static linear analysis**
  - **Equivalent Lateral Forces Method**

The concept LFM is attractive as an alternative widely used approach, because it converts a dynamic analysis into partly dynamic and partly static analyses for finding the maximum displacement (or stresses) induced in the structure due to earthquake excitation. Evaluation of seismic design shear using the 'lateral forces' method (Algerian Earthquake Resistant Regulations (RPA99/Version 2003). Although this is a poor approach, which gives no information on the periods of vibration of the scheme and on the effects of higher modes of vibration, its simplicity and physical evidence makes it an important tool for evaluating seismic elastic response, provided that the effect of the first mode of vibration is prevailing in the scheme and proper integrations or corrections are applied when necessary. One constant R-value is assigned for all the buildings having the same lateral load resisting system in the seismic codes, even their geometries, locations and weights are different.

Seismic storey displacement

As and as expected, the shape of the elastic seismic displacement is not truly linear for structures with weak storey as it is for regular structures, particularly with structures having 6 m as first floor column. It is worth mentioning that for the first storey, the results obtained herein show an irregularity of the curves representing the drift storey from regular structures up to the irregular ones. This will be more significant when the number of storeys became bigger.

- **Drift-storey**

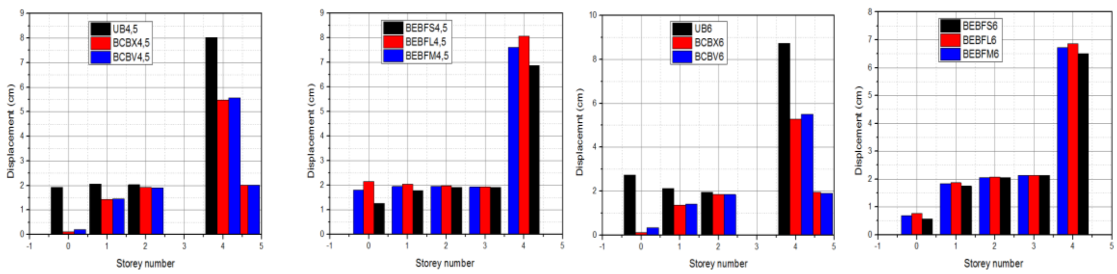
The most influential and accurate parameter to demonstrate the structural behaviour is the inter-storey drift ratio. The inter-storey drift ratio is defined, in all of the current earthquake codes, as the inter-storey displacement divided by the storey height. This definition is same It can be said that changing the kind of first-storey from concentric to eccentric in regular and irregular multi-stories structures has some significance in the behaviour of structure in terms of drift-storey and shear forces.

Results of LFM show that the limit drift displacement at the top of structure is being exceeded 1% storey's height in most studied structures, especially when in presence of an irregular first storey height, that is 4.5 m and 6.0 m respectively. However, the presence of any kind of bracing has certainly reduced the drift-displacement, depending on the type of bracing placed, the drift storey in terms of the shear forces. As it can be seen in figures 4.33 as the storey height increases from 3, 4.5 to 6m, the gap of the drift displacement increases, and the concentric braces shows better resistance as it undergoes less storey-drift displacement and EBF bracing show poorer contribution especially long link EBF.

The contribution of bracing is significant especially when using a CBX, and seems to lose this effect when the number of storeys increases. In other word, the presence of bracing at the first level seem to be efficient for lower structures, that is four storeys structure, then it is the case for six storeys frames. Also, it can be noticed that the value of drift is higher with UB frames and BCBX the smallest value which indicts the effect of braces.

It is also observed that, for the irregular structure models with the height of the first stories 4.5m and 6m, maximum inter-storey drift ratios are observed in the first storey and these values are quite higher from the floor above, as expected. On the other hand, the maximum inter-storey drift ratios for the regular models (H = 3m) are observed in the first and second stories which are close to each other.

A certain disparity in the drift-storey values can be clearly noticed from Figures 4.33. This is due essentially to the presence of stiffness irregularity located in the ground-storey. According to relative values of all limitations, it can be concluded that RPA99's provision of storey drift are being satisfied in most cases except for the top floors where these limitations where exceeded to lead to a (P-Delta) effects as stipulated in RPA99 recommendations.



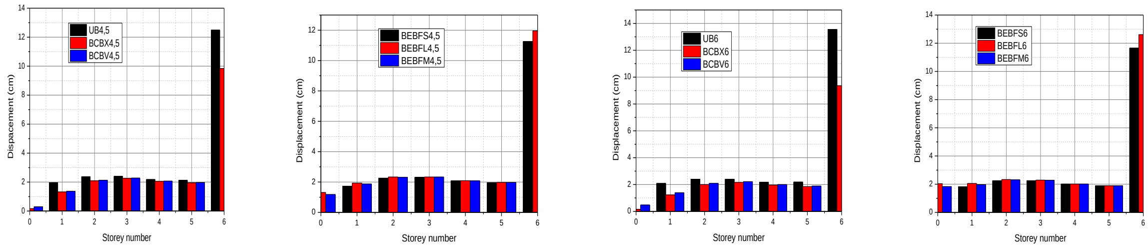


Figure 4. 33 Variation of the drift-storey vs storey number for for different configurations of irregular structures 4, 6 storeys frames

- **Response spectrum analysis**

Response spectrum represents the maximum response of a single degree of freedom (SDOF) system to a given in put motion, as a function of natural frequency and damping. An elastic response spectrum is a simple plot of the peak response in terms of acceleration, velocity or displacement of a series of frequency which are forced into motion by the same base vibration. Details on this particular method can be found in Chapter 3. The utility of the response spectrum lies in the fact that it gives a simple and direct indication of the overall displacement and acceleration demands of the earthquake ground motion, for structures having different period and damping characteristics, without needing to perform detailed numerical analysis.

Broadly speaking, the same conclusions can be drawn from the results of spectral analysis as it was previously discussed with the results of LFM. Obviously, the values obtained in the spectrum analysis are less than those obtained in LFM, in the same way as in the previous section.

- **Comparison**

First of all, it must be mentioned that for both Lateral forces method and spectral analysis, the same remarks can be thought about the results, with some disparity of course.

In general, conservative results of LFM for buildings of small to medium height characterized by regular distribution of mass and stiffness as the structural behaviour is assumed to be governed by the fundamental period of vibration and not significantly affected by contributions of higher vibration modes.

LFM analysis of plane frames provides safe results, compared to the spectral one, if mass and stiffness do not abruptly vary along the height. Results from this analysis confirms the well-known fact that, for frames having more than one storey, the base shear-force used in static analysis is quite larger than the corresponding value obtained by means of modal analysis (from 10 to 40% more).

Also, results show that the variation of drift storey in elevation obeys almost to the same function shape for both multi-stories structures with gap values for lower structures, and clearly show the effectiveness of the contribution of the bracing in structures with weak/soft storey. When the requirements to perform a simple lateral force, analysis are not met higher modes have to be included in the analysis.

The modal response method is a generalization of the previous method. It is also a linear method in which the inelastic behaviour is considered in design through the use of the behaviour factor. The method of modal response spectrum is used when the effects of higher vibration modes contribute significantly to the structural response. The sum of the effective modal masses should represent at least 90% of the total.

4.9.4 Inelastic behaviour of structures having irregularity in stiffness by pushover analysis

- **General**

Broadly speaking, the pushover analysis can be defined as an analysis wherein a mathematical model directly including the nonlinear load-deformation characteristics of individual components and elements of the structure shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement" is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. The structural Pushover analysis assesses performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm.

The seismic demand parameters are storey drifts, global displacement (at roof or any other reference point), storey forces, and component deformation and component forces. The analysis accounts for material inelasticity, geometrical nonlinearity and the redistribution of internal forces.

- **Pushover using SAP2000 NL**

Pushover-based seismic evaluation is now able to directly calculate the nonlinear seismic demand and evaluate its consequences on the structures, but also to reveal weaknesses in the elastic design performed previously in order to ensure the structural integrity. Applied Technology Council-40 (ATC-40), 1996 and Federal Emergency Management Agency (FEMA), 2002 proposed a simplified nonlinear static analysis (Pushover Analysis) procedure. The central focus of the simplified nonlinear procedure is the generation of the “Pushover” or Capacity curve.

The nonlinear static (Pushover) analysis has become increasingly popular in structural applications around the world. Numerical tools like SAP 2000 developed by Computers and Structures Inc., which can perform the pushover analysis. As nonlinear analyses can offer greater insight into the behaviour of the structure and determine if the structures satisfy performance requirements, pushover analyse implanted in SAP2000 program has been used. Default SAP2000 hinges, and target displacement are used in the analysis.

Target displacement: The target displacement is intended to represent the maximum likely roof displacement of the structure at its centre of mass during the design earthquake associated with a certain performance level. as defined in the previous section, the value taken in this study is SAP2000 NL defaults value.

Plastic Hinge Locations: For this research work, the chosen method to model nonlinearity is through discrete flexural plastic hinges because it is commonly used in academia and practical design. Modelling hinges provides an advantage with valuable insight into the structure's response to loading. However, default discrete plastic hinges were also selected in SAP2000 NL

Performance point: As mentioned earlier in section 4.8.5 in this thesis, the demand is given by the amount of displacement at the performance point in the capacity spectrum method and the target displacement in the seismic coefficient method. Member displacements and forces computed at this displacement are used as the inelastic demand and checked against available capacities. To determine the inelastic deformation demand, two widely used approximate procedures, implanted in SAP2000 NL that rely on pushover curve of the structure exist. These procedures, namely the Capacity Spectrum Method of ATC-40 and the Displacement Coefficient Method of FEMA 356 generally use a bi-linear representation of the original pushover curve to compute the approximate inelastic displacement demand.

Overstrength ratio

Overstrength of a structure due to its inelastic behaviour under lateral loading can be assessed by numerical analysis. Non-linear static analysis procedure (pushover) is a functional tool for estimating the lateral yield strength of the structure. Significant yield point is determined by idealizing the resultant capacity curve of the structure.

Base shear vs. displacement graph is reformed as a bilinear curve where the deflection point of the curve (start of the non-linearity) indicates the yielding point and the yield base shear is obtained. Values of the overstrength ratio obtained in this particular study are all exceeding the value 2, and depending on the kind of frames.

Due to lower values of the base shear in spectrum analysis, the values of overstrength ratio are higher than those coming from lateral forces method, sometime twice and even triple. This leads to the conclusion that the frames considered are somehow well designed at least as far as the elastic behaviour is concerned.

Pushover curves

As stated before, the pushover analyses are performed on the analytical models in order to investigate the inelastic behaviour of building structures having soft stories using SAP2000. The global pushover curves of all analytical models are given in Figures 4.34 and 4.35. Roof displacement versus base shear diagrams for each increment load pattern which will illustrate the pushover curve, or a capacity curve described in the section 7.7 which is a representation of the inelastic behaviour of a SDOF system expressed as base shear vs. roof displacement as shown in Figures 4.34 and 4.35 for four and six-storeys buildings respectively. The storey's capacity curves of structures are presented in Figures 4.36 to 4.41, highlighting the effect of braced ground storey, for different structure configurations, and once again for four and six-storeys structures respectively.

Global pushover curves

Figures 4.34 and 4.35 show the obtained overall capacity curves for four and six storeys frames. As it can be noted, all frames possess an actual capacity considerably higher than the one assumed in design. In addition, it can be noticed from both figures that the effect of reinforcing the ground storey has a beneficial effect on the global behaviour of four storeys frames than it is for six storey frames, where the presence or not does not affect considerably the lateral stiffness of the structures. In the evaluation of the global behaviours of the analytical models, the pushover curves are compared by considering the number of storeys and the bracing system adopted for the ground storey.

The analyses have shown that the use of any kind of bracing in the ground storey is effective in providing overall a reduction in the roof displacement and base shear forces with large design overstrength.

For the braced frames, the use of concentrically and eccentrically braces ensured that plastic hinges occurred in the diagonal with large brace ductility demand, mainly for the braces in compression. Because failure of ground storey indicates total loss of strength for the whole structure, monitoring that storey behaviour is, for the sample frame, a critically important measure of limit state attainment for the entire building.

For ductile multi - storey frames, e.g. with weak - beam strong - column, storey drifts are proportional to beam rotations. As well-known excessive inter -storey drifts are indicators of structural failure, such as weak storeys. Shear deformations of beam - to – column connections significantly contribute to horizontal drifts.

The deformed shape of the building, which is also included in the figure, confirms, at least, for some structures UB six storeys in particular, as the de storey displacement along levels is no more linear, the occurrence of the failure by weak storey, at the first floor. Assuming that lateral force does not increase as the displacement increases, weak storey behaviour occurs when the capacity curve shows a descending branch as displayed in Figures 4.34 and 4.35. If the ground storey loses its strength this will naturally affect the second or third storey failure. Therefore, the failure occurs at ground floor. At maximum base shear, all the columns of the first-storey exhibit plastic hinging at both ends.

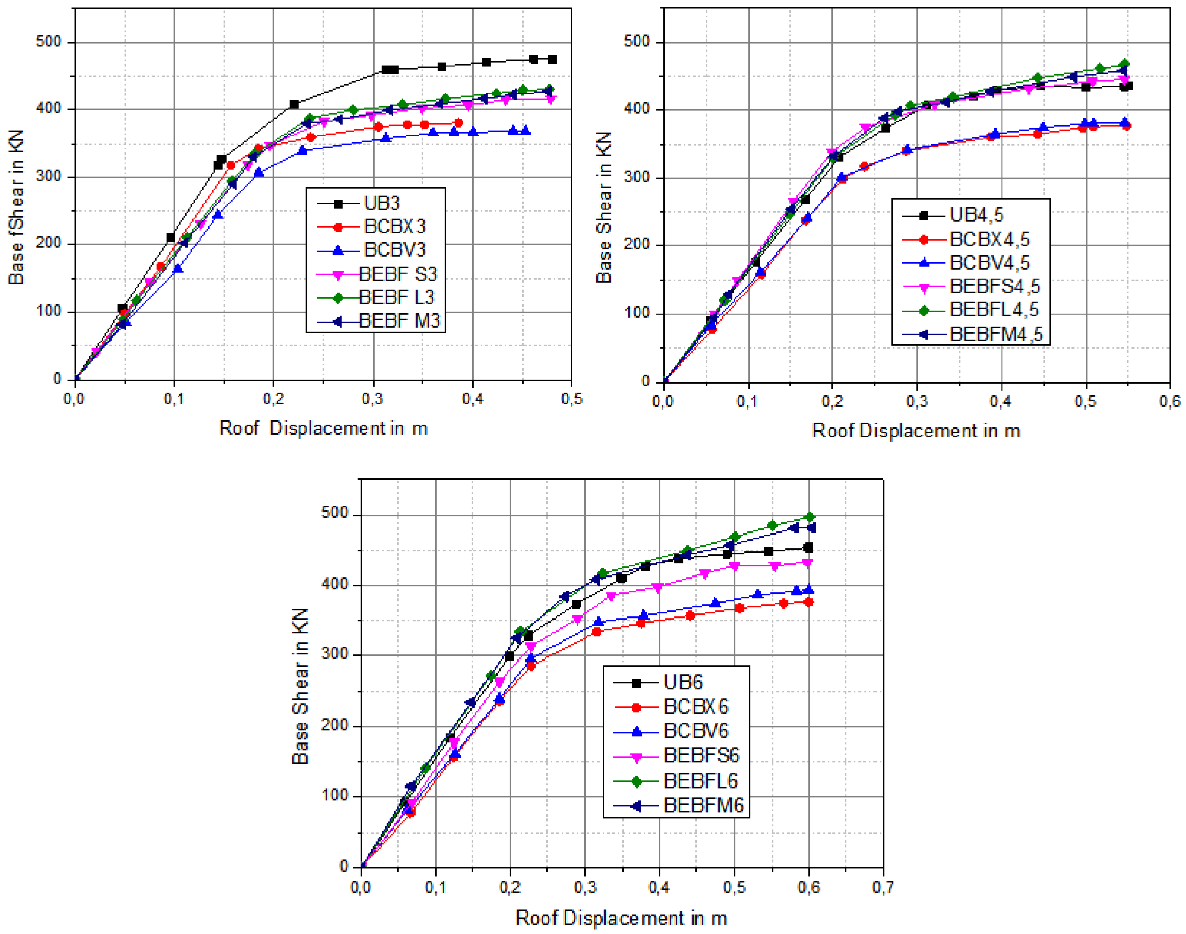


Figure 4. 34 Pushover curves for different configurations of four storey frames.

The comparison of response curves allows highlighting the influence of the investigated parameters. In particular, when it comes to the influence of number storeys, it can be noted that the four-storey frames experience larger V/V_d ratios than the eight-storey frames. This implies that the smaller the number of stories, the larger the design overstrength.

The ultimate plastic hinges distribution is obtained when the frames reached the maximum base shear and horizontal displacements. Both are observed, the effectiveness of capacity design criteria and the influence of the building aspect ratio (namely the ratio between the building height and the width), in relation to the damage evolution. The former is evidenced by the weak-beam/strong-column behaviour, being all plastic hinges located on ends of beam and column base as adopted in SAP 2000.

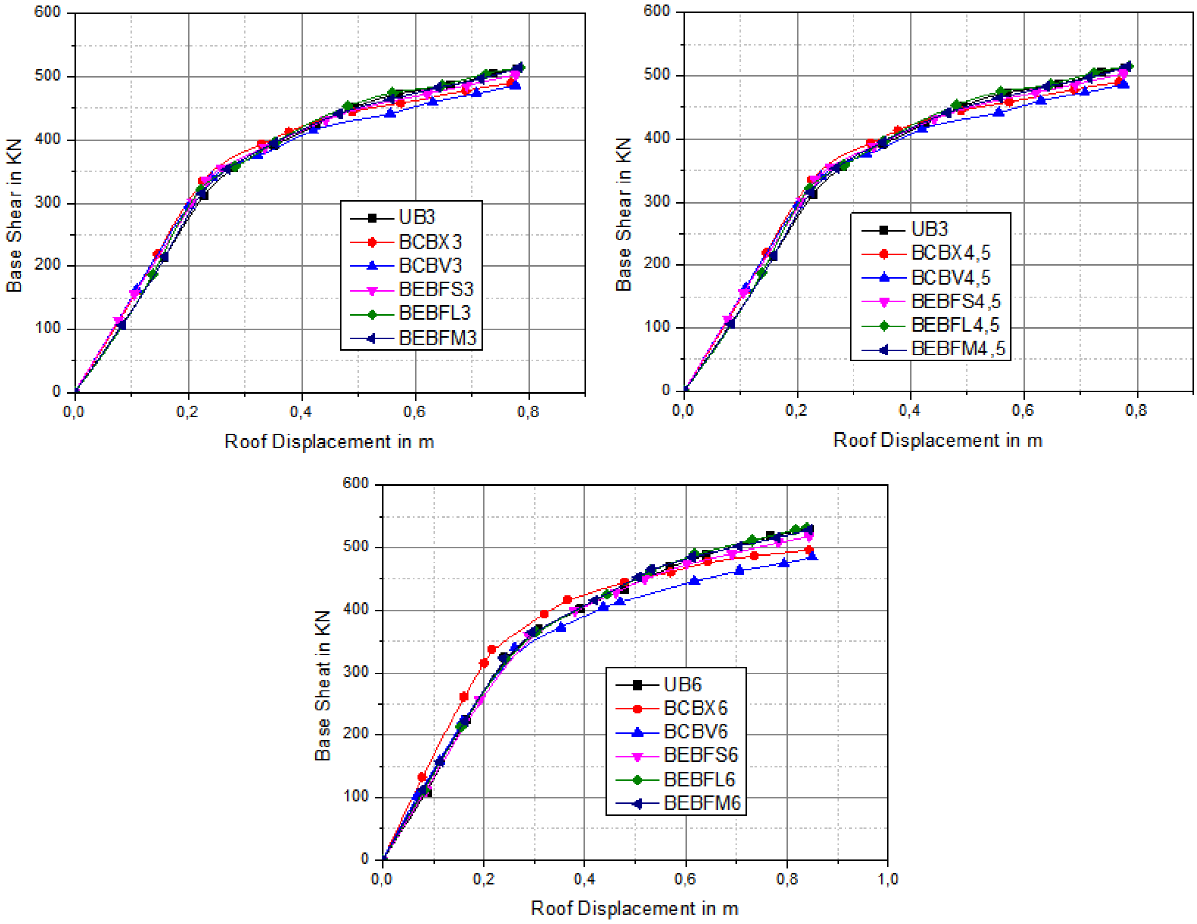


Figure 4. 35 Pushover curves for different configurations of six storeys frames.

- **Storey pushover curves for different configurations (4 storey)**

For analysis, a mathematical model is displaced by monotonically increasing lateral force or displacement, in gradual discrete increments (pushing the structure), until either a target displacement is exceeded or the collapse condition of a building is reached. At each discrete interval, the effect of existing gravity loads is taken into account.

The base shear at each incremental loading is plotted against corresponding lateral roof displacement. Also, at each stage, the resulting internal deformations and forces are determined and recorded, i.e. in the pushover curve, sequential elastic analysis is inbuilt. The resulting load displacement relationship is termed the capacity curve.

Investigating the storey pushover curves obtained for various frames values, the ultimate lateral displacement values are observed to be distributed regularly among the inelastic range for the first two or three stories for the regular models. This shows that the lateral forces are transmitted to the stories above. For the models having vertical irregularity in stiffness with a potential formation of a soft or weak storey, the displacements are observed to be concentrated to the first storey due to the dominant first storey behaviour. The structure cannot transmit the lateral forces and due to this, lateral loads concentrate of that first storey.

Additionally, in the regular analytical models, the plastic hinges which are formed at the first storey, are not very effective on the other storey's responses. The presence of bracing gives a more strength to frames as the base shear force decreases depending on the bracing type. It has been found in this particular study that the eccentrically bracings**;** BCBX and BCBV have better performance than the reference model which is the unbraced frame UB with and without irregular height of the ground storey.

Storey shears and storey displacements were extracted and plotted in figures 4.36 to 4.38 from pushover database at each step of pushover analyses for all lateral load patterns. Theses storey pushover curves (storey shear vs. storey displacement) were developed to illustrate the variation in storey shears due to the height-wise distribution of lateral load pattern. Also, the absolute maximum values of storey shears and storey displacements experienced under ground motion excitations were determined for each deformation level to approximate a dynamic storey pushover curve for case study frames.

Storey pushover curves for different configurations (4 storeys) are shown in curves 4.36 to 4.38, while storey pushover curves for different configurations (6 storeys) are illustrated in Figures 4.39 to 4.41. In the following, Figures are relative for:

-Four storeys frames Figure 4.36 to 4.38 Storey's pushover curves regular UB; BCBX; BCBV; BEBFS BEBFM BEBFL frames h = 3 m irregular h = 4.5 and h = 6 m respectively;

-Six storeys frames Figure 4.39 to 4.41 Storey's pushover curves regular UB; BCBX; BCBV; BEBFS BEBFM BEBFL frames h = 3 m irregular h = 4.5 and h = 6 m respectively.

The comparison of response curves allows highlighting the influence of the investigated parameters. In particular, when it comes to the influence of number of storeys, it can be noted that the four-storey frames experience larger V/V_d ratios than the eight-storey frames. This implies that the smaller the number of stories, the larger the design overstrength.

The lateral displacements are clearly seen to be in increasing order from zero to the ultimate displacement value for the regular, but in models with irregular ground storey, the maximum lateral displacement values are observed to be quite close to the ultimate displacement value.

In all figures, it can be noticed that each curve has two branches: linear and nonlinear parts, being more significant in upper storeys and for irregular frames specially. From Figure 4.34 an undeniable effect of reinforcing the ground storey by means of bracing system is remarkable for four storeys structures. However, for six storeys frames, this influence is getting weaker as can be noticeable from Figure 4.35.

Curves relative to the first storey that it behaviour in elastic manner for regular structures and a small part the curve is nonlinear, which becomes more significant as the height of the ground storey is getting larger changing the ratio (l/h) from 1, 1.5 to 2.0 which, as mentioned in previous sections, has a certain influence on the global behaviour of frames. As expected, for the upper storeys, a more visible linear and nonlinear curves were obtained, from the second storey up to the last storey, especially for irregular frames. It can also be noticed from figures that the move from elastic linear behaviour to nonlinear for all storeys occurs almost at the same loading level for all studied frames. The target displacement is reached with slightly less values or base shear forces.

Also, according the obtained results in this study, reinforcing the first floor with EBF bracing system seems to give good results, especially with long links, compared to other kind of bracing, as it mitigates the severity of the curve and it does not take space and does not interfere the traffic in a commercial building for example.

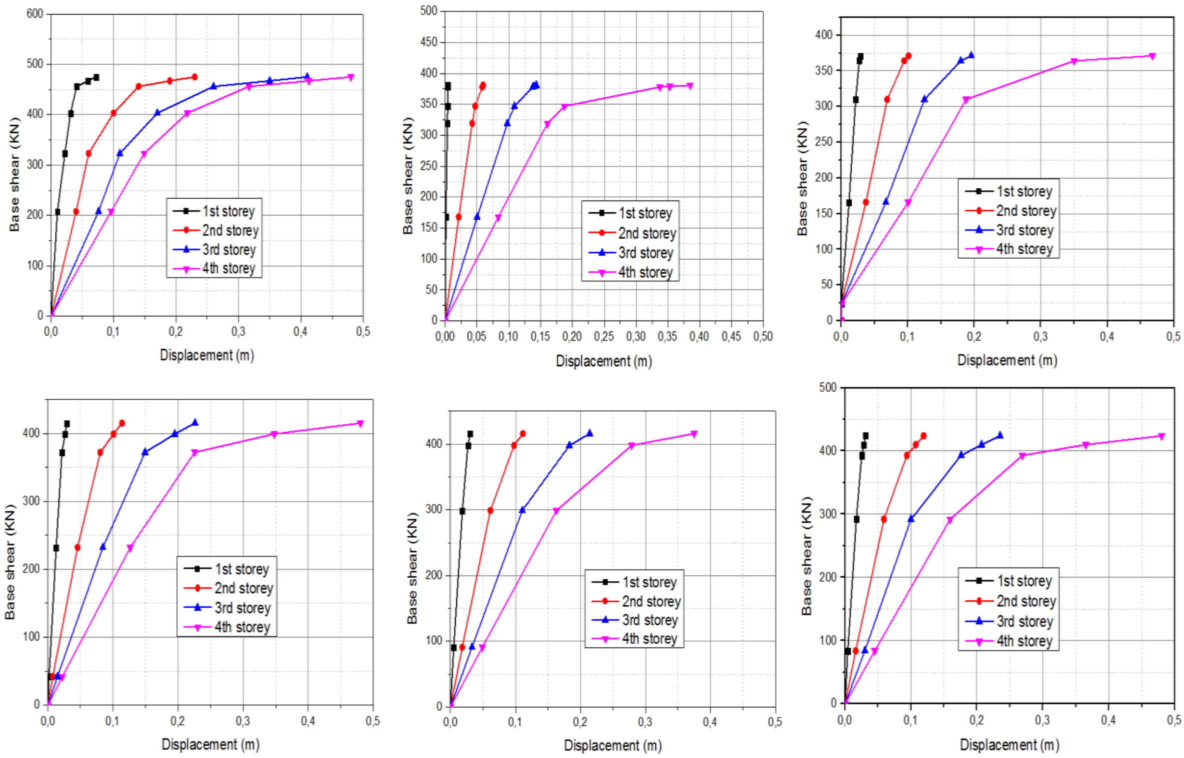
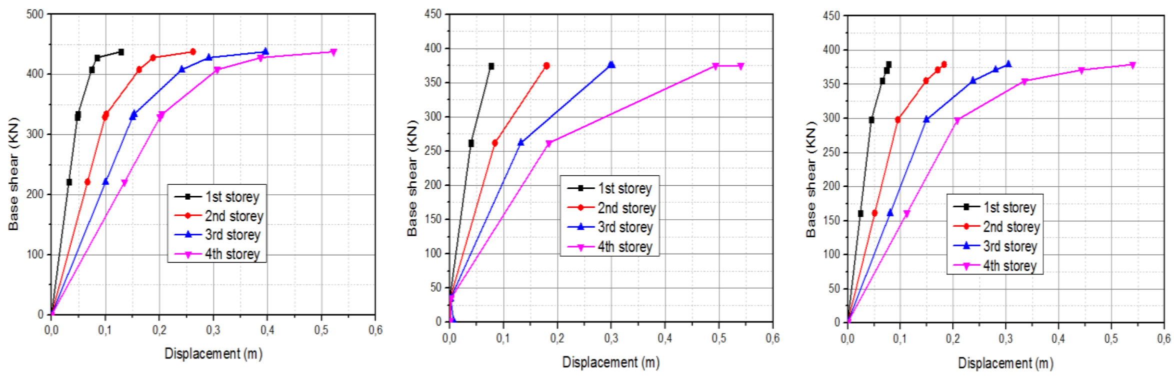


Figure 4. 36 Storey's pushover curves for four storeys regular UB3; BCBX3; BCBV3; BEBFS3 BEBFM3 BEBFL3 frames h = 3 m respectively



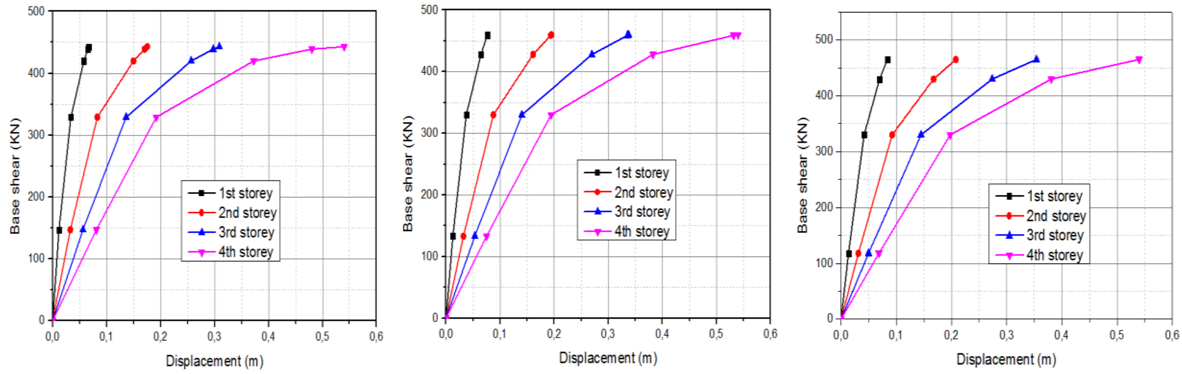


Figure 4. 37 Storey's pushover curves for four storeys irregular UB4.5, BCBX4.5; BCBV4.5; BEBFS4.5 BEBFM4.5 BEBFL4.5 frames h = 4.5 m respectively

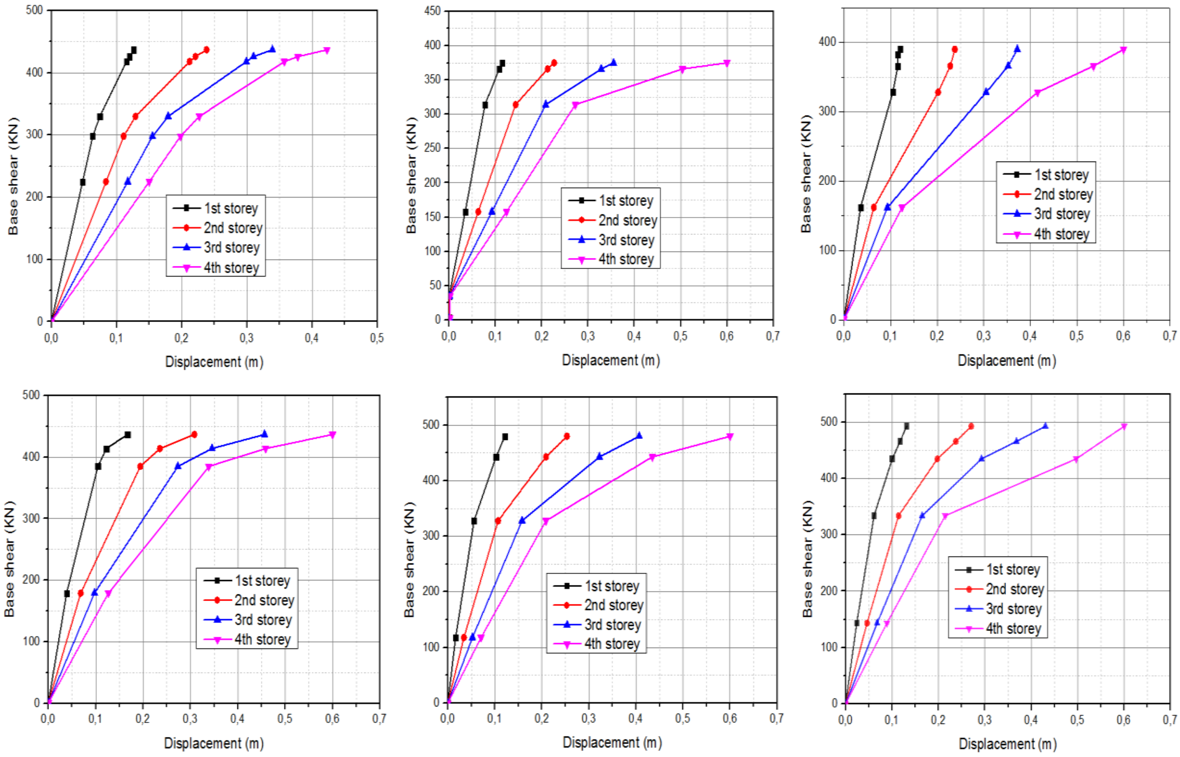


Figure 4. 38 Storey's pushover curves for four storeys irregular UB6; BCBX6; BCBV6; BEBFS6 BEBFM6 BEBFL6 frames h = 6 m respectively

For six storeys frames, curves have in all configurations, clear linear and nonlinear branches can be noticed. Same remarks concerning Loads- deflections curves can be made as above, except that curves resemble each other with roughly the same slopes and which means, once again, that the influence of bracing system is getting weaker.

From this analysis, it can be concluded that as the considered structures are getting higher, consequently slender, the impact of reinforcing by means of any kind of bracing becomes more and more insignificant, as it was demonstrated in upper sections when the performance of multi-storeys structures with full bracing were compared. However, the idea of reinforcing the irregular ground storey with bracing for low-rise structures gives a better performance and may lead to avoid soft weak-storey mechanism and limits the drift-storey values to satisfy the RPA99 limitations.

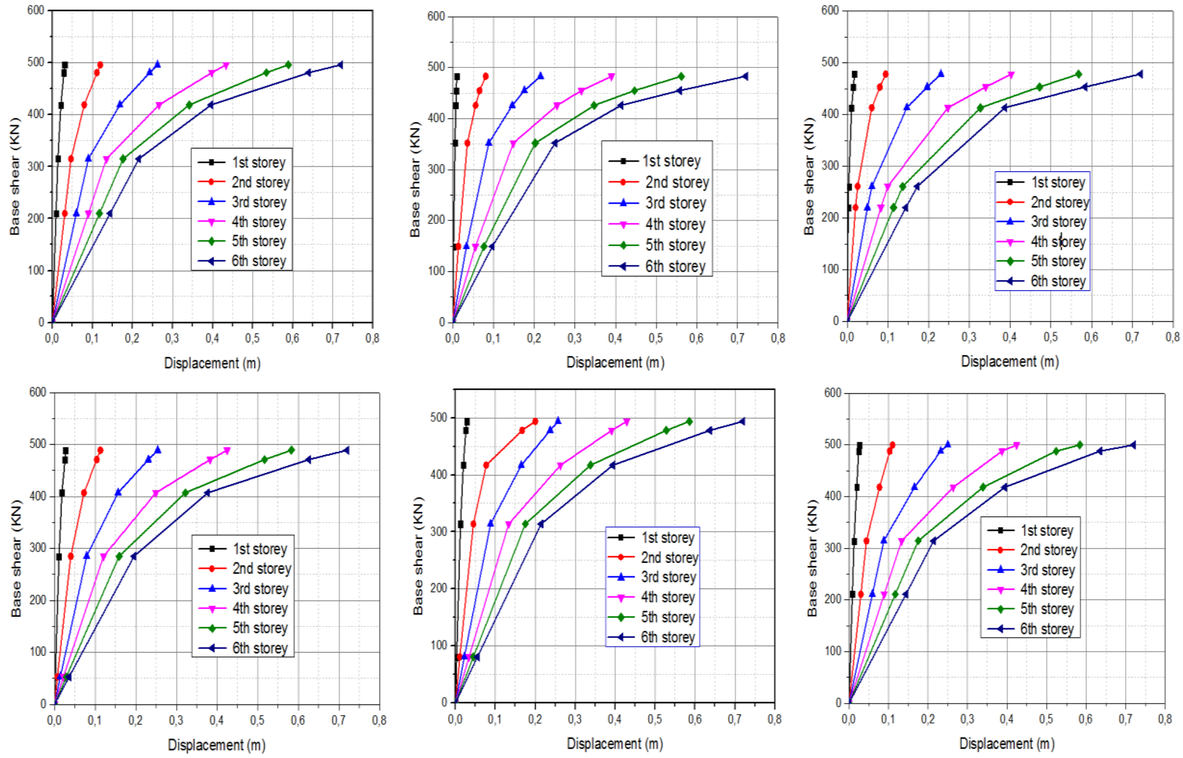


Figure 4. 39 Storey's pushover curves for six storeys regular UB3; BCBX3; BCBV3; BEBFS3 BEBFM3 BEBFL3 frames h = 3 m respectively

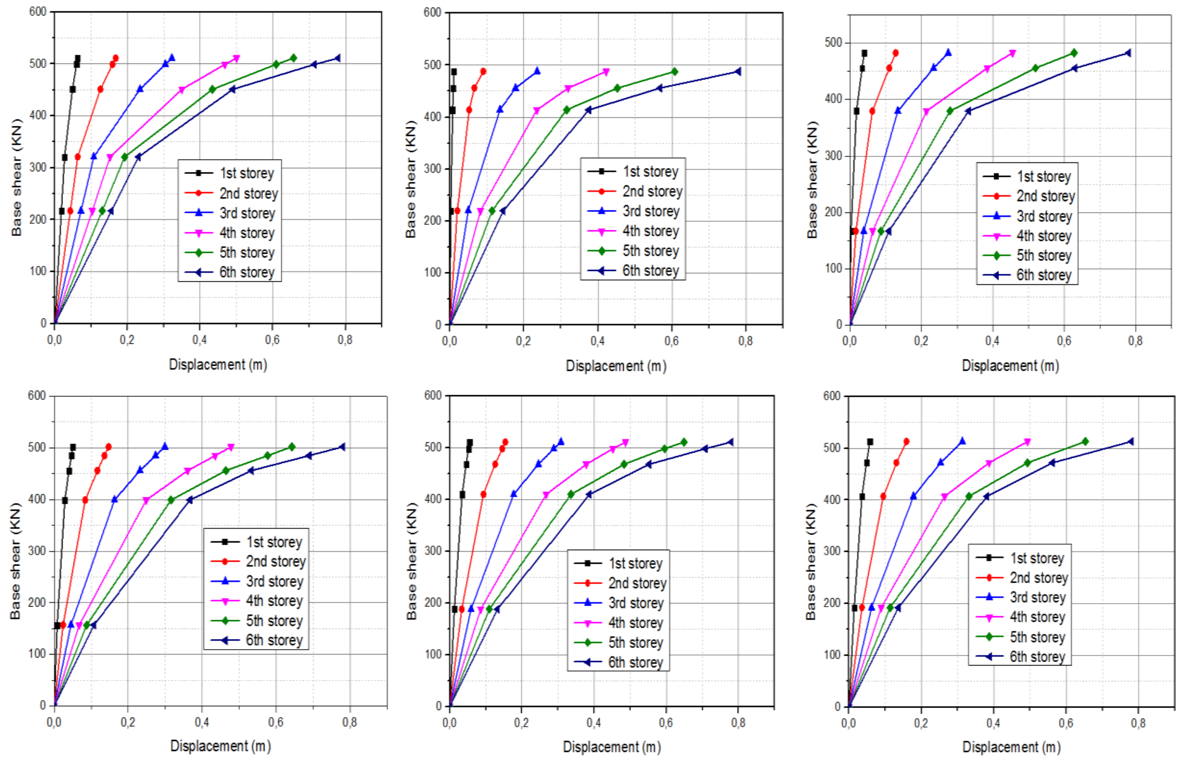


Figure 4. 40 Storey's pushover curves for six storeys irregular UB4.5, BCBX4.5; BCBV4.5; BEBFS4.5 BEBFM4.5 BEBFL4.5 frames h = 4.5 m respectively

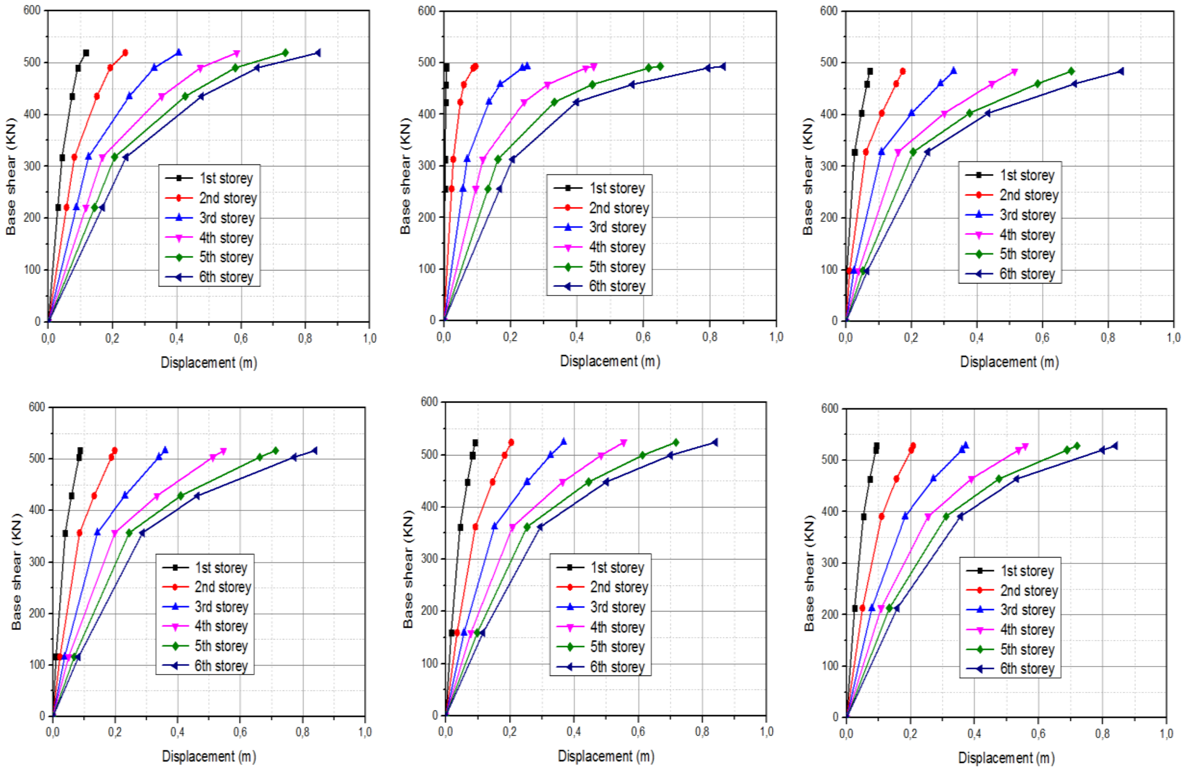


Figure 4. 41 Storey's pushover curves for six storeys irregular UB6; BCBX6; BCBV6; BEBFS6 BEBFM6 BEBFL6 frames h = 6 m respectively

4.9.5 Summary of discussion of overall results

In order to highlight the results obtained in this particular study, more detailed results will be provided in an international paper, which at the present time in progress and be finished in very near future.

In view of the results obtained by the linear and nonlinear static pushover analyses of the considered 2D structures, the following primary conclusions on the prediction of the linear and nonlinear behaviours of the models are obtained. In the examined cases namely within the range 3 to 6 m storey height to assess the influence of irregular ground storey which is found to be significant.

The soft-storey irregularity effects on the structures are observed to not be very pronounced, although all of the models are designed according to the current design codes where weak beam-strong column concept is satisfied. As an expected behaviour, the force and displacement demands are concentrated at these soft storeys. Another important conclusion is that even if the shear frames do not reveal the soft storey behaviour during the elastic range, however in the inelastic range soft storey behaviour may be expected and can easily be observed and collapses of these models always occur due to this behaviour if subjected to major earthquake.

To reduce this problem of deformability which is a characteristic of a “soft-floor” type of collapse mechanism that may lead to global instability due to second-order effects, a bracing system of the uprights is placed at the ground floor, that is to say where the soft weak-storey is potentially attended and consequently, models show a higher stiffness. The significant variation of column stiffness along the height causes a 'soft-storey’ in the irregular frame; large drifts are observed at the second floor.

The main conclusion which may be drawn that placing bracing systems at the ground storey has, to some extent, reliefs and improves the linear and nonlinear behaviours of mid or low-rise structures than it is for slender frames, that is six storeys and up.

The models with fewer stories system are found to be more vulnerable considering the soft- storey irregularity due to the design considerations but when equipped with a bracing system, this certainly improve the resistance to soft-storey mechanism. The ultimate deformation and force capacities are observed to be higher for the models with four stories than the six storeys frames.

Although the strong column-weak beam concept is applied for all of the models, which helps in improving the overall behaviour, the plastic hinges are not spread along all stories of these models. It is observed that for each building model, the plastic hinges along the height of the models are spread along the first two or three storeys from the base. Even the weak beam-strong column concept affects the behaviour as expected, the distribution of plastic hinges to even one more storey, leads to very big amounts in increase of the ultimate force level.

For modal analysis, LUSAS and SAP2000 give roughly the same figures of the eigen values for the first (fundamental) mode, SAP2000"s are slightly higher, and that the presence of first-ground bracing contributes can be noticed by decreasing the period for all mode shapes from CBX to EBF long link, especially for four storeys frames. From elastic seismic analyses, it can be noticed that the shape of the elastic displacement is not really linear for irregular structures, and a kind of sway can be detected particularly with for UB structures having 6 m and six stories as first floor column, which constitute potentially a soft-weak storey in a major earthquake whereas this study is carried out in a medium seismic zone.

The contribution of bracing is significant especially with CBX which reduces the drift storey especially for lower storey, where is potentially the location of a soft-storey mechanism.

The presence of concentrically braces influences more significantly in the overall behaviour of structures when subjected to lateral forces than it is the case for others in terms of lateral displacements and storey drifts, eccentric braces the displacement and drift are increasing. Also, it can be noticed that the value of drift is higher with UB frames and BCBX as the smallest value, the other bracing systems values are ranging between these values which clearly indicts the effect of braces.

As expected, for models with irregular height of the first storey, that is 4.5 m and 6 m, where the maximum inter-storey drift ratios are observed in the first storey and these values are quite higher from the floor above, and higher the limit as specified by code in linear dynamic analysis, which needs the checks out of (P-delta) effects.

According to relative values of all limitations, it can be concluded that provision of storey drift and lateral stiffness are safer.

As far as the nonlinear behaviour of frames is concerned, as in literature review has suggested that use of a pushover analysis of the steel frame is feasible, and is applicable to all types of buildings, they may be they regular or irregular in plan and/or elevation, the pushover curves are compared by considering the storey numbers, and the first storey height with its bracing configurations. In terms of the storey numbers, it is observed that as the number of floors increases, the ultimate force and deformation levels of the models increase.

The initial slopes of two-branched pushover curves are slightly same for all structures, with disparate displacement values depending on the storey number an undeniable, as it was for the elastic analyses, effect of bracing is felt on the pushover curves especially for four structures, with sensibly less consequence on six storeys frames.

The base shear forces acting at first level unbraced structures whether they are regular or irregular frames are higher with a larger displacement as the frames are slender which generates, naturally, a larger drift-storey values and consequently, the formation of soft weak-storey mechanism being more probable. In addition, it was also observed during the pushover process that, the calculated lateral displacements when the first plastic hinge is formed are increasing proportionally with the number of floors.

In the comparison of the pushover curves obtained by the various lateral stiffness configurations, it is observed that the force and deformation estimations for the models with regular storey heights is higher than the others models with first storey heights of 4.5 and 6 m, the pushover curves obtained by various load patterns are observed to be the same. This behaviour is due to the soft storey's governing displacement and force behaviours on the global structure. Storey pushover curves are also obtained for each storey including the roof displacements.

In the regular analytical models, the plastic hinges which are formed at the first storey, are not very effective on the other storey's responses and on the collapse mechanism especially for the unbraced models. The results obtained in this study, as expected, that unlike the UB frames, the location of plastic hinge at the beginning at beam then column. Moreover, for the frame with bracing the location is at the brace, then beam lastly at the column for all studied braced ground storey.

One of the best methods to overcome the problem of soft weak-storey may be to increase the stiffness of the lateral load resisting system, i.e. braced ground storey as discussed in this section. Another method about increasing the stiffness of the lateral force resisting system is designing the columns and beams of the soft stories according to very high seismic demands by utilizing an amplification factor only for these stories, consequently with stronger cross section classified as first class section to CCM 97 and EC3. It has also been demonstrating through this analysis that adopting weak beam vs. strong columns, which has to be clearly stated in future RPA99 code leads to better results to prevent collapse mechanism due to ground storey or weak storey phenomenon.

Another reason, in the design phase of building structures with soft storeys, the dynamic analysis procedure must be used and the member forces must be carefully evaluated in order to obtain the required life safety performance level in all of the current RPA99 code, either to be used in the design of the new buildings or in the evaluation and retrofitting of the old buildings is certainly insufficient in predicting the seismic demands accurately.

CHAPTER 5:

SEISMIC LINKS IN EBF STEEL STRUCTURES

CHAPTER 5: SEISMIC LINKS IN EBF STEEL STRUCTURES

5.1 GENERAL ON EBF STEEL STRUCTURES

Despite the fact that Eccentrically Braced Frames (EBFs) have been worldwide accepted and used as a seismic load resisting system, primarily in buildings. In addition to the absence in RPA National Seismic Code **(RPA 99, 2003)** of any explicit provisions concerning the nonlinear analysis of steel structures, as fully discussed in previous chapters when dealing with the global elastic and inelastic behaviour of multi-storeys steel structures, RPA99 do not cover either the EBF, while universally recognized and used for long time. An eccentrically braced frame (EBF) is a type of steel framing system including beams, columns and braces, where these members are arranged in a manner where at least one end of each brace is connected to isolate a segment of the beam called a link Steel eccentrically braced frame (EBF) structures, such as that sketched in Figure 5.1,were proposed in the late seventies **(Roeder 1977)** as a ductile structural system suitable for use in regions of high seismicity for the control of serviceability drift limits. EBFs are typically used as a lateral force resisting system for earthquake loading. EBFs successfully combine the high level of ductility of MRFs and the high level of stiffness of CBFs by introducing eccentricity "e", between a frames cross bracing and column **(Popov 1988)**.

The cross brace of an EBF provides the elastic stiffness of CBF and the eccentricity of the cross brace creates a link that is responsible for the ductility and, therefore, energy dissipation capacity of MRF.

There are several reasons for selecting a frame with eccentric bracings for an earthquake resistant structure as in (**Fardis 2005)**:

- Eccentric bracings combine stiffness with a high q factor (between 4 and 8);
- Connections are between three bars, not four as in frames with concentric bracings which results in less complicated connection details, which may also simplify the erection of the structure;
- Diagonals are parts of the structural system taking gravity loads, and are considered to provide strength and stiffness against these loads.

5.2 DEFINITION AND BASIC CONCEPT OF EBF

Eccentric braced frames can be defined as braced frames where component axes do not intersect at a single point and the eccentricity exceeds the width of the smallest member at the joint. The section between these points is defined as the link component with a span equal to the eccentricity. The other components of EBFs are columns, braces and beam outside of the link. Several different configurations are possible for EBFs as dictated in Figures 5.1 and 5.2.

The main characteristic of EBFs is that the eccentric connection of brace with beam causes a weak, small beam segment named as a link.

The design intent for a seismic resistant EBF is to provide high ductility under earthquake loading by yielding of the link. Design requirements for seismic resistant EBFs in the US are specified by the AISC *Seismic Provisions for Structural Steel Buildings* **(AISC 2005)**. European EC8 (version 2005) provides details concerning the EBF structures in chapter 6 **(EC8 2004).** The code provisions of both AISC (2005) and EC8 will be discussed later in chapter V of this thesis.

When a typical EBF is subject to lateral load, the link transmits high shear, high bending moment, and typically low levels of axial force. Consequently, links will normally experience shear and/or flexural yielding during an earthquake. Other members of an EBF, including the braces, the columns and the beams segments outside of the links are intended to remain essentially elastic during an earthquake.

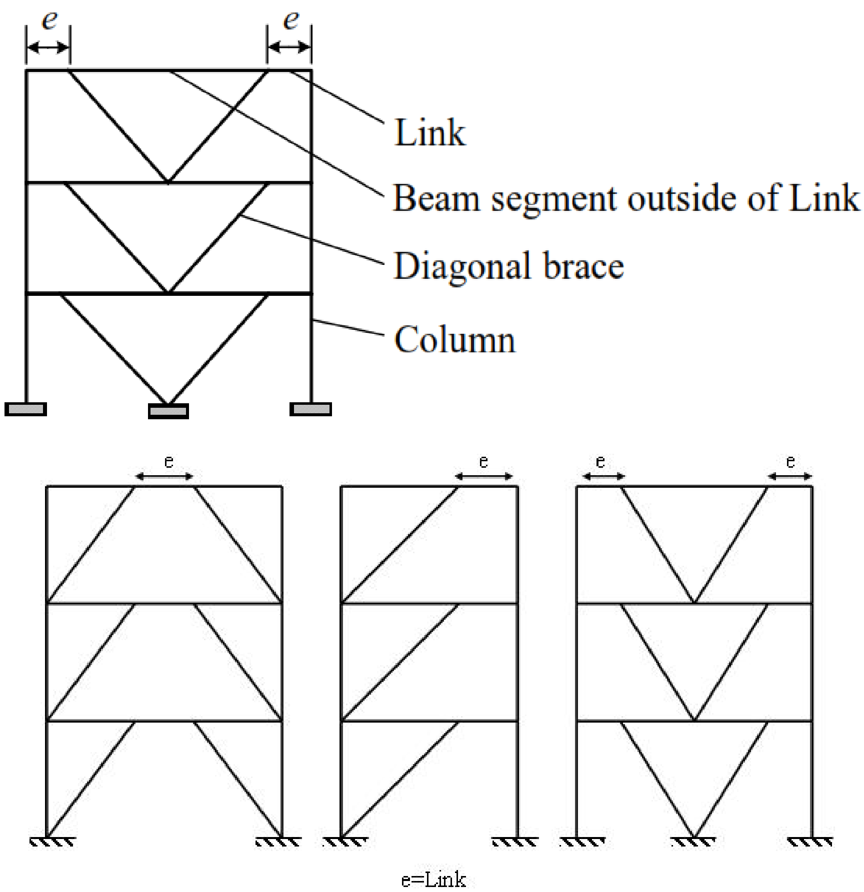


Figure 5. 1. Terminology and usual configuration of EBFs.

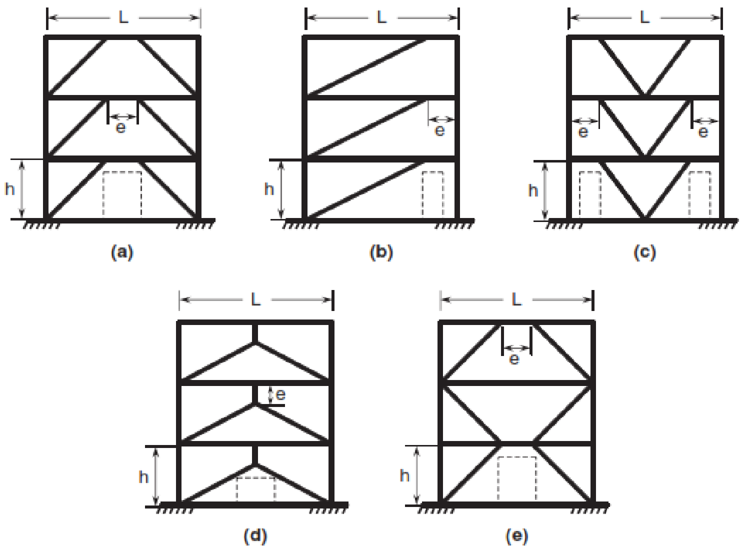


Figure 5. 2. Typical configurations of multi-storeys EBFs **(Bruneau 2011)**.

5.3 BASIC MECANISM OF EBF STEEL STRUCTURES

5.3.1 General

EBFs successfully combine the advantages of the moment frames and concentrically braced frames, namely high ductility and lateral stiffness, while eliminating the shortcomings of those frames by limiting the inelastic activity to ductile shear links and keeping braces essentially elastic without buckling, thus maintaining high lateral stiffness during earthquake events.

While steel moment frames can exhibit stable inelastic and ductile behaviour under cyclic seismic excitation, the concentrically braced frames usually possess higher lateral stiffness which can limit the damage due to drift. However, moment frames are relatively flexible and their design is usually governed by the drift limitations in order to control the damage. On the other hand, the ductility and energy dissipation capacity of concentrically braced frames can significantly deteriorate if braces buckle under seismic loading.

5.3.2 EBFs resistance to lateral loadings

EBFs resist lateral load through a combination of frame action and truss action. EBFs provide high levels of ductility similar to MRFs by concentrating inelastic action in the link, which can be designed and detailed for highly ductile response. At the same time, EBFs can provide high levels of elastic stiffness, similar to that provided by CBFs, so the code drift requirements can be met economically.

The segment of the frame generally designated by its length *e* is called the link. In EBF systems, yielding is concentrated only at link segments and all other members of the frame are proportioned to behave essentially in elastic manner. Therefore, during severe earthquakes, links can be considered as structural fuses which will dissipate the seismic input energy through stable and controlled plastic deformations.

A comparison of the expected global plastic mechanism between MRF, EBF and CBF is sketched in Figure 5.3 and showing the inelastic plastic locations in the three kinds of steel structures, namely MRFs, CBFs and EBFs.

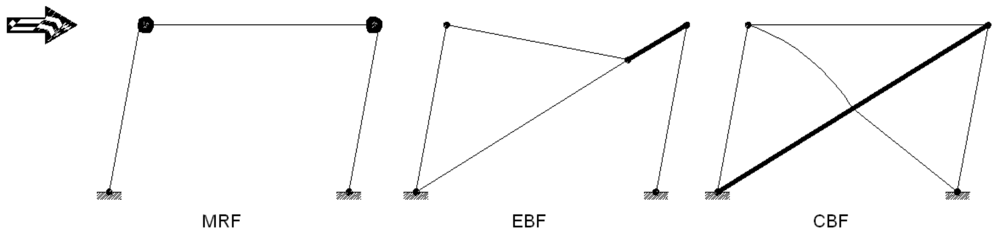


Figure 5. 3. Inelastic action locations in MRFs, EBFs and CBFs steel structures.

Figure 5.4 depicts the deformed shape of an EBF at yield. It is proposed that the storey yield drift can be estimated with account for the following three deformation components (**Sullivan 2013):**

(1) Beam (including link) deformations,
(2) Brace axial deformations,
(3) Column axial deformations.

It is also recognised that axial deformations of the beams may increase the storey drift at yield, but the effect should be relatively limited and is difficult to estimate since the axial stiffness will tend to benefit from the surrounding floor slab. Accurate evaluation of the elastic deformations at yield should be undertaken using second-order analyses. However, for design purposes it is argued that the beam, brace and column drift components can be evaluated separately and then added together.

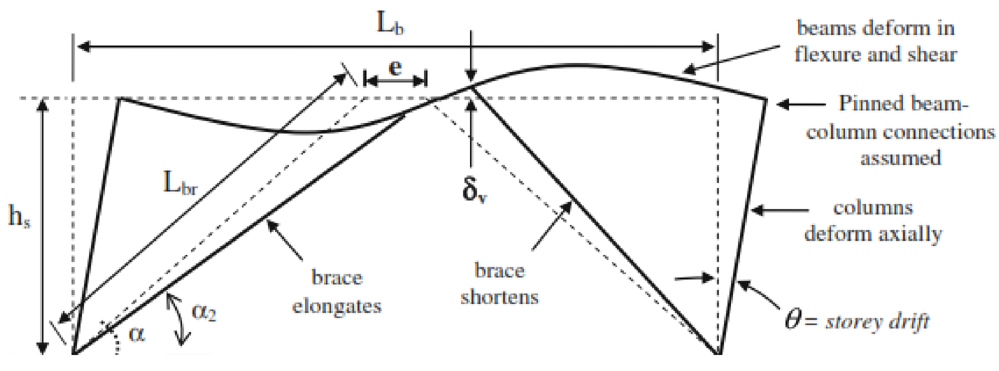


Figure 5. 4. Main elastic deformation components for an EBF **(Sullivan 2013).**

5.3.3 Typical inelastic behaviour of EBF's

Figure 5.5 illustrates the typical plastic mechanisms under lateral force in both directions of EBF single storey structure and having central link.

- **Single storey frames**

Figure 5. 5. Typical inelastic mechanism of one-storey EBF under lateral load

- **Multi-storeys frames**

For the case of multi-storey steel structures with eccentrically braced frame (EBF) structures, such as that sketched in Figure 5.6 were proposed in the late seventies **(Roeder 1977)** as a ductile structural system. Figure 5.6 shows the desirable plastic mechanism of EBF. Yielding of the links occurs along the height of the frame. All the other structural components (beam segments outside of the links, braces, columns, and connections) are proportioned following the capacity design principles to remain essentially elastic during the design earthquake.

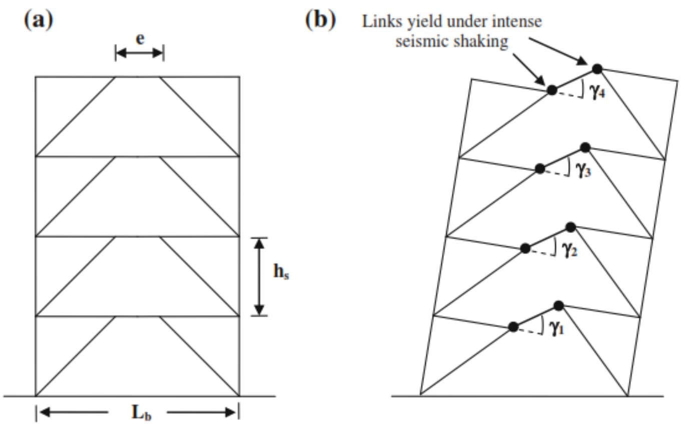
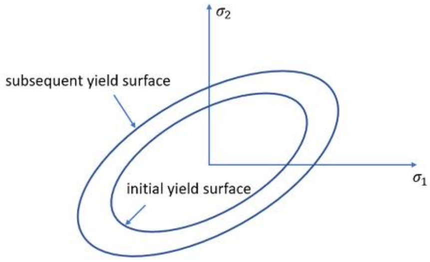


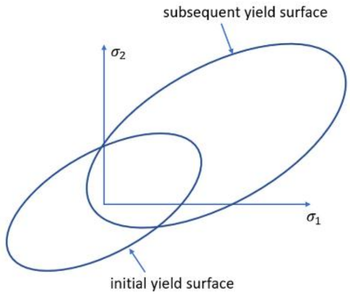
Figure 5. 6. Multi-storeys EBF steel structure
(a) Steel EBF with centrally located links
(b) Expected plastic mechanism under intense seismic shaking **(Roeder 1977).**

5.4 STRAIN HARDENING AND LINKS CLASSIFICATION

Hardening exists when the increase in stress leads to an increase in plastic strain. There are two types of hardening: (1) isotropic hardening of the yield surface (Figure 5.7(a) and kinematic hardening of the yield surface (Figure 5.7(b)).

The isotropic hardening changes the size of the yield surface but keeps the shape of the yield surface. It can model the behaviour of the metals under monotonic loading. On the other hand, kinematic hardening changes both the size and the shape of the yield surface. It is observed in cyclic loading. Sometimes, both hardening rules are used together to present the combined hardening model **(Yang 2020).**





(a) (b)

Figure 5. 7. Types of strain hardening **(Yang 2020)**

(a) Isotropic- hardening. (b) Kinematic hardening.

Within an EBF, the link segment is designed to undergo severe inelastic deformations. During a major seismic event, the link may experience strain on a magnitude that induce strain hardening. Figure 5.8 illustrates an idealised stress-strain curve for structural steel; for strain hardening to occur, the structure must pass through two stages of behaviour.

- During low loading, a structure should remain in region “a” the elastic range; in the elastic range an increase in stress results in a linear increase in strain related to the modulus of elasticity, E of the structural material.
- During moderate loading, a structure may enter region “b” with the transition between “a” and “b” characterized by inelastic (non-linear) behaviour. In region “b,” the strain increases at a constant stress level, or behaves plastically.

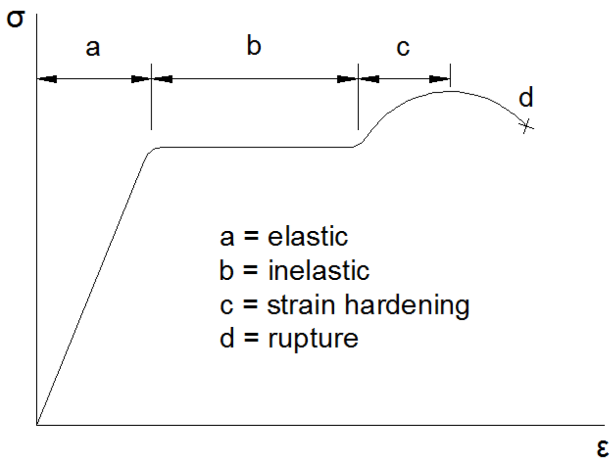


Figure 5. 8. Idealised structural steel stress-strain curve.

- During plastic deformation, permanent residual deformations occur though the deformations may not be detrimental to the structural capacity upon unloading. After the structure's plastic capacity is reached, additional inelastic behaviour occurs as strain hardening. During strain hardening, the structure can undergo further deformation with a non-linear increase in stress. After the maximum tensile load is reached, necking occurs in members as strain continues to increase. During necking, the cross-sectional area of the seismic fuse in the LRFD (load and resistance factor design) which is a method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations. System decreases reducing the stress; as strain continues to increase the member ruptures, indicated by point “d” in Figure 5.8.

Shear or short links: Strain hardening in the link element requires the reduction of the shear link length ratio limit from 2. Furthermore, as a link element experiences large rotation angles, large end moments and steep strain gradients develop causing large flange strain. Large flange strain leads to instability in the form of web buckling after yielding; for unstiffened webs, web buckling occurs very shortly after shear yielding. Web buckling of shear links causes a severe reduction in load-carrying capacity, reducing energy dissipation and ductility **(Kasai 1986a)**. This normalized link length, ρ, a non-dimensional link length or length ratio:

$$\rho = \frac{e}{\frac{M_P}{V_P}} \tag{5.1}$$

Intermediate Links: The lower bound for intermediate link elements is a length ratio of 1.6. As approaches the theoretical boundary, link failure involves shear and flexural yielding. Assuming the link moment is equally distributed between the link ends, link behaviour will occur in a progression similar to the following scenario:

1. Flexural yielding of the link flanges at both ends
2. Flexural yielding of the top flange of the brace panel
3. Shear yielding of the link web
4. Local buckling of the link flanges

After local buckling of the link flanges, which can be severe in appearance but does not mean a strength reduction, link behaviour depends on the slenderness of the flanges as link segments are the seismic fuse of EBFs, they must display highly ductile behaviour; as such, slenderness limits must preclude local failures that cause rapid strength degradation. Research has shown that for links with slender flanges, severe flange buckling of the top flange of the brace panel directly outside the link succeeds shear yielding of the web and causes rapid degradation of load-carrying capacity upon continued cyclic loading **(Engelhardt 1992)**.

Flexure or long Links: Links with are designated flexural links by AISC 341, though, as discussed in the previous section, combined behaviour may still occur in links with -values near the lower boundary. As increases above 3.0, flexural yielding dominates inelastic behaviour. The progression of yielding and instability is similar to that of intermediate links without web yielding and instability only occurring near the ends of a link. Yielding and instability for flexural links occurs in the following order, assuming equally distributed link end-moments:

1. Flexural yielding of link flanges at both ends;
2. Flexural yielding of the top flange of the brace panel;
3. Flexural yielding of previously yielded flanges increases in severity.

Following the increased flexural yielding, link behaviour depends on the slenderness of the flanges, as with intermediate links. For slender flanges, the first form of link stability is flange buckling at the link ends; the flange buckling is typically not detrimental to link capacity.

5.5 INELASTIC RESPONSE AND ENERGY DISSIPATION

As structures enter the inelastic range, sizable and permanent deformation occurs that causes damage to the structure. Eccentrically Braced Frame (EBF) structural system is a system that limits the inelastic behaviour to only the link beam that lies between two eccentric braces, while the outer beam, column and diagonal braces remain elastic during the seismic loading. As mentioned above the inelastic action during an earthquake is intended to occur within the link of an EBF. Figure 5.9 shows the inelastic mechanism for an EBF. Consequently, the link can experience very large inelastic rotations. As will be discussed later, a well-designed and detailed link should be able to sustain a cyclic inelastic rotation up to ±0.08 rad.

Working principle of eccentrically braced frames relies on the transfer of the moment and shear forces on a segment of the beam through the brace to column or another brace as axial force. This beam segment is called as active link or shear link. The link member yields after severe cyclic movement and dissipate large amount of energy **(Hjelmstad 1984)**. Comparing Figures 5.9 and 5.10, the energy dissipation mechanism of EBFs and difference between CBFs and EBFs can be observed more clearly as a result of the differences in the yielding mechanism.

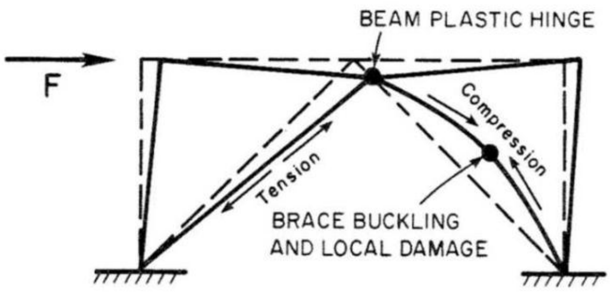


Figure 5. 9. Inelastic behaviour of concentrically braced frames during a major earthquake **(Kasai 1986a)**.

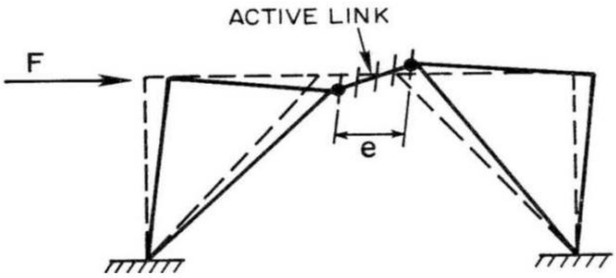


Figure 5. 10. Plastic deformation of K-braced frames **(Kasai 1986a)**.

5.6 EBF BEHAVIOUR AND CLASSIFICATION

An eccentrically braced frame is a framing system in which the axial forces induced in the braces are transferred either to a column or another brace through shear and bending in a small segment of the beam.

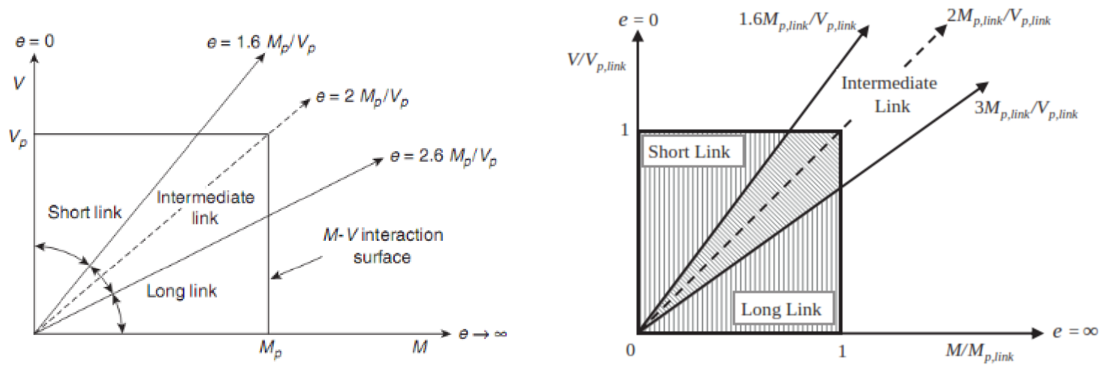
Typical EBF geometries are shown in Figures 5.1 and 5.2. Architecturally, EBF also provides more freedom for door opening than CBF. The critical beam segment is called a “link” and is designated by a length, e, in the figure. Links in EBFs act as structural fuses to dissipate the earthquake-induced energy in a building in a stable manner.

The geometry of frames with eccentric bracings is close to that of frames with concentric bracings; some intentional eccentricities in the layout of bars generate bending moments and shear. These structures resist horizontal forces essentially by axially loaded members, but they are designed to yield first in shear or bending in ‘seismic links. The intended behaviour of an EBF subject to earthquake loading is that yielding occurs within the ductile link while the other frame elements remain elastic.

To achieve this behaviour, the links must be the weakest elements in the frame and the braces, columns and the beam segment outside the links should therefore be necessarily stronger than the links. It can be said that links are the fuse elements of an EBF **(Popov 1988)**. The link of an EBF experiences three forces: shear, axial, and flexural. There are three possible link beam criteria in the EBF structural system that are; short links, intermediate links and long links. This criterion is determined from the normalization of link length with the ratio between plastic moment capacity M_p and plastic shear capacity V_p. The classification of these links is shown in Figure 5.11 **(Bruneau 2011)** that are link with length ratios less than 1.6 is categorized are short links or shear links due to the more dominance of shear yielding. Links with a length ratio of more than 2.6 are categorized as long links or moment links due to the more dominance of bend yielding. While links with long ratios ranging from 1.6 to 2.6 are categorized as intermediate links or moment-shear links because the yielding occurred is a combination of shear and bending **(Richards 2005)**.

There are substantial differences between the behaviour of short and long links. Although longer links provide more architectural freedom for openings, early experimental studies by Roeder **(Roeder 1977, Richards 2005; Hjelmstad 1983a, Berman 2010)** showed that the performance of short links is considerably better than that of long links under severe cyclic loadings in terms of strength and ductility.

Based on equations the following equations, Figure 5.11 (a) and (b) explains the classification criteria for seismic links, relating the link length to the bending-shear interaction domain. Short links develop only shear yielding, long links develop only flexural yielding, while intermediate links develop both shear and flexural yielding.



(a) (b)

Figure 5. 11. Link Classification (a) **Bruneau 2011 (b) Landolfo 2017**.

Short links: $e \le e_s = 1.6\frac{M_{p,link}}{V_{p,link}}$

Long links: $e \ge e_L = 3\frac{M_{p,link}}{V_{p,link}}$

Intermediate links: $e_s < e < e_L$

5.7 CHARACTERISTICS OF A SEISMIC LINK

5.7.1 Introduction

The link is the key distinguishing feature of an EBF, which is a portion of the beam set out between the braces or between one brace and the column. According to Eurocode 8 clause 6.8.1(1)P, and others seismic codes, the links are the components devoted to dissipate seismic energy.

On the basis of the type of plastic mechanism to be developed, links are classified into three categories: short (which dissipate energy by yielding essentially in shear), intermediate (in which the plastic mechanism involves bending and shear) and long (which dissipate energy by yielding essentially in bending), depending on the structural and geometric properties of the links as in EC8 (clause 6.8.2(2)).

The mechanical parameter influencing the plastic mechanism is the link length “e”, which is related to the ratio between the plastic bending moment and the plastic shear of the cross section of the link.

The analysis of frames with eccentric bracings does not require all the approximations made in the case of concentric bracings, because such frames are not designed so that diagonals will buckle under seismic conditions. Diagonals are part of the non-dissipative zones; they are capacity designed to the strength of the links, in order to remain elastic and to avoid buckling **(Faradis 2005)**.

5.7.2 Elastic stiffness

The variations of the lateral stiffness of a simple EBF with respect to the link length is shown in Figure 5.12 **(Hjelmstad 1984).**

Note that *(e/L)* ratios of 0.0 and 1.0 correspond to a concentrically braced frame and a moment frame, respectively. The figure clearly shows the advantage of using a short link for drift control.

A careful design of seismic links can lead good hysteresis loops with large stiffness and energy absorption. EBF structures exhibit good strength and stiffness in elastic range, so avoiding non-structural damage, and are also able to provide enough ductility to dissipate large amounts of energy in the inelastic range.

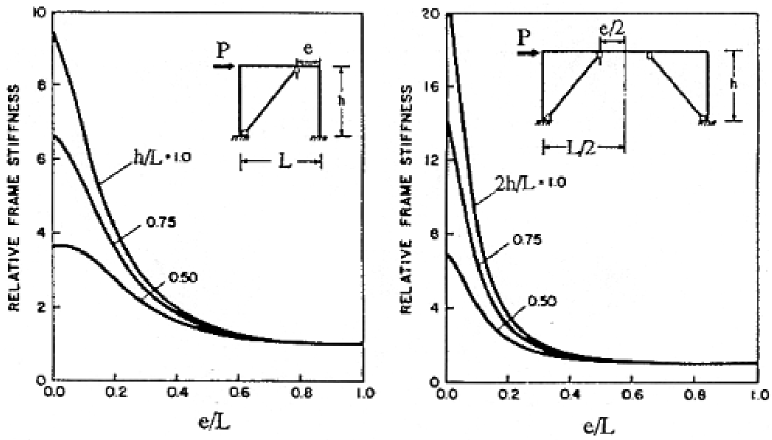


Figure 5. 12. Variations of lateral stiffness with respect to link length **(Hjelmstad 1984).**

5.7.3 Link plastic rotation angle

Experimental studies have shown that the rotation capacity of links depends significantly upon several factors such as the link length ratio, loading history, compactness and web stiffening. It is interesting to note that the type of link plastic mechanism is directly related to the available ductility, namely plastic rotation of a link is often described by its plastic rotation capacity denoted as γ_p (AISC 2010) and θ_p (EC8), provided by the link in order to withstand the seismic ductility demand. The link rotation is defined as the rotation angle, between the link and the element outside of the link.

A goal of EBF design is that the link plastic rotation capacity exceeds the plastic rotation demand of an earthquake. In EBF design, the link plastic rotation can be related to the plastic storey drift angle, θ_p, by the geometry of a rigid plastic mechanism as shown in Figures below. Figure 5.13 clarifies this definition and simple equations are given for estimating the link rotation, which should be consistent with global deformation, for the EBF configurations reported in Figure 5.13 (i.e. split-K-braced frame, V-braced, D-braced frame and inverted-Y-braced frame). It can be observed that the shorter is the link length and the greater is the ductility demand.

According to American code, AISC 2006, Figures 14(a), (b) and (c) show the rigid plastic mechanism for three common EBF geometries with the total beam span denoted *L* and the link length denoted as *e*. Equations showed in association with each figure present the relationship between link plastic rotation angle and the plastic storey drift angle for mechanisms. It's worth to mention that the ratio of span length to link length (L/e) increases, the link rotation angle also increases for a given plastic storey drift angle. Subsequently, large values of L/e can result in excessive plastic rotation demands on the link. However, the configuration shown in Figure 14(c), with two links in each level, places only one-half the plastic rotation demand on the link as compared to the other configurations.

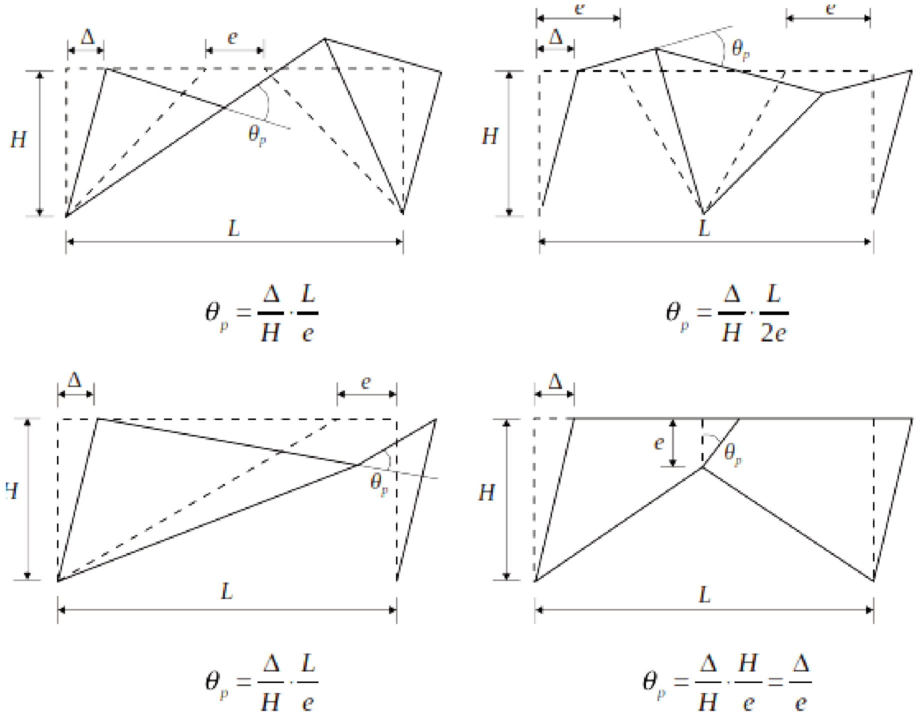


Figure 5. 13. Link rotation angle θ_p for the EBF configurations **(AISC 2006).**

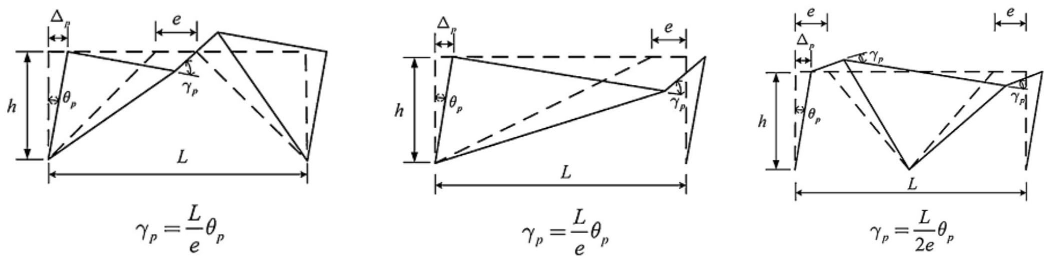


Figure 5. 14. Link plastic rotation of EBF
(a) one link at the middle of the beam (b) one link next to the column (c) with two links next to the columns **(Bouwkamp 2016).**

The allowable link deformation capacity depends on the link length, as depicted in Figure 3.15. With this regard, clause 6.8.2(10) of EC8 2004, states that the link rotation should not exceed the following values: θ_p

Short links: $\theta_p \leq \theta_{pR} = 0.08$ rad;

Long links: $\theta_p \leq \theta_{pR} = 0.02$ rad;

Intermediate links: $\theta_p \leq \theta_{pR}$ = the value determined by linear interpolation between the above values.

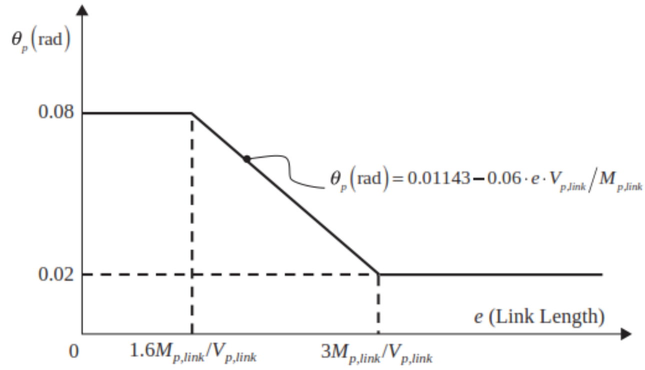


Figure 5. 15 Capacity rotations of seismic links **(Landolfo 2017).**

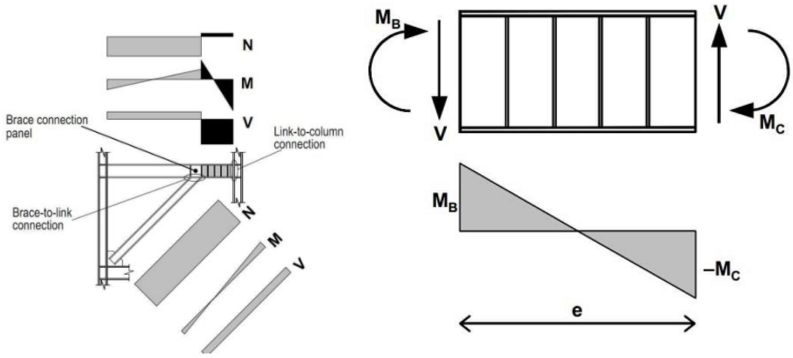
5.7.4 Equilibrium and acting forces in links and beams

An eccentrically braced frame is a framing system in which the axial force induced in the braces are transferred either to a column or another brace through shear and bending in a small segment of the beam. The critical beam segment is called a "link" and is designated by its length "e". Links in EBFs act as structural fuses to dissipate the earthquake induced energy in a building in a stable manner.

During the design process, it is necessary to estimate link end moments in order to determine the internal force distribution after formation of the expected plastic mechanism. For links located in the middle portion of floor beams (internal links) the end moments will almost be equal throughout a seismic loading. On the other hand, for links connected to columns (external links) the end moments will not be identical in the elastic range. However, early studies of **(Kasai 1986a, 1986b)** proved that for most cases these moments would equalize as the link goes through large plastic rotations. Thus, in both cases, the link end moments can be readily estimated using equilibrium.

Once again, the length of a link segment (e) is one of the key parameters that controls the stiffness, strength, ductility, and behaviour of an EBF system. The link length ratio noted $\rho = e/(M_P/V_P)$, where M the plastic moment and plastic shear capacities of the link, provides a convenient measure for the yield behaviour. The free-body diagram of an isolated link is shown in Figure 5.16.

The force distribution in an eccentrically braced frame under lateral forces is illustrated in Figure 5.16(a). Generally, constant shear, reverse curvature moment and small axial force are observed along a shear-link, and axial force is dominant in a brace member of EBF system. While Fig. 5.16(b) shows the end moments (M_B and M_C) and the shear force (V) that may develop on link element, and since the axial force is small it may be safely assumed negligible and zero.



(a) (b)

Figure 5. 16. Typical force distribution free body diagram of link in an EBF **(Okazaki 2004)**.

The free-body diagram of an isolated link is shown in Figures 5.17. Based on equilibrium, considering equal end moments at the ultimate state, no moment-shear interaction, and an elastic-perfectly plastic material, the theoretical dividing link length ratio between shear dominated and flexure dominated behaviour is ρ_{theor} = 2.0.

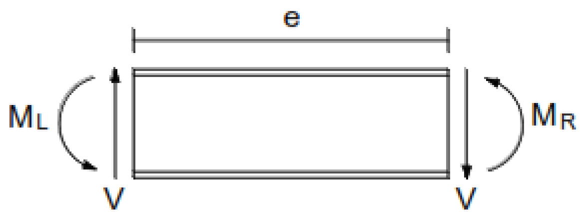


Figure 5. 17. Free-body diagram of an isolated link segment **(Bruneau 2011).**

In short (or shear) links, shear yielding of the web is found to be predominant (Figure 5.18(a)). On the other hand, in long (or moment) links, flexural yielding controls the link behaviour (Figure 5.18(c)). An intermediate link, however, would experience a combination of both shear and flexural yielding (Figure 5.18(b)).

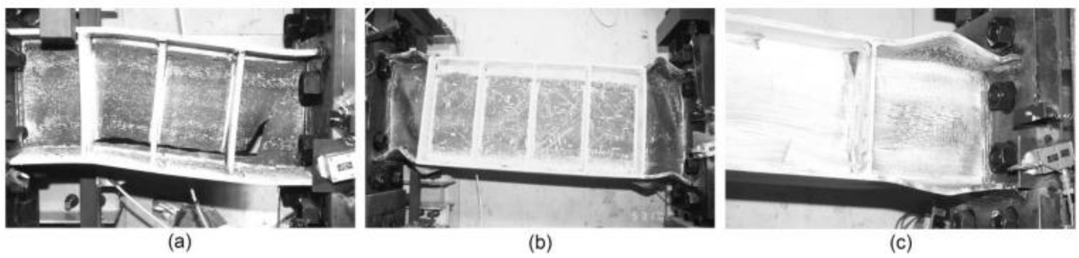


Figure 5. 18. Failure mechanisms of (a) short, (b) intermediate, and (c) long links **(Okazaki 2005).**

In addition to what has been said, Figure 5.19 shows more qualitatively the distribution of moment, shear and axial force in the link and beam segments outside of the link in an EBF subjected to lateral load. Two common EBF configurations are shown; one with the link at mid-span and the other with the link connected to the column. The link is generally subject to high shear along its full length, high end moments and low axial force. Yielding within the link can be shear yielding, flexural yielding or a combination of shear and flexural yielding. Yielding of links and the close relationship to link length will be discussed in greater details below.

As shown in the figure, the beam segment has a high bending moment immediately adjacent to the link. This is because the high moment at the end of the link must be resisted primarily by the beam segment. The figure shows a drop-in moment between the end of the link and the adjoining beam segment. This drop-in moment represents the portion of the link end moment transferred to the brace, assuming the connection between the brace and the link can transfer moment. In addition to high moment, the beam segment outside of the link is also typically subjected to high axial force. The bracing member in an EBF also undergoes high axial force, and the horizontal component of the brace axial force will generate high axial force in the beam segment.

Finally, as shown in Figure 5.19, the shear in the beam segment outside of the link is generally small. Consequently, the force environment for the beam segment outside of the link is dominated by high axial force and high moment. Since earthquake loads are cyclic, the beam segment outside of the link experiences both axial tension and axial compression. Designing the beam segment for these high moments and axial forces can be difficult, which is described in greater detail later.

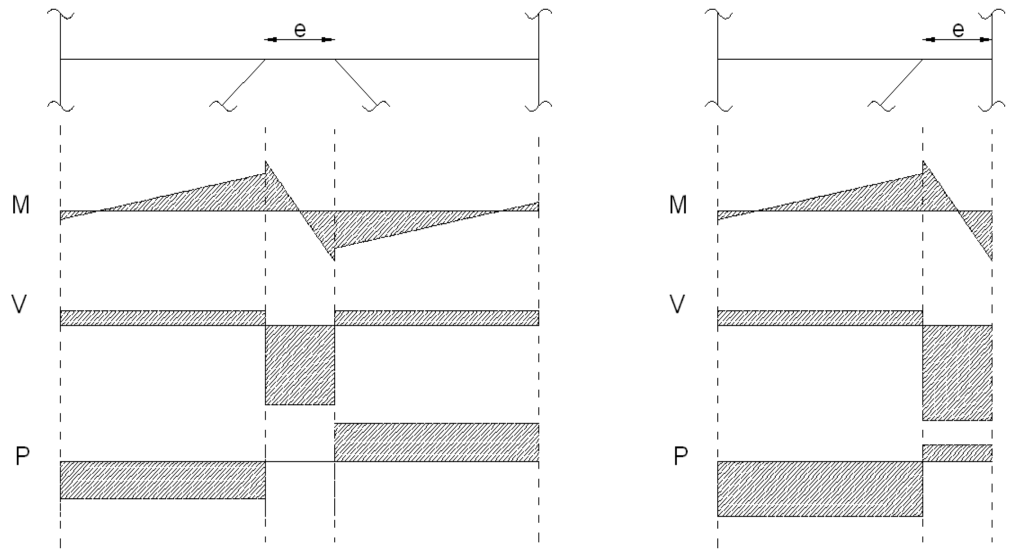


Figure 5. 19. Distribution of forces in the link and the beam outside the link.

Overall behaviour of an eccentrically braced frame subjected to lateral forces, namely seismic loadings, is illustrated in Figure 5.20 by **Okazaki (2004).**

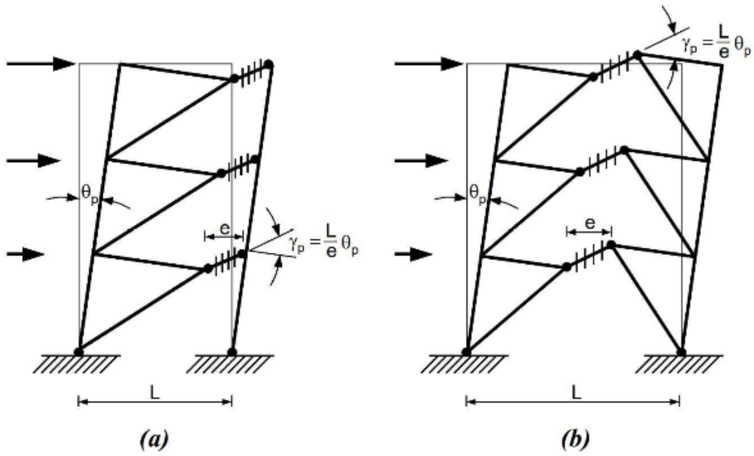


Figure 5. 20. Energy dissipation mechanisms **(Okazaki 2004)**.

5.7.5 Influence of web stiffeners

A common feature of EBFs are the stiffeners located within the link which guarantee a sufficient link rotation capacity. Once shear buckling occurs in a stiffened link, tearing along the perimeter of the link panes due to stress concentration created by the buckled web may cause significant strength degradation.

Stiffeners are fundamental details which can enhance the ductility of the links by delaying shear buckling of the link webs and allow to prevent inelastic web buckling, which impairs the link performance in the range of the expected ductility demand.

The stiffeners may be required to transfer link shear forces to the connected members. Also, as the shearing stress in the links is high, and under cyclic loading web, stiffeners must be provided to prevent a buckling of web and the flanges which may induce severe premature torsional buckling of the web. As per AISC 341 and EC8, each side of each end shall have a stiffener.

Assuming that the link web buckling modes are similar to those of a plate under shear loading, they applied plastic plate shear buckling theory to relate the stiffeners spacing to the maximum deformation angle of a shear link, thus deriving simple expressions for each required link deformation capacity. Indeed, full depth web stiffeners are required on both sides of the link web at the brace interfaces on all links.

As stipulated in many seismic codes, web stiffeners must be fillet welded to the link web and flanges and be detailed to avoid welding in the k-region of the link, as reduction in the plastic rotation capacity of the link can occur when welds extend into the k-region **(Okazaki 2004)**.

Several early experimental research results Popov and his colleagues on the effect of placing intermediate stiffeners were published. In fact, **(Malley 1983; Kasai 1986a; Popov 1988)** demonstrated that providing intermediate stiffeners could substantially improve the strength and energy dissipation capacity of links. **Hjelmstad (1983a)** proposed the first relation for determining the required intermediate stiffener spacing based on the expected energy dissipation of a link.

The proper use of end and intermediate web stiffeners in links is a major parameter for achieving stable and controlled hysteresis behaviour. End stiffeners are usually full-depth stiffeners provided for all link length ratios located on both sides of the web at link ends. In 1977, Roeder and Popov provided the rationale for the necessity of using end stiffeners to ensure local stability at a brace-link-beam connection panel. In the previous and current EBF specifications AISC and EC8 the use of end stiffeners has always been mandatory with an aim of improving the link shear force transfer to reacting elements as well as preventing premature local buckling in links.

5.8 DESIGN OF LINKS

In the design process, the active link, also called seismic links, must be designed in order to reach its bending and shear limit capacity before the achievement of the tension and compression limit strength of other members. The length of the seismic link with the manner of dissipating energy is responsible of the collapse mechanism. Thereby, a careful design of seismic links which may be horizontal or vertical components, can lead good hysteresis loops with large stiffness and energy absorption.

EBF designed in accordance with the following provisions EC8 and AISC 2005 are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links. Seismic links may be horizontal or vertical components. The link beam should be designed so that it is the weak part (dissipative zone) of the structure under severe seismic loading which is achieved by selecting the size of the steel section and the length of the link beam to match seismic load design requirements. Yielding or buckling of the columns must also be avoided. The brace and column design forces are needed to ensure that the brace and column do not buckle as the link beam undergoes strain hardening during inelastic deformation.

5.8.1 EC8 Provisions

5.8.1.1. General considerations design criteria

Eurocode 8 gives simple rules for the designing of EBFs where the seismic energy dissipation is taken by vertical or horizontal seismic links. According to the behaviour of link due to their dimensions and internal forces, three different types of links are defined by the code, namely, the short link (dissipation is guaranteed by yielding in shear), the long link (link dissipate energy by yielding in flexure) and the intermediate link (where plastic mechanisms is due to bending and shear).

Frames with eccentric bracings shall be designed so that specific elements or part of elements called seismic links, are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms. The structural system shall be designed so that a homogeneous dissipative behaviour of the whole set of seismic links is realised. Frames with eccentric bracings shall be designed so that specific elements, which is the seismic links are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms.

- The structural system shall be designed so that a homogeneous dissipative behaviour of the whole set of seismic links can be achieved.
- The rules given henceforward are intended to ensure that yielding, including strain hardening effects in the plastic hinges or shear panels, will take place in the links prior to any yielding or failure elsewhere in other parts of connecting members.

5.8.1.2 Seismic links

It is worth to note that the rules given hereafter are intended to ensure that yielding, including strain hardening effects in the plastic hinges or shear panels, will take place in the links prior to any yielding or failure elsewhere, and Seismic links may be horizontal or vertical component.

The web of a link shall be single thickness without doubler plate reinforcement and without hole or penetration. Seismic links are classified into 3 categories according to the type of plastic mechanism developed:

– Short links, which dissipate energy by yielding essentially in shear;
– Long links, which dissipate energy by yielding essentially in bending;
– Intermediate links, in which the plastic mechanism involves bending and shear.

5.8.1.3 Code requirements for seismic links

The link is the key distinguishing feature of an EBF, which is a portion of the beam set out between the braces or between one brace and the column. According to clause 6.8.1(1)P of EC8 2004, the links are the components devoted to dissipate seismic energy.

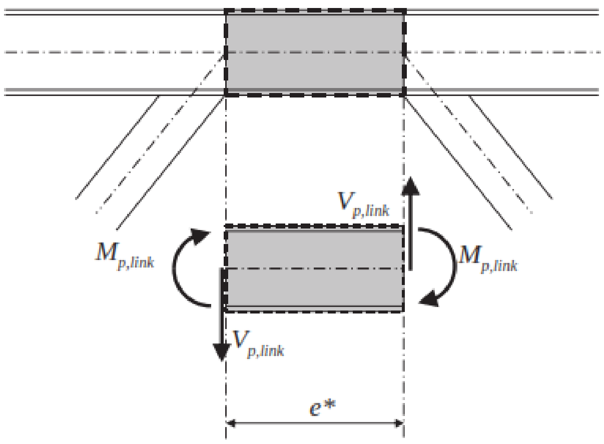


Figure 5. 21. Theoretical limit for link length (perfectly plastic behaviour with no bending shear interaction **(Landolfo 2017)**.

As previously mentioned, the mechanical parameter influencing the plastic mechanism is the link length "e", which is related to the ratio between the plastic bending moment and the plastic shear of the cross section of the link Figure 5.21. In order to clarify this concept, it is useful to refer to a link subjected to symmetric flexural actions equal to the plastic bending moment M_{plink} and a shear force equal to the plastic shear strength V_{yplink}, calculated according to clause 6.8.2(3) as follows:

$$M_{p,link} = f_y\, b\, t_f\, (h - t_f) \text{ And } V_{p,link} = (f_y / 3)\, t_w\, (h - t_f) \qquad (5.2)$$

where f_y is the value of the yield stress of steel, d is the depth of the cross section, t_f is the flange thickness and t_w is the web thickness.

For I sections, the categories are as shown in equations (5.3) below:

- Short links $e < e_s = 1,6\, M_{p,link} / V_{p.link}$ (5.3)
- Long links $e < e_L = 3,0\, M_{p,link} / V_{p.link}$
- Intermediate links $e_s < e < e_L$

For I sections, Figure 5.22, the following parameters are used to define the design resistances and limits of categories:

$$M_{p,\,link} = f_y\, b\, t_f\, (d - t_f) \qquad (5.4)$$

$$V_{p,link} = (f_y / \sqrt{3})\, t_w\, (d - t_f) \qquad (5.5)$$

where the intern forces are those showed in Figure 5.21.

If the ratio $(N_{Ed}/N_{pl,Rd}) \leq 0.15$, the design resistance of the link should satisfy both of the following relationships at both ends of the link:

$$V_{Ed} \leq V_{p,link} \qquad (5.6)$$

$$M_{Ed} \leq M_{p,link} \qquad (5.7)$$

where N_{Ed}, M_{Ed}, V_{Ed} design action effects, respectively design axial force, design bending moment and design shear, at both ends of the link.

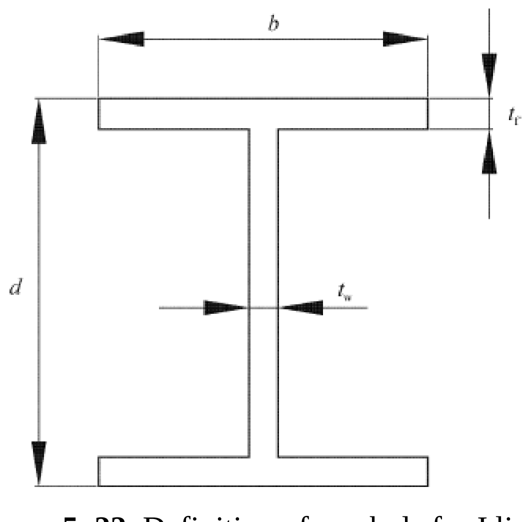


Figure 5. 22. Definition of symbols for I link sections.

If $N_{Ed}/N_{Rd} \geq 0.15$, the link length e should not exceed:

$e \leq 1{,}6\ M_{p,link}/V_{p,link}$ (5.8)

When $R = (N_{Ed}.\ t_w.\ (d - 2t_f) / V_{Ed.A}) < 0.3$, in which A is the gross area of the link

Or $e \leq (1.15 - 0.5\ R)\ 1.6\ M_{p,link}/V_{p,link}$ (5.9)

When $R \geq 0.3$.

To achieve a global dissipative behaviour of the structure, it should be checked that the individual values of the ratios i $\Omega = 1.5\ V_{pl,Rd,I} / V_{Ed,1}$ for short link and i $\Omega = 1.5\ M_{pl,Rd,i}/M_{Ed,1}$ for long links, do not exceed the minimum value Ω by more than 25%.

In design where equal moments would form simultaneously at both ends of the link, links may be classified according to the length e. For I sections, the categories are:

− Short links $e < es = 1.6\ Mp,_{link}/Vp,_{link}$ (5.10)

− Long links $e > e_L = 3.0\ Mp,_{link}/Vp,_{link}$ (5.11)

− Intermediate links $es < e < e_L$ (5.12)

In design where only one plastic hinge would form at one end of the link, the length e defining the categories of the links is, for I sections:

− Short links $e < e_s = 0.8\ (1+\alpha)\ M_{p,link}/V_{p,link}$ (5.13)

− Long links $e > e_L = 1.5\ (1+\alpha)\ M_{p,link}/V_{p,link}$ (5.14)

− Intermediate links $e_s < e < e_L$ (5.15)

where α is the ratio of the smaller bending moments $M_{Ed,A}$ at one end of the link in the seismic design situation, to the greater bending moments $M_{Ed,B}$ at the end where the plastic hinge would form, both moments being considered in absolute value, Figure 5.23.

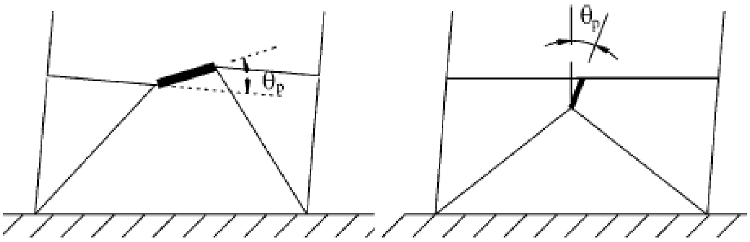


Figure 5. 23. a) equal moments at link ends; b) unequal moments at link ends **(EC8 2004.)**

The link rotation angle θp between the link and the element outside of the link should be consistent with global deformations. It should not exceed the following values:

− Short links $\theta_p \leq \theta_{pR} = 0.08$ rad (5.16)

− Long links $\theta_p \leq \theta_{pR} = 0.02$ rad (5.17)

− Intermediate links $\theta_p \leq \theta_{pR}$ = the value determined by linear interpolation between the above values.

- Provisions for stiffeners

To EC8, full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners should have a combined width not less than $(bf - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm, whichever is larger.

Links should be provided with intermediate web stiffeners as follows:

a) Short links should be provided with intermediate web stiffeners spaced at intervals not exceeding ($30t_w$ – $d/5$) for a link rotation angle of 0.08 radians or ($52t_w$ – $d/5$) for link rotation angles of 0.02 radians or less. Linear interpolation should be used for values between 0.08 and 0.02 radians;

b) Long links should be provided with one intermediate web stiffener placed at a distance of 1.5 times b from each end of the link where a plastic hinge would form;

c) Intermediate links, should be provided with intermediate web stiffeners meeting the requirements of a) and b) above.

d) Intermediate web stiffeners are not required in links of lengths greater than 5 M_p/V_p;

e) Intermediate web stiffeners should be full depth. For links that are less than 600 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm, whichever is larger, and the width should be not less than $(b/2) - t_w$. For links that are 600 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

- **Further provisions**

Fillet welds connecting a link stiffener to the link web should have a design strength adequate to resist a force of $\gamma_{ov} f_y A_{st}$, where A_{st} is the area of the stiffener. The design strength of fillet welds fastening the stiffener to the flanges should be adequate to resist a force of $\gamma_{ov} A_{st} f/4$.

Lateral supports should be provided at both the top and bottom link flanges at y the ends of the link. End lateral supports of links should have a design axial resistance sufficient to provide lateral support for forces of 6% of the expected nominal axial strength of the link flange computed as fy $b\ t$.

In beams where a seismic link is present, the shear buckling resistance of the f web panels outside of the link should be checked to conform to EN 1993-1-5:2004, Section 5.

- **Members not containing seismic links**

The members not containing seismic links, like the columns and diagonal members, if horizontal links in beams are used, and also the beam members, if vertical links are used, should be verified in compression considering the most unfavourable combination of the axial force and bending moments:

$$N_{Rd}\,(M_{Ed},V_{Ed}) \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{5.8}$$

N_{Rd} (M_{Ed}; V_{Ed}) is the axial design resistance of the column or diagonal member in accordance with EN 1993, taking into account the interaction with M_{Ed} the bending moment and the shear V_{Ed} taken at their design value in the seismic situation; is the compression force in the column or diagonal member due to the non-seismic actions included in the combination of actions for the seismic design situation; is the compression force in the column or diagonal member due to the design seismic action; γ_{ov} is the overstrength factor Ω is a multiplicative factor which is the minimum of the following values:

- the minimum value of $\Omega_I = 1.5\, V_{p,link,i} / V_{Ed,I}$ among all short links;
- the minimum value of $\Omega_i = 1.5\, M_{p,link,i}/M_{Ed,I}$ among all intermediate and long links;

where, $V_{Ed,i}$, $M_{Ed,I}$ are the design values of the shear force and of the bending moment in link *i* in the seismic design situation; $V_{p,link}$; $M_{p,link}$ are the shear and bending plastic design resistances of link *i*.

5.8.2 AISC provisions

The AISC Specification for Structural Steel Buildings (ANSI/AISC 360-05) is intended to cover common design criteria. The AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-05) with supplement No. 1 (ANSI/AISC 341s1-05) (hereafter referred to as the provisions) is a separate consensus standard that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems for high seismic applications.

Supplement No. 1 consists of modifications made to part I, section 14 of the provisions after the initial approval had been completed. These provisions are presented in two parts: Part I is intended for the design and construction of structural steel buildings, and is written in a unified format that addresses both LRFD and ASD; Part II is intended for the design and construction of composite structural steel/ reinforced concrete buildings, and is written to address LRFD only.

Eccentrically braced frames (EBF) of structural steel shall be designed in conformance with this section F3.

5.8.2.1 Basis of Design

This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centrelines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure. EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links.

5.8.2.2 Analysis

The required strength of diagonal braces and their connections beams outside links, and columns shall be determined using the capacity-limited seismic load effect.

5.8.2.3 System Requirements

- Link rotation angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total storey drift is equal to the design storey drift, Δ. The link rotation angle shall not exceed the following value:

- For links of length $1.6M_p/ V_p$ or less: 0.08 rad
- For links of length $4.6M_p/V_p$ or greater: 0.02 rad

where, M_p = plastic bending moment of a link, kip-in. (N.mm)

V_p = plastic shear strength of a link, kips (N)

Linear interpolation between the above values shall be used for links of length between 1.6Mp/Vp and 4.6Mp/Vp.

- Bracing of Link

Bracing shall be provided at both the top and bottom link flanges at the ends of the link for I-shaped sections. Bracing shall have an available strength and stiffness as required for expected plastic hinge locations by section D1.2c.

5.8.2.4 Members

- - Basic Requirements (AISC 2005)

Brace members shall satisfy width-to-thickness limitations in section D1.1 for moderately ductile members. Column members shall satisfy width-to-thickness limitations in section D1.1 for highly ductile members.

Where the beam outside of the link is a different section from the link, the beam shall satisfy the width-to-thickness limitations in section D1.1 for moderately ductile members.

- Links

Links subject to shear and flexure due to eccentricity between the intersections of brace centrelines and the beam centreline (or between the intersection of the brace and beam centrelines and the column centreline for links attached to columns) shall be provided. The link shall be considered to extend from brace connection to brace connection for centre links and from brace connection to column face for link-to column connections, except as permitted by section F3.6e.

- Limitations

Links shall be I-shaped cross sections (rolled wide-flange sections or built-up sections), or built-up box sections. HSS sections shall not be used as links.

Links shall satisfy the requirements of section D1.1 for highly ductile members**.**

Exceptions: Flanges of links with I-shaped sections with link lengths,

$e \le 1.6\ M_p/V_p$ are permitted to satisfy the requirements for moderately ductile members. Webs of links with box sections with link lengths, $e \le 1.6M_p/V_p$, are permitted to satisfy the requirements for moderately ductile members.

The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted. For links made of built-up cross sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges.

Links of built-up box sections shall have a moment of inertia, I, about an axis in the plane of the EBF limited to $I_y > 0.67I_x$, where I_{xy} is the moment of inertia about an axis perpendicular to the plane of the EBF.

- Shear strength

The link design shear strength, $\varphi_v Vn$, and the allowable shear strength, $Vn/\Omega v$, shall be the lower value obtained in accordance with the limit states of shear yielding in the web and flexural yielding in the gross section. For both limit states:

$\Phi_v = 0.90$(LRFD) and $\Omega_v = 1.67$

(a) For shear yielding

$V_n = V_p$

$V_p = 0.6F_vA_{lw}$ for $\alpha_s P_r/P_v \le 0.15$

$A_{tw} = (d\text{-}2t_f)\ t_w$ for I- shaped link sections

$A_{tw} = 2(d\text{-}\ 2t_{f)}\ t_w$ for box link sections

$P_r = P_u$ (LRFD) or P_a (ASD), as applicable

P_u = required axial strength using LRFD load combinations, kips (N)

P_a = required axial strength using ASD load combinations, kips (N)

P_y = nominal axial yield strength = F_yA_g, kips (N)

d = overall depth of link, in. (mm)

t_f = thickness of flange, in. (mm)

t_w = thickness of web, in. (mm)

- For flexural yielding:

$V_n = 2\ M_{p/}\ e$

where :

$M_p = F_y Z$ for $\alpha_s P_r/P_y \le 0.15$

$M_p = F_y Z\left(\frac{1-\alpha_s P_r/P_y}{0.85}\right)$ for $\alpha_s P_r/P_y > 0.15$

Z = plastic section modulus about the axis of bending, in^3 (mm^3).

e = length of link, defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face, in. (mm).

- Link Length

If $\alpha_s P_r/P_y > 0.15$), the length of the link shall be limited as follows:

When $\rho' \le 0.5$: $e \le 1.6\ M_p/V_p$

When $\rho' > 0.5$:

$e \leq \frac{1.6M_p}{V_p}(1.15 - 0.3\rho')$

$\rho' = \frac{P_{r/P_y}}{V_{r/V_y}}$

V_r = Vu (LRFD) or Va (ASD), as appropriate

V_u = required shear strength based on LRFD load combinations, kips

V_a = required shear strength based on ASD load combinations, kips

User Note: For links with low axial force there is no upper limit on link length. The limitations on link rotation angle in section F3.4a result in a practical lower limit on link length.

- Link Stiffeners for I-Shaped Cross Sections

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than (b_f - $2t_w$) and a thickness not less than $0.75t_w$ or (10 mm), whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively links shall be provided with intermediate web stiffeners as follows:

(a) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding ($30t_w$ – d/5) for a link rotation angle of 0.08 radian or ($52t_w$–d/5) for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.

(b) Links of length greater than $4.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.

(c) Links of length between $1.6M_p/V_p$ and $4.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.

d) Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.

(e) Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (635 mm) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or (10 mm), whichever is larger, and the width shall be not less than (bf /2) - t_w. For links that are 25 in (635 mm) in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}Fy$ (LRFD) or $A_{st}Fy/1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}Fy/4$ (LRFD) or $A_{st}Fy/4\alpha_s$ (ASD).

5.9 DETAILING OF LINKS TO EC8

To serve its intended purpose, a link needs to be properly detailed to have adequate strength and stable energy dissipation. All the other structural components (beam segments outside of the link, braces, columns, and connections) are proportioned following capacity design provisions to remain essentially elastic during the design earthquake.

- **According to clause 6.8.2(12a)**

Link web stiffeners, as previously stated, should be designed to prevent inelastic web buckling under large rotation demand. Clause 6.8.2(12a) of EC8 provides requirements for the spacing of stiffeners in short links that should be determined on the basis of equations in the latter clause depending on the level of expected ductility. See construction details in Figures 5.24 for suitable details for link length, Figure 5.25 for details for web stiffeners in short links, and Figure to 5.26 details for web stiffeners in long links.

Diverse details for stiffeners are required for long links. According to clause 6.8.2(12b), web stiffeners should be only placed at a distance of 1.5 times b from each end of the link where it is expected to have the formation of flexural plastic hinge. As shown figures 5.24, 5.25 and 5.26 details for stiffeners of intermediate length links should meet the requirements of both short and long links, as indicated by clause 6.8.2(12c).

Owing the high plastic strain concentration, besides the spacing, it is crucial to detail properly the type and the strength of the welds between the stiffeners and the link in order to guarantee a ductile link response. EC8 part 1 states that full depth intermediate web stiffeners should be considered. For links that are less than 600 mm in depth, stiffeners are required on only one side of the web of the link. The thickness of one-sided stiffeners should not be less than the link web thickness or 10 mm. For links that are 600 mm in depth or greater, similar intermediate stiffeners should be provided on both sides of the web. Moreover, the link stiffeners should be connected to the web of the link by means of fillet welds having a design strength larger than γ_{ov} f_y A_{st}, where A_{st} is the area of the stiffener.

The design strength of fillet welds connecting the stiffener to the flanges should be larger than γ_{ov} f_y $A_{st}/4$ (clause 6.8.2(13)). In order to strengthen the part of the beam outside the link at the intersection to the diagonal brace ends, full-depth web stiffeners should be provided on both sides of the web (as shown in Figures 5.23 and 5.24). These stiffeners should have a combined width of not less than b and a thickness t not less than $0.75t_w$ or 10 mm, whichever is larger, where b is the beam flange and ($b_f - 2t_w$) where t_w is the beam web thickness. Another important aspect that characterizes the link behaviour is its actual working length related to its theoretical value.

The real link length is the distance between the two intersection points between the axis of braces and the central axis of link profile. It is clear that the slope of bracing is the geometric parameter most affecting the link length. Hence, if "e" is the theoretical length assumed for design calculation, it is necessary to detail the bracing as shown in Figure 5.24(a), where the actual link length is equal or slightly smaller than the theoretical "e". Figure 5.25(b) shows an unsuitable detail. Otherwise, if it is not possible to modify the slope of the braces, it is necessary to consider the actual link length that would result larger than "e" in the design calculations.

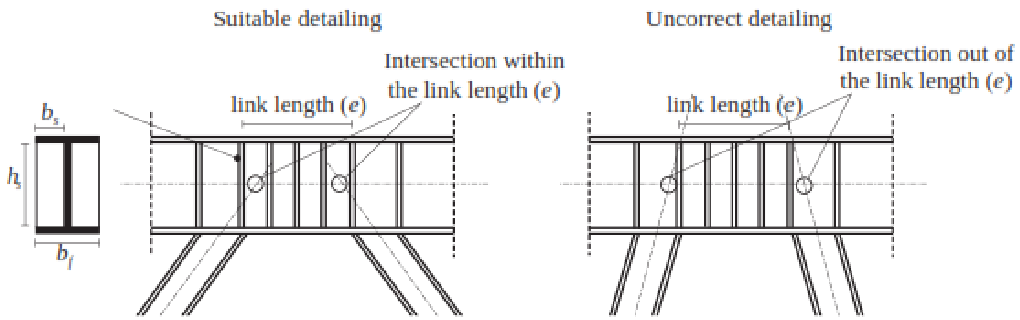


Figure 5. 24. Suitable details for link length.

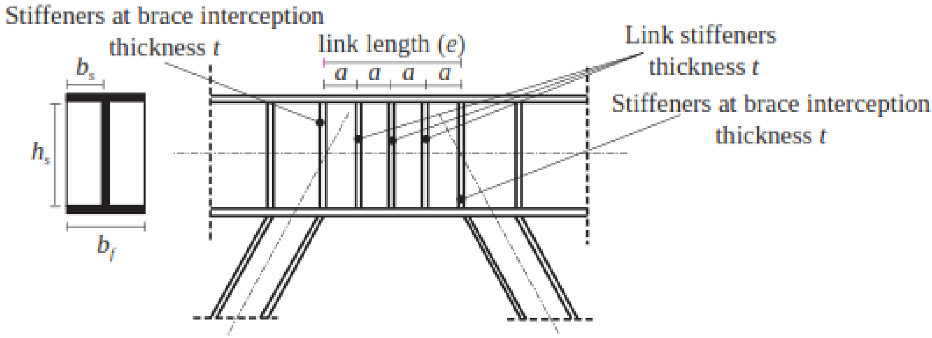


Figure 5. 25. Details for web stiffeners in short links.

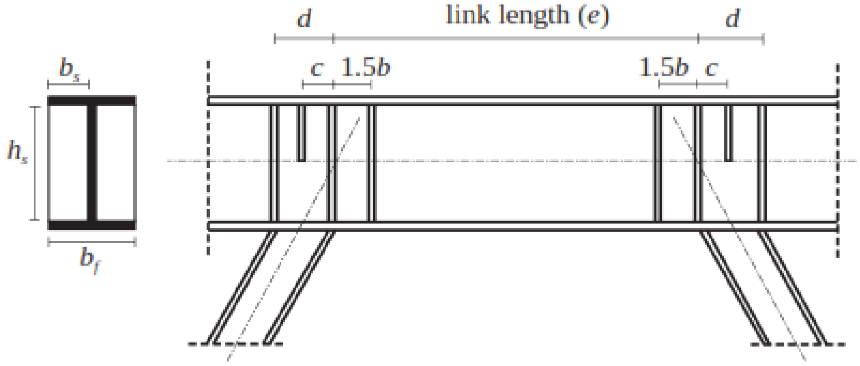


Figure 5. 26. Details for web stiffeners in long links.

5.10 COMPARISON OF EC8 AND AMERICAN PROVISIONS FOR EBF STRUCTURES

Seismic design of steel braced frames in the modern building codes follow the capacity design approach where some of the members are obliged to dissipate energy whereas others are taken care to be protected.

In this section the seismic design methodologies used by European and American approaches for and Eccentric Braced Frames (EBF) are highlighted. Synoptic table for the design of such frames of the most advance seismic codes i.e., Eurocode 8 and the seismic provisions of American Institute of Steel Construction (AISC) are provided. Another important aspect for the ductile behaviour of steel structures is the number of formations of plastic hinges; more plastic hinges can be observed in high ductile resisting systems whereas less ductile systems possess few numbers of plastic hinges. Formation of plastic hinge means dissipation of energy and therefore it is related to the behaviour factor of the structural type.

The concept of strength hierarchy in order to pre-define the location of the hinges to be form in the frame for a reliable mechanism is best explain by the chain analogy method of and **(Paulay 1992)**. As explain in previous section of this chapter, the strength of which is attributed to the weakest link, one ductile link may be used to achieve ductility for the entire chain. The nominal tensile strength of the ductile link is subjected to uncertainties of material strength and strain hardening effects when high strained. The other links are presumed to be brittle, but their failure can be prevented if their strength is in excess of the real strength of the ductile weak link at the level of ductility envisaged.

Synoptic Table for EBFs

In all seismic codes where EBF structures are covered, there are a number of special design provisions that must be satisfied by Eccentric Braced Frames as developed in previous sections of this chapter. Indeed, in EBF, link must be provided at least at one end of each brace. The link beam should be designed so that it is the weak part (dissipative zone) of the structure under severe seismic loading which is achieved by selecting the size of the steel section and the length of the link beam to match seismic load design requirements.

In the following synoptic scheme, see Table 5.1, a comparison of the capacity design rules according to Eurocodes **(EC3 2003, EC8 2005)** versus **(AISC 2010)- (ASCE 2010)** for the design of EBF, the noticeable features provided by the relevant codes are illustrated briefly given in Table 5.1 **(Naqash 2014).** From Table 5.1, it is apparent that the design provisions of AISC are rather straight forward. In the case of EBF, the overstrength factor in EC8 is given by the ratio of the plastic shear resistance to the applied design shear action when the link is short or the ratio of the plastic flexural resistance to the applied design flexural action when the link is long.

With regard to the reduction of seismic action (behaviour factor in EC8 and response modification factor in AISC) quite high factor is given by the AISC for EBF (R equals 8) compare to EC8 (q equals $5\alpha_u/ \alpha_1$ for DCH and 4.0 for DCM). In general, it is concluded that the seismic provisions of EC8 seem complicated compare to that of AISC with clear differences in the proposed values of the important factors that are normally adopted by the seismic codes.

The energy dissipation in the connections must not be encouraged; therefore, the strength of the connections should be stronger than the members themselves. Similarly, others remarks can be drawn when comparing the seismic provisions between the American and European EC8 seismic codes, among which:

- Length ratio: in the American code the length ratio is an opened interval unlike the Eurocode which provides a precise value.
- Rotational angle: the rotational angle is the same in codes, 0.02 in the long and 0.08 in the short links, and the code UBC mentioned the angle for the long to have the same as 0.02 but gives the short link a 0.09 rad value.
- Stiffeners: stiffeners are given the same classification in the two codes.

- The American code has a protocol for the seismic charge.EC8 has recently proposes a cyclic protocol.
- The link length in the American code is slightly longer than the Euro one, for example the short IPE 360 has a length of 740mm in the Eurocode and an 848mm in the American.

Table 5. 1. Provisions for Eccentric Braced Frames **(Naqash et al 2014)**.

Description	Eurocodes (EC3/EC8)	AISC/ASCE	Remarks
Energy dissipation philosophy	EBFs shall be designed so that specific elements or parts of elements called seismic links are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms.	EBFs are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake.	An almost same criterion is considered
Description	**Eurocodes (EC3/EC8)**	**AISC/ASCE**	**Remarks**
Rotation capacity (local ductility concept)	Plastic hinge rotation is limited to 35 mrad for structures of DCH and 25 mrad for structures of DCM	Link rotation angle shall not exceed (a) 0.08 radians for links of length $1.6M_p/V_p$ or less and (b) 0.02 radians for links of length $2.6M_p/V_p$ or greater.	For high seismicity it is recommended by both codes to apply ductility concept
Dissipative members	Plastic Hinges should take place in links prior to yielding or failure elsewhere.	EBFs are expected to withstand significant in-elastic deformations in the links when subjected to forces resulting from the motions of the design earthquake.	Links can be short, long and Intermediate. Which fail due to Shear, bending and bending & Shear respectively.
Design Checks	If $N_{ED} / N_{pl,Rd} \leq 0.15$ then Check for Design Resistance of Link is $V_{ED} \leq V_{p,link}$ $M_{ED} \leq M_{p,link}$	Effect of axial force on the link, available shear strength need not be considered if $P_u \leq 0.15P_y$ (LRFD) or $P_a \leq 0.15/1.5P_y$ (ASD)	N_{ED}, M_{ED}& V_{ED} respectively are the design axial force, design bending moment and design shear at both ends of the link.
Check to achieve global dissipative behaviour of the structure	The maximum overstrength Ωi should not differ from the minimum value Ω by more than 25%	Ω is a multiplicative factor which is the minimum value of Ωi=1.5V p,link,i/VED,I among all short links and minimum value of Ωi=1.5Mp,link,i/MED,I among all intermediate and long links.	
Cross section limitations	For $q > 4$ only class 1 sections are allowed, for $2 < q \leq 4$ class 1 and class 2 and for $1.5 < q \leq 2$ class 1, 2 and 3 are allowed	Limits λ_p to λ_{ps}, i.e. to use seismically compact section and is obtained by modified slenderness ratio	Class 1 and seismically compact sections are unaffected by local buckling
Seismic load reduction factor	A behaviour factor (q) equal to 4 for DCM and$5\alpha_u/\alpha_1$ for DCH is provided.	A response modification factor (R) equal to 8.0 for EBFs is given	An almost same criterion is considered
Overstrength factor	the minimum value of $\Omega_i = 1{,}5\ V_{p,link,i} / V_{Ed,i}$ among all short links, whereas the minimum value of $\Omega_i = 1{,}5\ M_{p,link,i}/M_{Ed,i}$ among all intermediate and long links;	Ω_o equal to 2 for EBFs is given	Ω_o in EC8 is $(1.1\gamma_{ov}\ \Omega)$
Drift philosophy (Reduction)	Spectrum is reduced by 2.0 and 2.5 for importance classes I & II, and III &IV, respectively	Reduction factor is (C_d/R) equals (4/8) for EBF	Overall EC8 check for drift is more stringent

CHAPTER 6:
INELASTIC CYCLIC BEHAVIOUR OF SEISMIC LINKS

CHAPTER 6: INELASTIC CYCLIC BEHAVIOUR OF SEISMIC LINKS

6.1 INTRODUCTION

All structural elements have limited strength and deformation capacities; and collapse safeties as well as damage control are depending on our ability to assess these capacities with some confidence. The structural properties of a structure deteriorate when deformations reach the range of inelastic behaviour. A possible consequence of deterioration of the hysteretic behaviour of a structure is failure of critical elements at deformation levels that are significantly smaller than its ultimate deformation capacity. It is a fact of life that earthquakes come in all magnitudes: small and large. The present state of the art does not permit an accurate prediction of the exact location of an earthquake or its size (**Sen 2009)**.

There is no unique and "best" loading history, since no two earthquakes are alike and because that a specimen may be part of many different structural configurations. The need for representative loading histories is becoming more prevalent as performance-based seismic design, which requires quantification of performance, is becoming a more widely accepted alternative to routine code design, and as more and more innovative performance enhancement systems become available.

The working mechanism of eccentrically braced frames (EBFs) is different for different intensities of earthquakes **(Li 2007)**. For small or moderate intensity of earthquakes, the contribution of the lateral stiffness of structures mainly comes from eccentric braces, which act as braces in concentrically braced frames CBFs. For major earthquakes, however, shear or bending or both yielding, depending on the length of the seismic links, can occur in eccentric beams to consume earthquake energy, and eccentric braces are prevented from compressive instability.

Seismic hazard models based on probabilistic theories are in use. Informed decision-making helps with the selection of a scenario for large earthquakes. The engineer is still faced with the task of designing for small earthquakes and large earthquakes. Large earthquakes are rare, with a low probability. Buildings subject to large earthquakes are designed to accept large deformations but not to collapse.

6.2 BEHAVIOUR OF STEEL MEMBERS UNDER CYCLIC LOADING

6.2.1 General

Steel structures are widely used in high seismic risk areas, due to their excellent performances in terms of strength and ductility. The fulfilment of the design requirements is possible by the inherent mechanical behaviour of materials, as steel is a ductile material, equally strong in compression and tension, so it is ideally suited for earthquake resistant structures, in structural elements as well as non-structural elements, but also to the wide range of possibilities, in choosing an adequate structural seismic-resistant typology (among traditional and innovative bracing systems).

6.2.2 Hysteric energy

A most important property of steels subjected to large cyclic inelastic loading is their ability to dissipate hysteretic energy **(Bruneau 2011)**. Hysteric energy is the energy dissipated by inelastic cyclic deformations and is given by the area within the load-deformation curve also called the hysteretic curve. In structures having low hysteretic energy dissipation capacities, even if the deformations are well below the ultimate deformation, the structure is likely to collapse due to low-cycle fatigue effect as the degradation of strength and stiffness under repeated inelastic cycling. Ensuring that the structure is able to dissipate a large amount of hysteretic energy in each cycle can minimise low-cycle fatigue effect (See 2.5.5 for more details).

6.2.3 Hysteretic behaviour illustration example

Buildings are required to withstand cyclic loading as consequence of earthquake action. While the design intent under operating conditions has focused on members remaining elastic which is not practical under a severe earthquake, often referred to as the ductility level earthquake Ductility is measured by the hysteretic behaviour of critical components such as a column-beam assembly of a moment frame. It is obtained by cyclic testing of moment-rotation (or force-deflection) behaviour of the assembly **(Taranath 2005)**.

As steel frames are likely to be subjected to cyclic loads under earthquake and other forms of dynamic loadings will require the study of cyclic and hysteretic behaviours. Experiments have shown that after yielding at one loading step, assuming that it is the n^{th} loading, the initial yielding stress of steel in the next unloading and reversal loading, the $(n+1)^{th}$ loading, will be lower than before, as it can be seen from Figure 6.1 representing (σ-ε) curve, in which, $\sigma_{sn+1} < \sigma_{sn}$ which is well-known as Bauschinger effect. If the reversal loading continues, the stress in steel continues to increase till it meets the ultimate yielding stress being higher than before, i.e. $\sigma_{pn+1} > \sigma_{pn}$ which is known as the strain-hardening effect. Consider σ_{un} be the stress at the beginning of unloading at the ultimate loading and $\sigma_{un} > \sigma_{pn}$ the ultimate yielding stress at the $(n+1)^{th}$ loading, σ_{pn+1}, can be approximately equal to σ_{un}, i.e. $\sigma_{pn+1} = \sigma_{un}$. For the first loading $\sigma_{s1} = \sigma_{p1} = \sigma_s$.

When a steel member is subjected to cyclic moments, the Bauschninger effect will reflect similarly in the M-ϕ relationship, Figure 6.2, where $M_{sn+1} < M_{sn}$. In the same way and due to strain hardening effect $M_{pn+1} > M_{pn}$ and $M_{pn+1} = M_{un}$. For the first loading, $M_{sn1} = M_s$ and $M_{un} = M_p$.

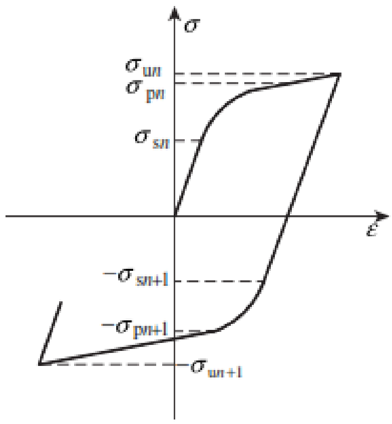


Figure 6. 1. Stress–strain curve of steel under cyclic loading **(Li 2007)**.

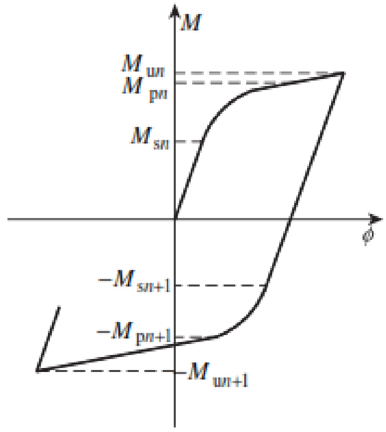


Figure 6. 2. Moment–curvature curve under cyclic loading **(Li 2007).**

The hysteretic curves also show the inelastic deformation that can be tolerated at various resistance levels. The common grades of mild steel, i.e. S235, have satisfactory ductility and perform well under cyclic reversal of stresses.

6.2.4 Hysteretic models

The design of a structure in accordance with seismic provisions will not fully ensure against earthquake damage because the horizontal deformations that can be expected during a major earthquake are several times larger than those calculated under design loads.

The definition of deformation may be measured in terms of deflection, rotation or curvature. The numerical value of ductility will also vary depending on the particular combination of applied forces and moments under which the deformations are measured. Ductility is generally desirable in structures because of the gentler and less explosive onset of failure than that occurring in brittle materials.

This ductility is particularly useful in seismic problems because it is accompanied by an increase in strength in the inelastic range **(Chen 2006).**

- **Definition of a model**

A hysteretic model is the one that describes the relationship between force and displacement of structural members under cyclic loading conditions. Factors such as structural material, structural system, and connection configuration influence the hysteretic behaviour. Consequently, arriving at an appropriate mathematical model to describe the inelastic behaviour of structures during earthquakes is a difficult task. Other hysteretic models such as stiffness and strength degrading have also been suggested **(Chen 2006).**

- **Bauschinger effect**

Bauschinger effect is a general phenomenon associated with most polycrystalline metals. It refers to the property of a material for which the stress–strain characteristics change because of the macroscopic stress distribution of the material. The mechanism for the Bauschinger effect is related to the dislocation of structure in the cold worked metals. The plastic deformation of the metal increases the tensile yield strength and decreases the compressive yield strength. This decrease in the yield strength takes place only when the direction of the strain changes **(Duggal 2013)**.

- **Simple example of model**

As previously discussed, the ductility of a member or structure may be defined in general terms by the ratio ductility = deformation at failure/deformation at yield. The favourable ductility of mild steel may be seen from Figure 6.3(a) by the large value of ductility in direct tension measured by the ratio ε.

A simple model which has extensively been used to approximate the inelastic behaviour of structural systems and components is the bilinear model. The bilinear model can represent the basic hysteretic behaviour of a steel beam under cyclic loading, but cannot reflect the nonlinear phase from initial yielding to ultimate yielding very well. In this model, unloading and subsequent loadings are assumed to be parallel to the original loading curve.

In the reversed loading of steel, the Bauschinger effect occurs, as explain in the previous section, i.e. after loading past the yield point in one direction the yield stress in the opposite direction is reduced. Another characteristic of the cyclic loading of steel is the increased non-linearity in the elastic range which occurs with load reversal (Figure 6.3).

Stiffness degradation is an important feature of inelastic cyclic loading of concrete and masonry materials. The stiffness as measured by the overall stress/strain ratio of each hysteresis loop of Figure 6.3 (c)–(f) is clearly reducing with each successive loading cycle.

- **Other proposed models**

A large amount of effort has been made worldwide to the stress–strain relationship of the steel material under cyclic loading. Several hysteretic models have been developed, where the simplest one is the perfectly elastic–plastic model (see Figure 6.3(a)) ignoring the strain-hardening effect and the Bauschinger effect. The models given in Figure 6.3(b)–(d) can consider both the strain-hardening effect and the Bauschinger effect. The model in Figure 6.3(b) is a bilinear model and the other two are trilinear models with, respectively, softening phase and yielding plateau.

The bilinear model is employed in this chapter because it can capture the principal characteristics of the steel material under cyclic loadings and is convenient to programming **(Li 2007)**.

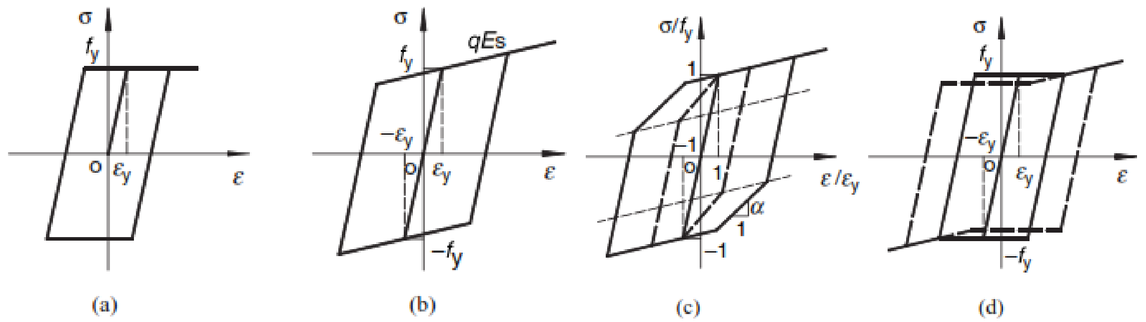


Figure 6.3 Hysteretic stress–strain relationships of the steel material:
(a) Perfectly elastic–plastic model; (b) Bilinear model; (c) Trilinear model with softening;
(d) Trilinear model with yielding **(Li 2007).**

6.2.5 Hysteretic model of shear beam

For the particular case of shear beam, the hysteretic model many specimens were tested to investigate the elasto-plastic hysteretic relationship between the shear and shear deformation of shear beams, as already explain in previous chapters, **(Kasai 1986a; Roeder 1978.** The cyclic test set-up and specimen of a shear beam is given in Figure 6.4.

The following findings can be drawn from the test results:

(1) Shear beams have stable hysteretic performance and very good ductility. The ductility factor of shear beams (ratio of maximum shear strain to shear yielding strain) may be larger than 100.
(2) The dominant failure mode of shear beams is the local buckling of beam webs. This failure can be restrained or delayed by adding stiffeners to beam webs.

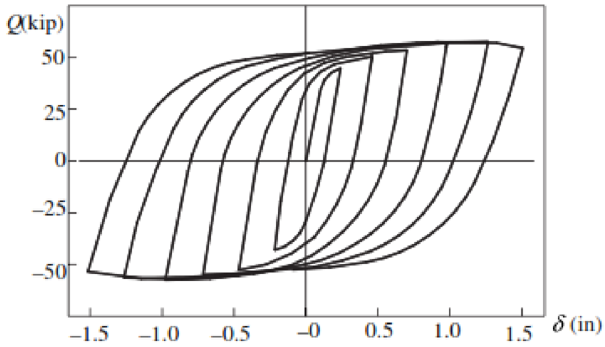


Figure 6. 4. Experiment of a shear beam under cyclic hysteretic curves.

Figure 6.5 shows a typical behaviour of a steel member under cyclic bending and with a constant axial force (a beam column). With $N_y = A_s f_y$ where A_s is the sectional area of the member and f is the specified yield stress. Since earthquake loads are cyclic, the beam segment outside of the link experiences both axial tension and axial compression. Designing the beam segment for these high moments and axial forces can be difficult, which is described in greater details later on.

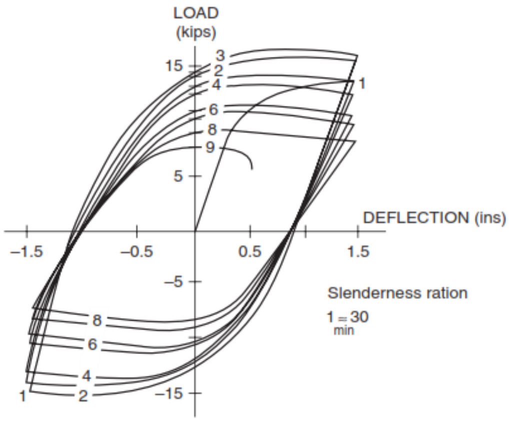


Figure 6. 5. Hysteresis loops for a steel member under cyclic bending and a constant axial force of N = 0.3Ny **(Vann 1973).**

Figure 6.6 shows an example of the hysteretic response of a beam-to-column connection obtained from a quasi-static cyclic test.

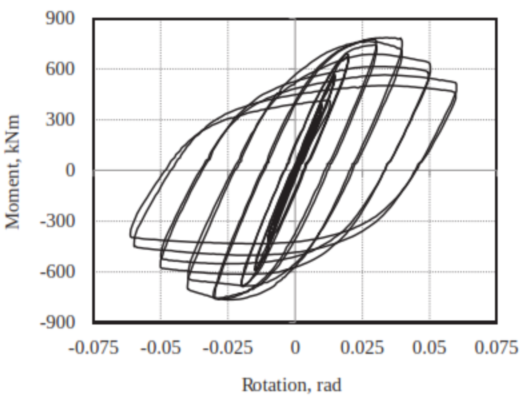


Figure 6. 6. Example of hysteretic response of a beam-to-column connection obtained as result of quasi-static cyclic testing **(Landolfo 2017).**

6.2.6 Hysteretic model of ordinary braces

The members of braced frame act as a truss system and are subjected primarily to axial stress. Current research shows that significant inelastic deformation occurs in the beams and columns of braced frames in addition to the buckling of the brace.

Depending on the diagonal force, length, required stiffness, and clearances, the diagonal members can be made of double angles, channels, tees, tubes or even wide flange shapes **(Subramanian 2011).**

The hysteretic model of braces is a mathematical approach for describing the hysteretic relationship between the axial force and the axial deformation of braces. The hysteretic model can be used to determine the axial stiffness of braces subjected to repeated and reversed loadings. It is found from experimental investigations and theoretical analysis that the hysteretic behaviour of braces is very complicated (**Li 2007).** However, for the purpose of engineering applications, the hysteretic model of braces can be simplified by reserving the principle characteristics, as shown in Figure 6.7. The hysteretic phase of braces depends on the cyclic state of the axial force and deformation of braces during the loading process.

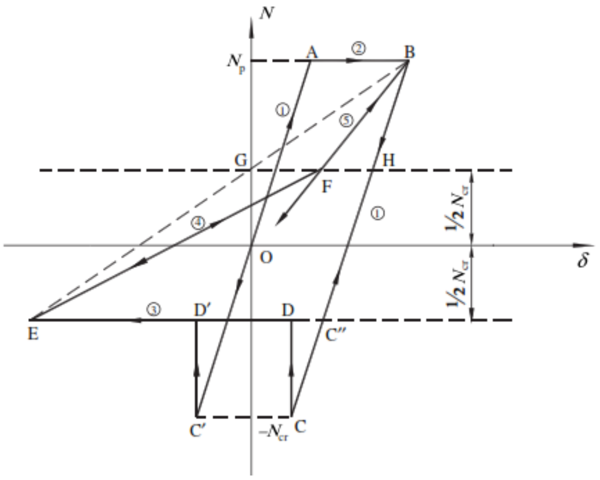


Figure 6. 7. Simplified hysteretic model of ordinary braces **(Li 2007).**

6.2.7 Behaviour under bending cyclic loading

In steel flexural members subjected to cyclic loading, strength deterioration is often caused by cracks in the zone of maximum inelastic deformation because of repeated bending or by local buckling and/or lateral buckling of the web following local buckling of the flange. Hysteresis loops for small rotation amplitude are stable, but strength degradation becomes severe when the rotation amplitude exceeds a value which is less than half of the rotation capacity under monotonic loading.

In Figure 6.8, the typical hysteresis loops of a steel beam are shown, wherein, the decay is mainly due to web buckling. Flange buckling and lateral-torsional buckling also influence the loss of strength and stiffness of the beams to some extent, and therefore, a shorter, laterally unsupported length must be specified for beams subjected to cyclic loading. Both the flanges of beams should, therefore, be laterally supported, directly or indirectly.

In a potential plastic-hinge region, the width-to-thickness ratio of the beam should be kept small, and the lateral braces should be spaced with a small pitch to ensure sufficient rotation capacity of the beam. Outside the plastic-hinge regions, beams need only resist external forces (ductility not required), and, therefore, a larger spacing of lateral braces is allowed.

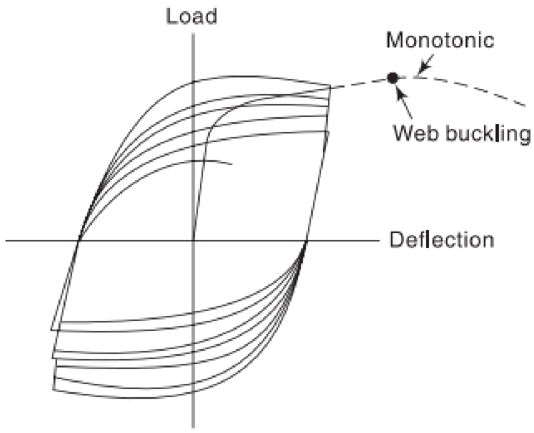


Figure 6. 8. Typical hysteresis loops for a steel beam under cyclic bending **(Duggal 2013).**

6.3 INSTABILITY DUE TO AN EARTHQUAKE ACTION IN STEEL STRUCTURES AND MEMBERS

Structural elements which are subjected to tensile forces are inherently stable and will generally fail when the stress in the cross-section exceeds the ultimate strength of the material. In the case of elements subjected to compressive forces, secondary bending effects caused by, for example, imperfections within materials and/or fabrication processes, inaccurate positioning of loads or asymmetry of the cross-section, can induce premature failure either in a part of the cross-section, such as the outstand flange of an I section, or of the element as a whole. It is also important to note that the forces due to earthquakes are not static but dynamic, (cyclic and repetitive) and hence the deformations will be well beyond those determined from the elastic design.

Buckling may be defined as a structural behaviour in which a deformation develops in a direction or plane perpendicular to that of the load which produced it; this deformation changes rapidly with variations in the magnitude of applied load (**Subramanian 2011)**.

Buckling occurs mainly in members that are subjected to compression **(Dowling 1988)**. Its effect is to decrease the load carrying capacity of a structure (i.e., reduce the strength) and also to increase the deformation (i.e., reduce the stiffness. Considerable care is also needed to check failures due to instability and brittle fracture to ensure the development of full ductility and energy dissipation capacity under earthquake loading.

In such cases, the failure mode is normally buckling (i.e. lateral movement), of which there are three main types:

- Overall buckling
- Local buckling
- Torsional buckling

The causes of instability in a steel member are as follows **(Duggal 2013):**

- **Local buckling of plate elements** (e.g., web, flange): A steel member containing plate elements with a large width-to-thickness ratio is unable to reach its yield strength, because of prior local buckling. Even if the yield strength is attained, ductility will be inadequate. Under cyclic loading, the strength and ductility decrease with increasing width-to-thickness ratio, and local buckling of web causes further degradation.
- **Flexural buckling of long columns and braces:** Long columns may fail by buckling. This mode of instability is sudden and can occur when the axial load in a column reaches a certain critical value. In most cases, the stress in the column may never reach the yield. Even a small lateral force under such conditions will produce a substantial deflection leading to instability and the phenomenon is called flexural buckling.

The capacity of slender columns is, therefore, limited by the stiffness of the member, rather than by strength of the material. The lateral stiffness of frames, therefore, is increased by bracing the frames. However, buckling of braces is a potential source of instability of steel frames. Steel bracing dissipates considerable energy by yielding under tension, but buckle without much energy dissipation in compression. Therefore, the energy dissipation capacity of concentrically braced frames is markedly less, due to buckling of braces, than that of the moment frames.

- **Lateral-torsional buckling of beams:** During moderate-to-strong shaking of the ground, additional forces are developed in various members of a structure. For a beam loaded in flexure, the load bearing side (generally the top) carries the load in compression, whereas the non-load bearing side (generally the bottom) will be in tension.

If the beam is not supported in the opposite direction of bending, and the flexural load increases to a critical limit, the beam will fail due to local buckling on the compression side. In wide-flange sections, designed for flexure only, if the top flange buckles laterally, the rest of the section will twist, resulting in a failure mode known as lateral-torsional buckling.

- **P-Δ effects** in frames subjected to large vertical loads if the lateral stiffness is not high enough, the building as a whole, or one or more storeys can fail due to the P-Δ effect. This is due to the secondary effect on shears and moments of the frame members, caused by the action of vertical loads, which interact with the lateral displacement of the building resulting from seismic forces (Appendix X).
- **Uplift of braced frames** Earthquakes have a vertical component of movement in addition to the traditionally considered horizontal effects. The stresses produced due to vertical motion are generally considered not to be significant to cause instability. However, due to the horizontal component of movement, the overturning moments produce additional longitudinal stresses in walls and columns and additional upward (uplifting) and downward (thrust) forces in foundations causing instability.

6.4 CONCISE REVIEW ON EBF STRUCTURES (EBFs)

In the following section, concise review on EBF steel structures will be given; details can be found in chapter 4 of this thesis.

6.4.1 General

Seismic resistant eccentrically braced frame systems (EBFs) are a lateral load-resisting system for steel buildings that combine high stiffness in the elastic range with good ductility and energy dissipation in the inelastic range. Also, EBFs can be viewed as a hybrid between concentrically braced frames (CBFs) and moment resisting frames (MRFs).

EBF structures exhibit good strength and stiffness in elastic range, so avoiding non-structural damage, and are also able to provide enough ductility to dissipate large amounts of energy in the inelastic range. The typology of the bracing members in the EBFs ensures EBF structures exhibit good strength and stiffness in elastic range, so avoiding non-structural damage, and are also able to provide enough ductility to dissipate large amounts of energy in the inelastic range. Under severe cyclic loading, the eccentricity of the layout is such that energy can be dissipated through seismic links by either cyclic bending or cyclic shear. This ability is based largely on the capability of EBF to undergo inelastic deformation restricted and localized primarily in the links, which are designed and detailed to sustain such inelastic deformations without loss of strength.

Yet, under severe earthquake loading, as it is the case for major earthquakes, have to be as properly designed and detailed EBFs in order to provide sufficient ductility and energy dissipating. In the inelastic range the principle mechanism for energy dissipation is through inelastic deformation, and this is taken into account through the hysteretic behaviour of the restoring force.

6.4.2 Basic definition and overall behaviour of EBF

6.4.2.1 Definition

According to ASCE (2010), Eccentrically Braced Frame (EBF) is a diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace. Eccentric braced frames (EBFs) represent an economically effective way of designing steel structures for earthquake loading. Indeed, by selecting a suitable frame stiffness and yield level, it is possible to resist moderate earthquakes elastically, with only moderate displacements, and to resist major earthquakes inelastically.

The primary benefit of EBFs is that substantial system ductility can be developed. Moreover, because of the ease with which access can be gained through the plane of the braced panel, they may be located within the building. One potential drawback is the possibility of floor damage near the link beam (Figure 6.9) during major earthquakes, but in view of the levels of damage normally regarded as acceptable, this is not serious. Because EBFs possess high elastic stiffness in addition to significant energy dissipation and high degree of ductility at inelastic range, it can be thought as a hybrid system between moment resisting frames (MRFs) and concentrically braced frames (CBFs). Eccentric braced frames shall be defined as braced frames where component axes do not intersect at a single point and the eccentricity exceeds the width of the smallest member at the joint.

6.4.2.2 Basic behaviour

In structural analysis an (EBFs) are lateral force resisting systems providing good inelastic capacity for steel structures under large cyclic loading, Figure 6.9. The geometry of frames with eccentric bracings is close to that of frames with concentric bracings; some intentional eccentricities in the layout of bars generate bending moments and shear. These structures resist horizontal forces essentially by axially loaded members, but they are designed to yield first in shear or bending in 'seismic links'. The latter are zones created by the shift of bars of the reference concentric brace away from the usual intersection with other bars (Figure 6.9).

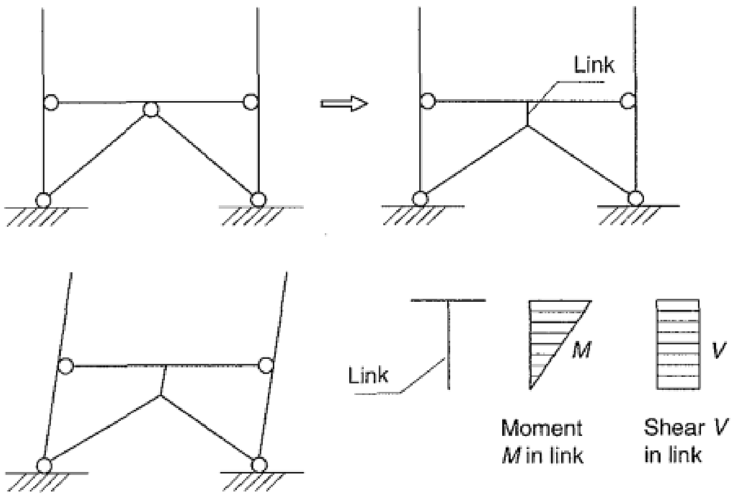


Figure 6. 9. Conversion of concentric V bracing to eccentric bracing with a vertical link **(Fardis 2005)**.

6.4.2.3 Characteristics of EBFs

The main characteristic of EBFs is that the eccentric connection of brace with beam causes a weak, small beam segment named as a link. The other components of EBFs are columns, braces and beam outside of the link.

When a typical EBF is subject to lateral load, the link transmits high shear, high bending moment, and typically low levels of axial force. Consequently, links will normally experience shear and/or flexural yielding during an earthquake. Other members of an EBF, including the braces, the columns and the beams segments outside of the links are intended to remain essentially elastic during an earthquake.

As shown in Figure 6.10, eccentrically braced frames subjected to lateral sway deform in such a way as to cause distress in the floors and non-structural elements.

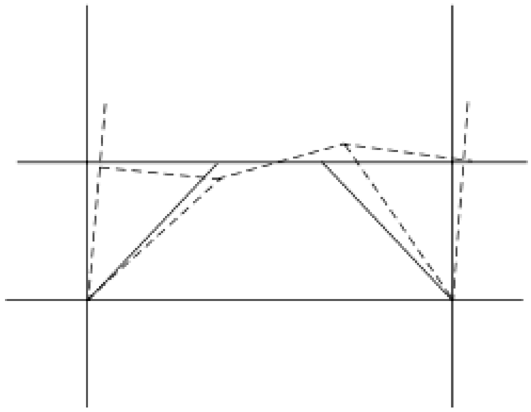


Figure 6. 10. Deformed shape of eccentrically braced frame subjected to lateral sway, **(Dowrick 2003)**.

6.4.3 Seismic links

Eccentric braced frames shall be defined as braced frames where component axes do not intersect at a single point and the eccentricity exceeds the width of the smallest member at the joint. The section between these points is defined as the link component with a span equal to the eccentricity. The link in EBF is a ductile segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column and specially designed to act as an energy dissipater. The length of the link is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face **(Chen 2006).**

The section between these points is defined as the link component with a span equal to the eccentricity. For a short link, energy is dissipated primarily through inelastic shearing of the link web on the other hand for a long link; the energy is dissipated primarily through flexural yielding at the ends of the link.

An eccentrically braced frame (EBF) is a type of steel framing system including beams, columns and braces, where these members are arranged in a manner where at least one end of each brace is connected to isolate a segment of the beam called a link. EBFs are typically used as a lateral force resisting system for earthquake loading. EC8 provides details concerning the EBF structures in chapter 6 of **(EC8 2004).**

The code provisions of both AISC (2005) and EC8 will be discussed later on in this chapter, as no relative provisions on EBF structures exist so far in RPA99. The design intent for a seismic resistant EBF is to provide high ductility under earthquake loading by yielding of the link. Design requirements for seismic resistant EBFs in the US are specified by the AISC Seismic Provisions for Structural Steel Buildings **(AISC 2005).**

6.5 DESIGN AND FAILURE MECHANISM OF SEISMIC LINKS

6.5.1 General definition of a seismic link

The link is the key distinguishing feature of an EBF from CBF steel structures, which is, in fact a portion of the beam set out between the braces or between one brace and the column and are the components devoted to dissipate seismic energy by means of either cyclic bending or cyclic shear depending on the link length.

6.5.2 Expected behaviour of seismic link

Special steel moment-resisting frames and eccentric braced frames, see chapter 4 for details are capable of developing large plastic deformations and large hysteretic areas. As a result, they are designed for larger values of *R*, thus smaller seismic forces and greater inelastic deformation. This hysteretic behaviour is important, since it dampens the inelastic response and improves the seismic performance of the structure without requiring excessive strength or deformation in the structure **(Brockenbrough 2006).**

- **Conditions of inelastic response**

According to **Ziemian (2010),** adequate seismic inelastic response can only be achieved if the following two basic conditions are satisfied:

(1) proper detailing and bracing are provided to ensure that the intended yielding components can sustain the anticipated cyclic inelastic deformation demand, without strength degradation or fracture, so that the lateral strength and energy dissipation capacity of the structure is maintained during the earthquake;

(2) Proper strength and yielding hierarchy is implemented in the structure to ensure that the intended yielding mechanism actually forms and the structural integrity is preserved under strong ground shaking.

The seismic links are designed to enforce plastic engagement of all dissipative links and expected to undertake large nonlinear deformations by an excellent dissipation of energy without losing resistance.

- **Differences on behaviours of links**

There are substantial differences between the behaviour of short and long links **(EC8 2005; AISC 2005, UBC 1997)**. The plastic hinge model may be derived to reproduce the pure bending moment rotation response (e.g. for modelling plasticity of beams or long links), the axial force-axial displacement response (e.g. for modelling plasticity of braces) or the shear force-shear distortion response (e.g. for modelling plasticity of shear links). In addition, they can be extended in order to account for the interaction between bending moment and axial force (e.g. for modelling plasticity of a column).

Generally speaking, the behaviour of link elements, and afterwards the way they dissipate energy, is related to their length (e): short links (i.e. characterized by a ratio between the plastic shear and the plastic moment smaller than 1.6 times the link length) generally develop high shear deformations, while long links (i.e. characterized by a ratio between the plastic shear and the plastic moment higher than 2.5 times the link length) mainly dissipate energy trough the formation of flexural deformations **(Okazaki 2007)**.

(a) Static equilibrium of link (free body equilibrium)

(b) Typical force distributions in beams and links of EBF's under lateral load.

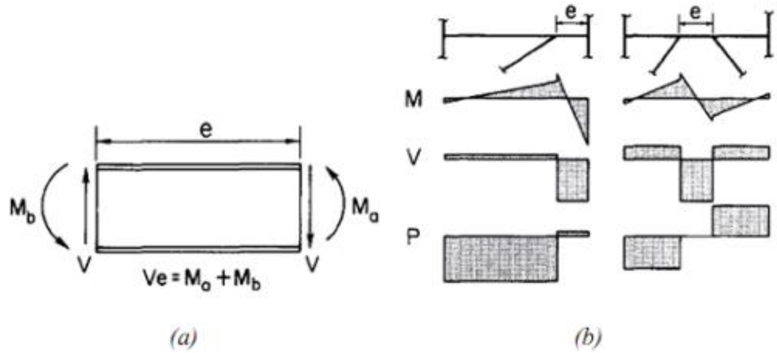


Figure 6. 11. Behaviour of links in EBF Structures **(Popov 1988)**.

6.5.3 Previous research on seismic links

Previous research studies show that behaviour of links is fairly complicated and affected by various parameters, and as a result, significant amount of research interest has been directed towards both experimental and numerical determination of the nonlinear behaviour and cyclic energy dissipation characteristics of the links.

The ability of EBFs in dissipating energy strictly depends on the criteria adopted in the design: the plastic deformations are essentially located on link elements, dimensioned for yielding before beams, braces and columns that, otherwise, are proportioned using the forces generated by the yielded and hardened links in order to remain in the elastic field, according with the principles of capacity design **(Bruneau 2011)**.

Previous published analytical and experimental researches, available in literature, have demonstrated that when properly designed eccentrically braced frame (EBF) systems can provide the ductility and energy dissipation capacity needed to serve as an effective lateral load resisting system to resist earthquake demands **(Hjelmstad 1983a; Malley 1984; Kasai 1986a; Badalassi 2013)**. Although longer links provide more architectural freedom for openings, previous studies showed that the performance of short links is considerably better than that of long links under severe cyclic loadings in terms of strength and ductility.

6.5.4 Design of seismic links

Design of EBFs habitually starts by selecting the length of links, e, at all levels based on seismic code criteria **(EC8 2005; AISC 2005; UBC 1997)**, such as architectural constraints.

After sizing the links, the selected length of link should be checked using material properties in order to satisfy code equations EC8 **(EC8 2005)**, AISC **(AISC 2005)** to determine which category of links belongs to. Equations to determine the length ranges and allowable link inelastic rotation angles have been developed for I sections as specified in chapter 8 of EC8 and AISC Seismic Provisions. In these provisions, the length of link-beams affects the type of hinges and consequently the type of mechanism.

As reported in many researches works, the ultimate failure modes of short links and long links are quite different. For a short link, energy is dissipated primarily through inelastic shearing of the link web on the other hand for a long link; the energy is dissipated primarily through flexural yielding at the ends of the link.

The inelastic response of a link is strongly affected by the link length and the (M_p/V_p) ratio of the link cross-section. Here, M_p = nominal plastic flexural strength of the link; V_p = nominal shear strength of the link. Considering simple plastic theory assuming no strain hardening and (M–V) interaction, it can be observed that e = ($2M_p/V_p$) is the theoretical dividing line between a shear link and a flexural link. The shear yielding energy dissipation mechanism is more efficient than the flexural plastic hinging mechanism **(Bruneau 2011)**. Typical distribution of actions (axial force P, bending moment M and shear force V) in the braces, beams and links of a D braced frame and split K braced frame are shown in Figure 6.11 **(Popov 1988)**.

6.5.5 Link stiffeners

In order to guarantee a sufficient link rotation capacity, link web stiffeners should be designed and placed. Owing the high ductility demand on links, the flange area of the link, surface of cross section of links might experience buckling phenomena; thus, it is required to install web stiffeners which are fundamental details that allow to prevent inelastic web buckling, which impairs the link performance in the range of the expected ductility demand In the case of absence of stiffeners, premature buckling may occur on the web which in turn might cause local buckling of parts of the section.

In previous studies, **(Hjelmstad 1983a)** developed several cyclic tests in order to relate the web stiffeners spacing to link energy dissipation. Subsequently, **(Kasai 1986a)** developed simple rules to relate stiffeners spacing and maximum link inelastic rotation up to web buckling.

- **Effect of stiffeners**

Considering the problematic effects of the stiffeners on the overall behaviour of EBF links, it must be reminded that stiffeners should be provided and placed symmetrically on both sides of the link web on both sides of the link web at the diagonal brace ends. Further, the link needs to be stiffened in order to delay the onset of web buckling and to prevent flange local buckling **(EC3 2003; Krawinkler 2009a; FEMA 2007)**. Also, previous research, such as **(Hjelmstad 1983a, Badalassi 2013)** have determined a few simple requirements regarding web stiffener spacing with a maximum inelastic rotational angle (γ_p) to the beginning of web torsional buckling.

Assuming that the link web buckling modes are similar to those of a plate under shear loading, they applied plastic plate shear buckling theory to relate the stiffeners spacing to the maximum deformation angle of a shear link, thus deriving simple expressions for each required link deformation capacity. The shear yielding energy dissipation mechanism is more efficient than the flexural plastic hinging mechanism **(Ramadan 1991)**.

6.6 LOADING PROTOCOL

6.6.1 Introduction

As well-known, there is no unique and "best" loading history, because no two earthquakes are alike and because the specimen may be part of many different structural configurations. The need for representative loading histories is becoming more prevalent as performance-based seismic design, which requires quantification of performance, is becoming a more widely accepted alternative to routine code design, and as more and more innovative performance enhancement systems become available. Present codes, standards, and guidelines make reference to the need for performance assessment through testing, but with few exceptions (e.g., AISC 341-05 and testing of base isolation systems in ASCE (2010)) they remain mostly silent on testing and acceptance criteria to be used for this purpose.

6.6.2 Objective of cyclic loading protocol

The objective of a cyclic seismic loading protocol is to simulate the number of inelastic cycles, cumulative inelastic demand, and peak displacement demand associated with a design seismic event **(Krawinkler 2009a)**. The cyclic loading protocol for the links was used to impose deformation demands consistent with earthquake loading effects. The loading protocol was adapted from the AISC **(AISC 2005)** quasi-static cyclic deformation controlled.

Several existing loading protocols have been developed in the literature for different types of structural and non-structural components **(ATC-24 1992; FEMA 2007; EN-12512 2001; Imani 2015; Suswanto et al 2017**, which recommend slightly different loading histories, but in most cases, they differ more in details than in the concept. These protocols are used for quasi-static cyclic testing of structures and are based on recordings from regions of high seismicity. Hence, existing loading Protocols may over estimate seismic demands for regions of low to moderate seismicity. As the overriding issue is to account for cumulative damage effects through cyclic loading, the protocol of **AISC (2005)** has been used and applied to all models.

6.6.3 Concepts behind development of loading protocols

The cyclic loading protocol requires important considerations because the damage is a cumulative process and is affected by the history of excursions (an excursion is the path from one peak loading value to the next loading peak value). The loading history should be defined such that the number of cycles experienced by the component at the onset of significant damage states is of the same order of magnitude as that experienced by real components in buildings subjected to strong earthquake motion **(Landolfo 2017)**.

The overriding issue is to account for cumulative damage effects through cyclic loading. If there is no cumulative damage, there is no need for cyclic loading. The number and amplitudes of cycles applied to the specimen may be derived from analytical studies in which models of representative structural systems are subjected to representative earthquake ground motions and the response is evaluated statistically.

Unfortunately, in earthquake engineering, strength and deformation capacities depend (sometimes weakly and sometimes strongly) on cumulative damage, which implies that every component has a permanent memory of past damaging events and at any instance in time it will remember all the past excursions (or cycles) that have contributed to the deterioration in its state of health. Thus, performance depends on the history of previously applied damaging cycles, and the only reasonable way to assess the consequences of history (short of developing complex analytical models that can be used for damage state predictions) is to replicate, to the best we can, the load and deformation histories a component will undergo in an earthquake (or several earthquakes.

6.6.4 Examples used loading protocols

6.6.4.1 US protocols

Many loading protocols have been proposed in the literature and several have been used in multi-institutional testing programs **(ATC-24 1992; Clark 1997; Krawinkler 2000)**, or are contained **(FEMA 2007; AISC 2005; ASTM 2003).**

Testing protocols for determining the seismic performance characteristics of structural and non-structural components have been developed in the United States **(FEMA 2007; AISC 341 2016)**. These protocols recommend somewhat different loading histories, but in most cases, they differ more in detail than in concept. Cyclic testing, is generally performed at progressively increasing amplitudes, aims at simulating the alternating character of the seismic load. When compared to monotonic testing, cyclic testing has the advantage of inducing alternating inelastic deflections in the specimens, thus reproducing more precisely the real stress and deformation levels within tested elements.

The FEMA 461 guidelines were developed shortly after the 1994 Northridge earthquake, and were limited to the evaluation, repair, modification, and design of welded steel moment frame structures in seismic regions.

The AISC 341-16 document specifies loading protocols for qualification testing of connections and other components, based upon the SAC guidelines and subsequent modifications made in 2002 and 2005 editions. Similarly, in Europe EN 15129 **(CEN 2009)** may be used for the determination of performance characteristics by means of specific experimental tests. Similarly, to Europe, some protocols are being developed.

In the following, some details will be provided for different proposed cyclic protocols used in the US as shown in Figures 6.12 to 6.15.

- **ATC-24 Protocol for steel (ATC-24, 1992) (Figure 6.12)**

This protocol, which was specifically developed for components of steel structures, was one of the first formal protocols developed in the U.S. for seismic performance evaluation of components using a cyclic loading history. It uses the yield deformation, Δ_{yield}, as the reference for increasing the amplitude of cycles. The history contains at least 6 elastic cycles (amplitude $< \Delta_{yield}$), followed by three cycles each of amplitude Δ_{yield}, $2\Delta_{yield}$ and $3\Delta_{yield}$, followed by pairs of cycles whose amplitude increases in increments of Δ_{yield} until severe cyclic deterioration occurs.

The relative and absolute amplitudes of the cycles were derived from statistical studies of time history responses of SDOF systems, and therefore represent global (roof or storey) drift histories and not local deformation histories such as those experienced, for instance, by links in eccentrically braced frames. This protocol was employed in phase I of the SAC steel program. In the SAC Phase I experiments it was found that inconsistent measures of "yield deformation" were employed by different investigators (for test control, Δ_{yield} at yield had to be predicted before the test), which led to results that were difficult to compare with each other. Thus, the choice of a "yield deformation" as the test control parameter appears to be ambiguous even for steel (and certainly more so for materials such as reinforced concrete and wood). This discovery came too late to prevent adoption of this protocol in the 1997 AISC Seismic Provisions (Appendix S) for qualification testing of steel components.

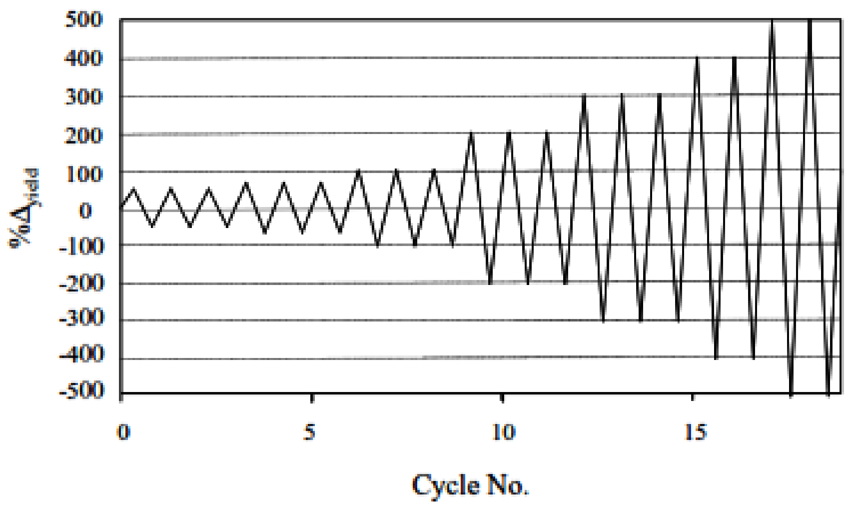


Figure 6. 12. Loading protocol for Steel ATC-24 **(ATC-24 1992).**

- **Steel - SAC Protocol (Clark 1997) (Figure 6.13)**

Because of the Δ_{yield} ambiguity, and because of the opportunity the SAC program offered to develop a specific loading protocol for steel moment frames, a statistical study was performed on the number and amplitudes of storey drift cycles of the SAC Los Angeles and Seattle 3 and 9 storey frame structures **(Krawinkler 2000)**. For steel frame structures the storey yield drift is confined to a rather narrow range around 0.01 radians, which permits an approximate correlation between the ATC-24 and SAC protocols.

The SAC protocol contains smaller (elastic) cycles (which were added because of the observed Northridge weld fractures that occurred before yielding took place), two cycles of intermediate amplitude of 0.015 radians, but slightly fewer cycles of larger amplitude. In general, the two protocols are very similar in cumulative damage potential, but because of the commitment to storey drift as the control parameter, the SAC protocol should not be applied to configurations other than steel beam-to-column assemblies that are representative of typical stories.

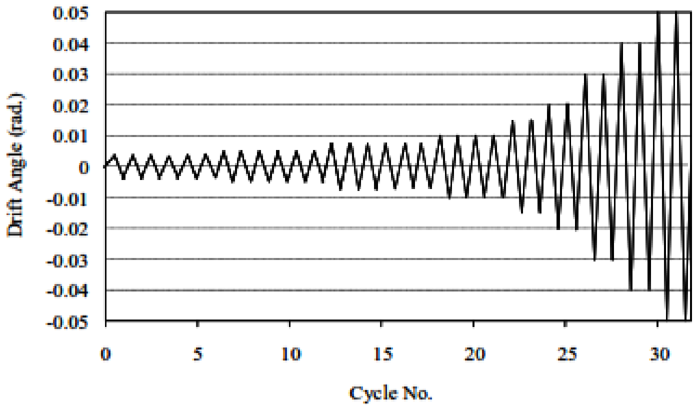


Figure 6. 13. Loading protocol for steel SAC **(Clark 1997).**

- **FEMA 461 (FEMA 2007) (Figure 6.14)**

The FEMA 461 guidelines were developed shortly after the 1994 Northridge earthquake, and were limited to the evaluation, repair, modification, and design of welded steel moment frame structures in seismic regions.

The FEMA 461's protocol was developed originally for testing of drift sensitive non-structural components, but is applicable in general also to drift sensitive structural components. It uses a targeted maximum deformation amplitude, Δ_m, and a targeted smallest deformation amplitude, Δ_0, as reference values, and a predetermined number of increments, *n*, to determine the loading history (a value of $n \geq 10$ is recommended). The amplitude "ai" of the step-wise increasing deformation cycles is given by the equation ai+1/an = 1.4 (ai/an), where a1 is equal to Δ_0 (or a value close to it) and an is equal to Δ_m (or a value close to it). Two cycles are to be executed for each amplitude. If the last damage state has not yet occurred at the target value Δ_m, the loading history shall be continued by using further increments of amplitude of $0.3\Delta_m$.

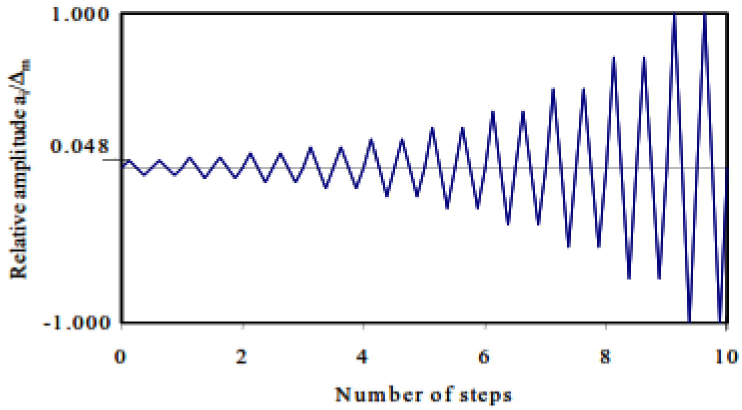


Figure 6. 14. Protocol of FEMA 461 **(FEMA 2007).**

In recognition of different behaviour and demands for structural components, the AISC 341-16 document specifies different loading protocols for beam-to-column connections, link-to-column connections and also buckling restrained braces. Figure 6.15 shows Loading protocol specified in AISC 341-16 for testing beam-to-column connections

Qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames shall be conducted by controlling the inter-storey drift angle, imposed on the test specimen, as specified below

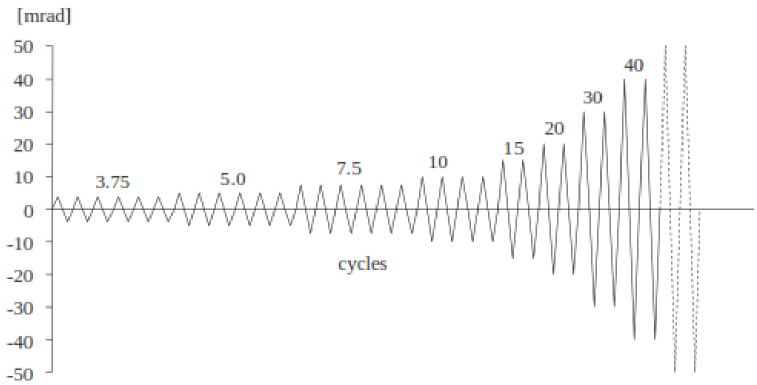


Figure 6. 15. Loading protocol specified in AISC 341-16 for testing beam-to-column connections.

- Loading protocol used in this investigation (Loading Sequence for Beam-to-Column AISC **(2005, 2006, 2010).**

- **Moment Connections**

Qualifying cyclic tests of link-to-column moment connections in eccentrically braced frames shall be conducted by controlling the total link rotation angle, γ_{total}, imposed on the test specimen, as specified below:

(1) 6 cycles at $\theta = 0.00375$ rad

(2) 6 cycles at $\theta = 0.005$ rad

(3) 6 cycles at $\theta = 0.0075$ rad

(4) 4 cycles at $\theta = 0.01$ rad
(5) 2 cycles at $\theta = 0.015$ rad
(6) 2 cycles at $\theta = 0.02$ rad
(7) 2 cycles at $\theta = 0.03$ rad
(8) 2 cycles at $\theta = 0.04$ rad

Continue loading at increments of $\theta = 0.01$ rad, with two cycles of loading at each step.

- **Loading Sequence for Link-to-Column Connections S6.3 AISC 2005**

Qualifying cyclic tests of link-to-column moment connections in eccentrically braced frames shall be conducted by controlling the total link rotation angle, γ_{total}, imposed on the test specimen, as follows:

(1) 6 cycles at $\gamma_{total} = 0.00375$ rad
(2) 6 cycles at $\gamma_{total} = 0.005$ rad
(3) 6 cycles at $\gamma_{total} = 0.0075$ rad
(4) 6 cycles at $\gamma_{total} = 0.01$ rad
(5) 4 cycles at $\gamma_{total} = 0.015$ rad
(6) 4 cycles at $\gamma_{total} = 0.02$ rad
(7) 2 cycles at $\gamma_{total} = 0.03$ rad
(8) 1 cycle at $\gamma_{total} = 0.04$ rad
(9) 1 cycle at $\gamma_{total} = 0.05$ rad
(10) 1 cycle at $\gamma_{total} = 0.07$ rad
(11) 1 cycle at $\gamma_{total} = 0.09$ rad

Continue loading at increments of $\gamma_{total} = 0.02$ rad, with one cycle of loading at each step.

6.6.4.2 European protocols

In addition to the guidance of Annex D of EN 1990, specific rules need to be followed in the design assisted by testing of seismic components or devices. EN 15129 **(CEN 2009)** provides the functional requirements, general design rules, material characteristics, manufacturing and testing requirements, evaluation of conformity, installation and maintenance requirements of anti-seismic devices for use in structures erected in seismic areas in accordance with EC8.

In order to simulate the real dynamic conditions of a seismic event in a controlled laboratory environment with the objective of validating the use of a component or device using design assisted by testing methodologies, three main types of experimental testing may be carried out:

− Quasi-static monotonic and cyclic testing;
− Pseudo-dynamic testing;
− Dynamic testing.

The cyclic loading protocol requires important considerations because the damage is a cumulative process and is affected by the history of excursions (an excursion is the path from one peak loading value to the next loading peak value). The loading history should be defined such that the number of cycles experienced by the component at the onset of significant damage states is of the same order of magnitude as that experienced by real components in buildings subjected to strong earthquake motion **(Landolfo 2017)**.

Another protocol that can be applied to cyclic testing is the test protocol defined in EN 15129. This protocol states that, unless the Structural Engineer prescribes a different program, the test procedure shall include the steps listed below (Figures 6.16 and 6.17):

− Evaluation of the force-displacement cycle. Increasing amplitude cycles shall be imposed, at 25%, 50% and 100% of the maximum displacement, which shall be at least equal to ± d_{bd}, where d is the design displacement. Five cycles for each intermediate amplitude and at least ten cycles for the maximum amplitude shall be applied. If the fundamental period of the structural system in which the device has to be used is considerably less than 2s, a corresponding increase of the number of test cycles at ± d_{bd} shall be prescribed by the Structural Engineer;
- Ramp test for the static evaluation of the failure displacement. Deformations shall be applied at low speed. A displacement not less than d_{db} multiplied d by γ_b and γ_x or a force not less than V_{Ebd} multiplied by γ_b and γ_x, whichever is reached first, shall be imposed. d_{bd} is the design displacement, V is the force corresponding to d_{bd} obtained at the 3rd cycle during a quasi-static test, γ_b is the partial safety factor and γ_x is he reliability factor.

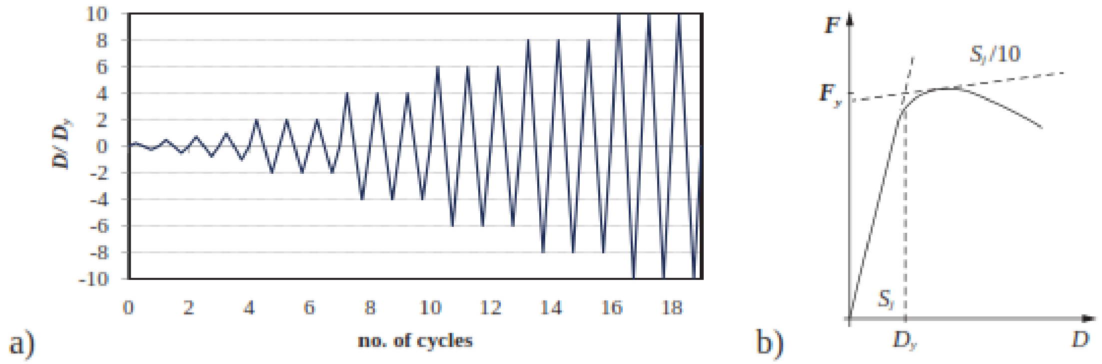


Figure 6. 16. Cyclic loading protocol (a) determination of yielding displacement **(Landolfo 2017)**.

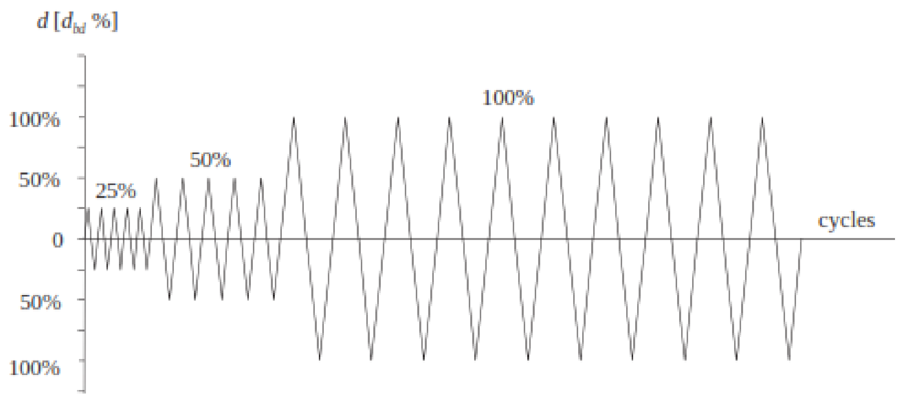


Figure 6. 17. Loading protocol specified in EN 15129 **(Landolfo 2017).**

6.6.5 General Requirements for a cyclic protocol (AISC 2005)

The test specimen shall be subjected to cyclic loads according to the requirements prescribed in section 6.2 for beam-to-column moment connections in special and intermediate moment frames, and according to the requirements prescribed in section 6.3 for link-to-column connections in eccentrically braced frames. Loading sequences other than those specified in sections 6.2 and 6.3 may be used when they are demonstrated to be of equivalent or greater severity.

The link loading protocol in Appendix S of the *AISC Seismic Provisions* was used for the study. This displacement-based protocol species three cycles each at 0.0025, 0.005, and 0.01 rad of total link rotation followed by two cycles at 0.01 rad increments up to failure. Total link rotation was defined as the imposed transverse displacement divided by the link length. Models were loaded up to a maximum of 0.10 rad total rotation.

CHAPTER 7:
NUMERICAL INVESTIGATION OF THE INELASTIC BEHAVIOUR OF SEISMIC LINKS CASES STUDIED

CHAPTER 7: NUMERICAL INVESTIGATION OF THE INELASTIC BEHAVIOUR OF SEISMIC LINKS CASES STUDIED

In addition, the very notable deficiency recommendations in current RPA provisions, even in its latest version, which are mainly represented by the absence of recommendations concerning eccentrically bracing systems (EBF), while universally recognized and used for long time, this will be fully discussed in the next part of the work.

This chapter deals with the numerical modelling of the inelastic behaviour of seismic links in EBF structures. It starts with an overview of the finite element analysis method than some information on ABAQUS software including its the features and capabilities. Then cases studied will be given and discussed. The first case treats the nonlinear behaviour of shear links where its data are taken from literature, with and extension studied cases proposed by the author. A part of this work has been the object of a conference paper in an international seminar.

In the second part of this work, in order to have a better understanding of the inelastic cyclic behaviour and parameters that influence that behaviour, a comprehensive finite element study is conducted by the means of the general-purpose finite element analysis software ABAQUS **(ABAQUS 2006)**. The second part, which has been published in AJCE 2019, which deals with a more complicated situation, taking into account the effect of cyclic loading, through a cyclic protocol, of long and short links designed in early chapter.

7.1 PROBLEMATIC

When subjected to severe earthquakes, steel structures are expected to undertake large inelastic deformations. As consequence, the real behaviour cannot be determined directly by an elastic analysis. Therefore, a nonlinear analysis procedure must be used for evaluation on the nonlinear and post-elastic behaviour.

Despite the fact that Eccentrically Braced Frames (EBFs) have been worldwide accepted and used as a seismic load resisting system, primarily in buildings, it is worth to point out that the EBF structures are not yet covered by the Algerian National Seismic Code (RPA99, 2003). The main motivation of the research work described herein is aimed to contribute to a better understanding of the performance of steel structures designed to RPA99 criterion in order to come out with some proposals to be integrated in the future version of RPA.

As detailed in the previous chapter, which represents the first part of the work. As in **(Labed 2014)**, in which the global elastic and inelastic behaviour of multi- stories steel X and inverted V concentrically structures designed to RPA99 provisions have been analysed. The assessment of the same structures has been modified in their configurations to become eccentrically steel structures EBFs by using EC8 **(EC8 2005)** recommendations and have been analysed their global linear and nonlinear behaviours. Also, as explained in Chapter 6 of this thesis, low and mid-rise steel structures having soft-storey reinforced with bracings including EBF configurations have been studied in both linear and nonlinear behaviours. The main conclusion of this comprehensive study is that for multi-storey steel structures being low, medium and high-rise and located in low and even medium seismicity regions, the design criteria of RPA for multi-storeys steel structures are efficient and can be safely used even for EBF structures, regular or irregular structures.

The second part of the undertaken research work in this thesis is the study of the seismic links which characterise EBFs structures, which will be detailed in this chapter and concerns the nonlinear and inelastic behaviour of seismic links which characterise EBFs.

Firstly, the author has analysed the nonlinear behaviour of isolated short seismic links, as very popular in practice, in EBF-D structures under static loadings taken from literature. Short links are proscribed, since they tend to undergo very high shearing deformations and develop very high and unpredictable forces. In this study which deals exclusively with shear or short links, and a particular analysis on the nonlinear behaviour taking into account the effect of the number of stiffeners their configuration and spacing on the nonlinear behaviour of the short link in EBF is carried out.

The main aim of this research is to examine the possibility of increasing the strength and stiffness of shear links from preventing the local stability to occur in the web with the contribution of different kind of stiffeners including diagonal ones. In fact, the investigation of nonlinear behaviour of this type of segment beams when changing the numbers, the location, the orientation and varying the thickness of intermediate stiffeners. To do so, three-dimensional numerical nonlinear investigation of the behaviour of short link equipped with vertical and diagonal stiffeners were implanted using the finite elements commercial package advanced computer program ABAQUS 6.13. Additional models, which are not included in **(Čaušević 2008)** are being modelled in the same manner to show the effect and the contribution of diagonal stiffeners on the nonlinear behaviour of seismic links. The models account for material and geometric nonlinearities.

The outcomes of this investigation show clearly the influence of stiffeners on the nonlinear behaviour of shear links. The results are compared to some experimental test results for the cases of transverse stiffeners available in the literature in terms of displacements and shear forces. The results of the numerical investigation are compared to some experimental test results available in the literature in terms of displacements and shear forces for the case of transverse stiffeners.

Secondly, as for steel frames which are likely to be subjected to cyclic loads under earthquake and other forms of dynamic loadings will require the study of cyclic and hysteretic behaviours. Full details on the behaviour of seismic links can be found in the corresponding chapter, i.e. chapters 4 and 5 in this thesis. For a better understanding of the inelastic cyclic behaviour and parameters that influence that behaviour, a comprehensive finite element study is conducted on long and short seismic links by the means of the general-purpose finite element analysis software ABAQUS **(ABAQUS 2006).**

The link beam shall contain a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame. Long links must be Class I sections as flexural hinging is expected at link ends, whereas short links may have Class 2 flanges, provided the web is Class I. Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Short links will yield in shear prior to flexural hinging at the link ends, whereas long links will yield in flexure before shear.

As stated earlier, the overall objective of the second part the research work undertaken in this chapter, more complicated by the nature of loading which is cyclic loading. The links were subjected to the AISC 2005 loading protocol **(AISC 2005)** with 32 cycles for long links and 35 cycles for short links. The overall objective of this research work was devoted to study the influence of key parameters on the inelastic behaviour of links, namely the cyclic loading behaviour of stiffened panels with varying stiffener rigidities.

Accordingly, more than twenty models have been implanted in ABAQUS. The model's data were taken and designed from structures studied in previous chapter. In fact, two IPE sections (IPE 360, IPE 450) have been selected from previous study designed on the weak beams vs. strong columns philosophy **(EC8 2005)**, and numerically modelled. All link segments were simulated as simply supported under constant monotonic cyclic loadings.

It's worth to point out that results presented and discussed herein in the related sections are depending on the applied loading protocols as well as the selected links boundary conditions and isotropic hardening. The long and shear links considered in this study were expected to behave in an inelastic manner and were reinforced with stiffeners with different configurations in order to improve their stiffness, plastic deformation capacity and resistance against any premature buckling.

Based on the finite element analysis, the results have shown, as in many recent research works, that the length of a link segment (*e*) along with the cross-section properties are the key parameters that controls the stiffness, strength, ductility. Substantial differences performances under inelastic behaviour were observed throughout this study of short and long links having the same cross-section. The apparition of buckling phenomenon, either in the flange or in the web or in both, is noticeable when the section was no more behaving in elastic range.

Conclusively, it is worth to recall that the links used in this study were parts of multi-storied structures were designed, in previous author's works, to RPA99 provisions, the authors are seriously thinking that the time has come to RPA in its future version, to include the EBF structures as seismic lateral resisting system and to adopt provisions of links from international seismic codes: EC8, AISC as their provisions are very close to each other's.

7.2 OVERVIEW OF THE FEA MODELLING OF LINKS USING ABAQUS SOFTWARE

This section deals with the use of numerical analyses, based on the finite element method, for the investigation of the inelastic behaviour of seismic links, particularly, under cyclic loading. General information on the finite element method is provided. Furthermore, in the second part of the chapter, the main characteristics of the leading code ABAQUS advanced computer program are briefly defined, focusing in particular on the features and capabilities used for the setup of the refined models of links.

7.2.1 Generalities on FEA

- **Introduction**

Broadly speaking, the Finite Element Method (FEM) and its practical application, often known as Finite Element Analysis (FEA) is a numerical technique for finding approximate solutions of partial differential equations as well as integral equations. FEM has been applied to a number of physical problems, where the governing differential equations are available. In addition, FEA often are the only way to get an answer in the case of particular problems, such as, for example, the mechanical behaviour of a system subjected to extreme loading conditions which are impossible to duplicate in an experiment **(ABAQUS, 2006).**

One of the key processes in finite element analysis is to build material models. When finite element models are built-up created, a variety of material models corresponding to different materials must adopted, because under the same geometry and loadings and boundary conditions, different materials may have unique behaviours **(Yang 2020).**

The method essentially consists of assuming the piecewise continuous function for the solution and obtaining the parameters of the functions in a manner that reduces the error in the solution. In general, a model may be defined as a simplified representation of reality. The degree of simplification depends on the problem peculiarities and on the particular aspects to investigate. In a macroscopic phenomenological perspective, the majority of the surrounding objects and systems as continuous **(Esposto 2007).**

- **FEA Software packages**

Realistic finite element problems might consist of up to hundreds of thousands, and even several millions, of elements and nodes, and therefore they are usually solved in practice using commercially available software packages.

There is currently a large number of commercially available for solving a wide range of problems: solid and structural mechanics, heat and mass transfer, fluid mechanics, acoustics and multi-physics, which might be static, dynamic, linear and nonlinear.

Most of these software packages use the finite element method, or are used in combination with other numerical methods. All these software packages are developed based on similar methodology with many detailed and fine-tuned techniques and schemes **(Liu 2013).**

In fact, FEA allow for:

- Identifying the forming problems prior to the tooling fabrication;
- Minimizing the tooling rework;
- Reducing the overall prototyping effort while identifying the possible shortcomings in the design;
- Minimizing the amount of material used during the fabrication process.

- **ABAQUS**

As the entire modelling of numerical inelastic analyses developed in this research are entirely carried out by means of the ABAQUS multi-purpose computer program **(ABAQUS 2004)**, based on the Finite Element Method. This program is chosen on the basis of its capability of reliably facing up complex problems which may be affected by strong non-linearities. Such peculiarity is fundamental in the modelling of seismic links, whose behaviour is noticeably non-linear. In ABAQUS a wide range of finite elements is available, in order to match the different requirements related to the physical problems of interest. In the following section a brief description will be made on ABAQUS as a suite of commercial finite element codes due to its strong capabilities in dealing with nonlinear problems.

7.2.2 Modelling with ABAQUS

7.2.2.1 Introduction

ABAQUS is one of the leading finite element packages and has much operational and verification experience to back it up, notwithstanding the quality of the pre- and post-processing capabilities. The ABAQUS finite element software is a suite of commercial finite element codes which has strong capabilities for solving, specifically, nonlinear problems and was developed by Hibbitt, Karlsson & Sorenson, Inc. Now ABAQUS is a registered trade mark of Dassault Systèmes.

The solution of a general problem by ABAQUS involves three stages: ABAQUS Pre-processor, ABAQUS Solver, and ABAQUS Postprocessor. ABAQUS/CAE or another suitable pre-processor provides a compatible input file to ABAQUS. ABAQUS/Standard or ABAQUS/Explicit based on implicit algorithm, is good for static, strongly nonlinear problems and can be used as ABAQUS/Solver. With the development of convenient user interfaces, most finite element software can be used as a ‘black box’, and is used by many users without proper knowledge of the FEM. Abaqus can be done either through Abaqus/CAE or CATIA, which are intuitive graphic user interfaces. They also allow monitoring and viewing of results. Data can be entered in or using an input file prepared with a text editor and executed through the command line, or using a script prepared with Python **(Rao 2017, Khennane 2013)**.

ABAQUS/ Explicit, based on explicit algorithm, is intended for dynamic problems. Both ABAQUS/ Standard and ABAQUS/Explicit can be executed under ABAQUS/CAE.

ABAQUS/CAE provides a complete ABAQUS environment that provides a simple, consistent interface for creating, submitting, monitoring, and evaluating results from ABAQUS/Standard and ABAQUS/Explicit simulations.

The ABAQUS/Viewer provides graphical displays of ABAQUS finite element models and results. It obtains the model and results information from the output database. We can control the output information displayed. For example, we can obtain plots such as undeformed shape, deformed shape, contours, x-y data, and time history

7.2.2.2 Features

In the following sections, a summary of the principal features of the used program and environment are illustrated. Such preliminary presentation is aimed at focusing on the program particularities, in order to create a clearly understandable basis for the choices made in the modelling of the highly nonlinear behaviour of seismic links, specially under cyclic loadings, presented in the following chapters.

In general, each finite element is characterized by five features, which are listed in the following:

Family: The element family is essentially related to the used geometry type which is essentially related to the used geometry type (beam, shell, solid, etc.). The most commonly used families in stress analyses are shown in Figure 7.1.

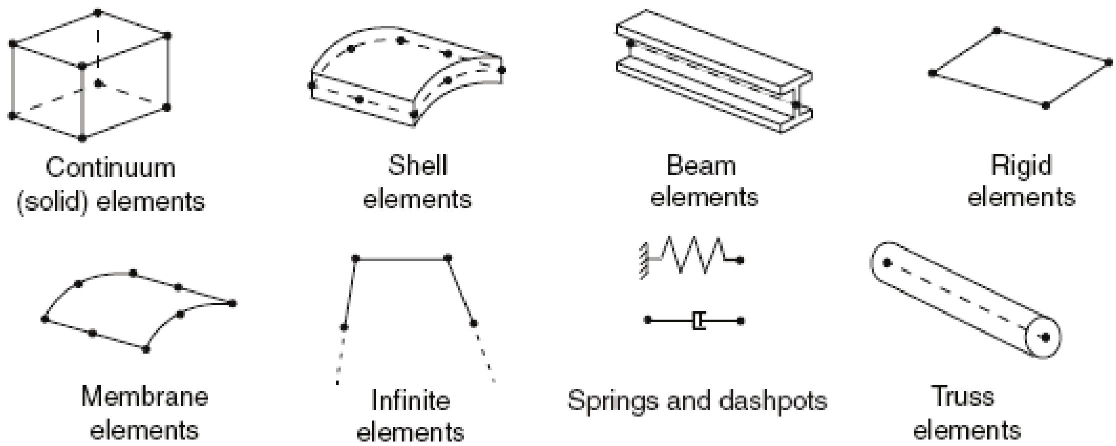


Figure 7. 1. Finite element families available in **ABAQUS (2004)**

ABAQUS does not have plate elements as such. Instead it uses shell elements. In ABAQUS, a plate is merely considered as a flat shell. A shell element can be considered as a sophisticated version of a plate element that can carry in-plane forces. ABAQUS offers two types of three-dimensional shell elements: conventional shell elements and continuum shell elements **(Khennane 2013)**.

Elements: Example of modelling using three-dimensional Shell Elements S8R5W, with the following terminology:

- S, conventional stress/displacement shell; SC, continuum stress/displacement shell; STRI, triangular stress/displacement thin shell; DS, heat transfer shell
- 8, number of nodes
- R, reduced integration (optional)
- 5, number of degrees of freedom per node (optional)
- W, warping considered in small-strain formulation

As reported by **(Khennane 2013)**, thick versus thin Conventional Shell Before choosing a shell element in Abaqus, it is worthwhile to check whether it is suitable for thin shells only, thick shells only, or both. The following elements are suitable for both: S3, S3R, S3RS, S4, S4R, S4RSW, SAX1, SAX2, SAX2T, SC6R, and SC8R. They include the transverse shear deformation, which becomes very small as the shell thickness decreases. The following elements S8R and S8RT are only for use in thick shell problems. Elements STRI3, S4R5, STRI65, S8R, S9R5, SAXA1n, and SAXA2n should not be used for thick shells where transverse shear deformation is important

Degrees of freedom (nodal displacements): The degrees of freedom are the discrete parameters constituting the fundamental unknowns of the problem.
Number of nodes: The number of nodes per element defines the order of interpolation used for defining the deformed shape of the element edges.
Formulation: is referred to the mathematical theory used for defining the element behaviour. Lagrangian or Eulerian formulations may be used.
Integration: The integration indicates the way different quantities are integrated over the volume of each element. Both full and reduced integration options are available, they being referred to the number of points required to integrate the polynomial terms in the stiffness matrix in an element.
Mesh: the mesh refinement of the surfaces involved in contact interactions derives from the rigid master-slave algorithm used in contact by ABAQUS/Standard, described in section 3.2.6 of ABAQUS 's GUIDE 2004, which implies that slave surfaces must be meshed in a finer way than the master ones. In particular, referring to the modelled contact interactions, contact plates have slave surfaces as respect to column flanges, PT bar has slave surface as respect to PT column hole and ED bars have slave surfaces as respect to the confining cylinders
Definition of the materials: ABAQUS provides a wide range of material types, which allow to cover problems involving metals, plastics, rubbers, foams and so on. With particular regard to ductile materials, such as steel, plasticity can be reliably caught, also accounting for hardening phenomena, and so the non-linearity due to the material characteristics can be well included in the models.
Non-linear problems: In general, in a non-linear analysis the solution cannot be calculated by solving a single system of equations, as would be done in a linear problem, and so the solution may be found by gradually and incrementally applying the specified loads, proceeding toward the final solution. Therefore, ABAQUS/Standard breaks the simulation into a number of loads increments and finds the approximate equilibrium configuration at the end of each increment,

by means of an iterative procedure. An iteration is here defined as an attempt to find an equilibrium solution. If the model is not in equilibrium at the end of the iteration, ABAQUS/Standard tries another iteration. At each iteration, the obtained solution should be closer to equilibrium, and sometimes the program may need many iterations to obtain a solution. For each iteration, ABAQUS/Standard forms the model's stiffness matrix and solves a system of equations. Consequently, in computational costs perspective, each iteration is equivalent to a complete linear analysis. The latter consideration underlines the large computational expense of a non-linear analysis in ABAQUS/Standard.

Increment: the increment is complete when an equilibrium solution is obtained. The sum of all of the incremental responses is the approximate solution for the non-linear analysis. Thus, ABAQUS/Standard combines incremental and iterative procedures for solving non-linear problems. The increment is complete when an equilibrium solution is obtained. The sum of all of the incremental responses is the approximate solution for the non-linear analysis. Thus, ABAQUS/Standard combines incremental and iterative procedures for solving non-linear problems **(Esposto 2008)**. The sum of all of the incremental responses is the approximate solution for the non-linear analysis. Thus, ABAQUS combines incremental and iterative procedures for solving non-linear problems. The user suggests the size of the first increment in each step of the simulation, and this can improve the control on the simulation convergence by the user, but the size of the load increments used for the solution of non-linear problems is automatically adjusted by ABAQUS **(Esposto 2008)**.

Convergence: the user has to suggest the size of the first increment in each step of the simulation, and this can improve the control on the simulation convergence by the user, who can indicate a small or large fraction of the increment size depending on the expected non-linearity at the beginning of the step. After each equilibrium iteration, equilibrium convergence checks are carried out. If convergence is achieved, the increment is completed; otherwise the iteration process has to restart **(ABAQUS 2004)**.

Contact: in general, contacts allow to model the behaviour of parts that can be in contact or not, depending on the system configuration, and they can take into account the friction properties between the surfaces in contact

Boundary conditions: in this module the definition of load and boundary conditions is made. Several different situations may occur depending on the purpose of the FE-simulation. Symmetry boundary conditions are imposed accordingly, so that the out-of-plane displacements of points belonging to the symmetry plane are prevented.

Loads: the size of the load increments used for the solution of non-linear problems is automatically adjusted by ABAQUS/Standard. The user has to suggest the size of the first increment in each step of the simulation, and this can improve the control on the simulation convergence by the user, who can indicate a small or large fraction of the increment size depending on the expected non-linearity at the beginning of the step. If ABAQUS cannot apply the load as a whole, it keeps reducing the increment until it reaches this minimum value.

7.2.2.3 Preparing a model using ABAQUS

Mesh generation is the process of dividing a physical domain into smaller subdomains (called elements) to facilitate an approximate solution of the governing ordinary or partial differential equation. Automatic mesh generation involves the sub division of a given domain, which may be in the form of a curve, surface, or solid throughout the case studies in this chapter. The analyses are carried out using ABAQUS/Standard finite element software (version 6.13).

The FEA with ABAQUS software is subdivided into three main steps:

- The pre-processing phase, that is the one in which the finite element model of the real problem is created;
- The simulation phase, that is the one in which the software solves the numerical problem defined in the model;
- The post-processing phase, that is the one in which analysis results are obtained.

The essential components used to describe the physical problems in ABAQUS are: the discretized geometry, the element section properties, the material data, the loads and boundary conditions, the analysis type, and the output requests that are defined into the pre-processing phase. There are other modules in the ABAQUS finite element package, including ABAQUS/Explicit, ABAQUS/CAE. The analyses presented in this work are carried out by means of the ABAQUS/Standard implicit solver, which uses the Newton-Raphson method to obtain solutions for non-linear problems.

- **Model Data in ABAQUS**

Some of the data that must be included in the model data are as follows, as described in **(Liu 2013):**

- Geometry of the model: The geometry of the model is described by its elements and nodes.
- Material definitions, which are usually associated with parts of the geometry.

Other optional data in the model data section are:

- Parts and an assembly: the geometry can be divided into parts, which are positioned relative to one another in an assembly.
- Initial conditions: non-zero initial conditions such as initial stresses, temperatures or velocities can be specified.
- Boundary conditions: zero-valued boundary conditions (including symmetry conditions) can be imposed on the model.
- Constraints: linear constraint equations or multi-point constraints can be defined.
- Contact interactions: contact conditions between surfaces or parts can be defined.
- Amplitude definitions: amplitude curves for which the loads or boundary conditions are to follow can be defined.
- Output control: options for controlling model definition output to the data file can be included.
- Environment properties: environment properties such as the attributes of a fluid.

- Surrounding the model can be defined.
- User subroutines: user-defined subroutines, which allow the user to customize.
- ABAQUS, can be defined.
- Analysis continuation: it is possible to write restart data or to use the results from a previous analysis and continue the analysis with new model or history data.

7.3 LINKS MODELLING PROCEDURE

7.3.1 General

In this research work including the assessment the nonlinear behaviour of isolated seismic links the actual geometrical and mechanical features of the structural system can be reproduced in detail by means of ABAQUS software, so that the behaviour of any component parts can be investigated. ABAQUS has been used as it has ability to consider both geometric and material nonlinearities in a given model. Large displacement effects were accounted for by utilizing the nonlinear geometry option in ABAQUS.

The first part of the present work concerns the nonlinear behaviour of shear links where the data are taken from literature. The second part, deals with the inelastic monotonic cyclic behaviours of a series of isolated seismic links ranging from long to short links.

7.3.2 Developed models

Once again, ABAQUS program is chosen on the basis of its capability of reliably facing up complex problems which may be affected by strong non-linearities. The finite element mesh of the models is always composed by continuum tri-dimensional linear elements with reduced integration (C3D8R) is made by tri-dimensional continuum first-order elements with reduced integration (C3D8R) for all the component parts of the model. They are 8-nodes hexahedral elements with one integration point at the centre, which usually give the best results for the minimum cost. In addition, their features are practically essential in the case of simulations involving both material plasticity and contact interactions, as described in section 7.1.3.

As in a non-linear analysis the solution cannot be calculated by solving a single system of equations, as would be done in a linear problem, and so the solution may be found by gradually and incrementally applying the specified loads, proceeding toward the final solution. Therefore, the simulation is subdivided into a number of load increments and the approximate equilibrium configuration at the end of each increment is found, by means of an iterative procedure.

7.4 CASES STUDIED OF THE NONLINEAR BEHAVIOUR OF SHEAR (SHORT) SEISMIC LINKS

7.4.1 General considerations

As it was fully detailed in previous chapters (i.e. 3, 4 and 5), the seismic links are designed and expected to undertake large nonlinear deformations by an excellent dissipation of energy without losing resistance as it is stated in several seismic code provisions such as **(EC8 2005), (AISC 2010) and (UBC 1997).** Indeed, EBFs can be designed to balance stiffness and ductility, by maintaining the frame stiffness similar to that of CBFs, while selecting a reasonable link length to efficiently dissipate seismic energy primarily by inelastic action in the links**.**

Generally speaking, the seismic links are of three kinds: short, medium and long links. An investigation of the inelastic behaviour of isolated short links, various types of links are considered including transverse and diagonal stiffeners using a nonlinear inelastic three-dimensional finite element model. A particular numerical analysis on the non-linear behaviour of shear (short) link associated with the effect of the number of stiffeners their configuration and spacing on the is being carried out using ABAQUS. The study is based on a preliminary numerical model of an EBF with basic geometrical configuration found in the literature shown in Figure 7.2 **(Hjelmstad 1984).**

7.4.2 Configurations of the models

The major purpose of placing stiffeners in the seismic link is to preserve their local web buckling, i.e. to achieve a full yielding of the cross section by shear and the possibility of increasing the strength of the web.

The geometry of the section of each model corresponds to the centreline dimensions of the modelled isolated short link beams as per EC8.

The stiffeners should have a combined width of not less than (bf – 2tw) and a thickness not less than $0.75t_w$ nor 10 mm, whichever is larger. Also, links should be provided with intermediate web stiffeners. Short links should be provided with intermediate web stiffeners spaced at intervals not exceeding (30 t – h/5) for a link θ_p rotation angle of 0.08 radians or ($52t_w$– /5) for link rotation angles θ_p of 0.02 radians or less. Linear interpolation should be used for values of θ_p between 0.08 and 0.02 radians. The web local buckling will be prevented by adding number of transverse stiffeners along the web of the link. Fillet welds connecting a link stiffener to the link web should have a design strength adequate to resist a force of γ_{0v} f_y A_{st}, where A_{st} is the area of the stiffener. The design strength of fillet welds fastening the V_y of stiffener to the flanges should be adequate to resist a force of γ_{0v} f_y A_{st}.

where

b_f flange's width

t_w web's thickness

h total height of the section

θ_p plastic rotation capacity

7.4.3 Objectives

In this first investigation work, the effect of the web stiffeners number, configuration and spacing on the nonlinear behaviour of short seismic link in EBF-D structures will be presented. Understanding strength, stiffness and ductility properties of EBFs requires studying the nonlinear behaviour of links. For the case of short links, which are the subject of the investigation carried out in this section, some details will be given.

7.4.4 Shear links

The use of short link beams in (EBFs) is generally preferred to long ones, mainly due to their high rotation capacity and energy dissipation under cyclic loadings. Their primary function is to dissipate earthquake energy through large inelastic deformations.

The use of stiffeners used in the seismic link is intended to preserve from the local web buckling to happen, i.e. to achieve a full yielding of the cross section by shear and the possibility of increasing the strength of the web. In this study, and in order to investigate the inelastic behaviour of isolated short links, various types of links are considered including transverse and diagonal stiffeners using a nonlinear inelastic three-dimensional finite element model.

7.4.5 Design of the studied cases of seismic links

When properly designed and detailed EBFs behave in a ductile manner through shear or flexural yielding of a link element. According to the well-developed seismic codes **(EC8 2004), (AISC 2010)** and **(UBC 1997)** and other international seismic codes, an eccentric braced frame requires at least one end of every brace to frame into a link. However, it should be noted that these kinds of structures are not yet covered by the Algerian Seismic code RPA **(RPA99 2003).** Although the American and European seismic codes share several theories on the design of seismic links having the same limits, their approaches to the problem is rather different. In this work the European approach is used. As far as the sizing of links including the stiffeners is concerned.

7.4.5.1 Recall of shear links properties

In the following, a brief recall of the properties of seismic segments will be given. As several times mentioned and explained in previous chapters, the purpose of the link in EBF structures is to transfer the shear and bending forces on the beam to the bracing strut as axial force. As such, the link of an EBF experiences three forces: shear, axial, and flexural. Axial forces have been shown to be negligible for cases where link required axial strength, is marginal compared to nominal axial yield strength.

Generally, designers prefer the use of short link beams in (EBFs) to long ones, mainly due to their high rotation capacity and energy dissipation under cyclic loadings. The inelastic behaviour will become more significant as a result of shear forces when the link is getting shorter. The first stage in mathematical modelling involves the representation of the entire structure by elements to which physical and material properties can be assigned.

- **Basic behaviour of shear links**

The simplest eccentrically braced frame is shown in the Figure 7.2. The seismic link is characterized by its length e and of the geometries of the various configurations of the EBFs. The link of an EBF experiences three forces: shear, axial, and flexural (see Figure 7.3). Axial forces have been shown to be negligible for cases where link required axial strength, is marginal compared to nominal axial yield strength. Depending on the length of the link, either shear or flexural forces will dominate failure behaviour.

The link length for this analysis was chosen in a way to fulfil the requirements of EC8 and AISC for both short links:

$e < e_s = 1.6\ M_{p,link}\ /V_{p,link}$ (7.1)

And the distance between link stiffeners

$a_{com} = 30t_w - d/5$ (7.2)

Having this in mind the following value for short link length was used in this analysis:

$e_s = 1.1\ M_{p,link}\ /V_{p,link} = 300mm$ (7.3)

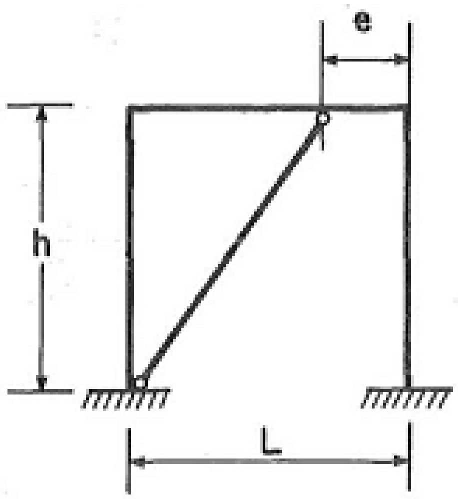


Figure 7. 2. Simplest eccentrically braced frame **(Hjelmstad 1984)**

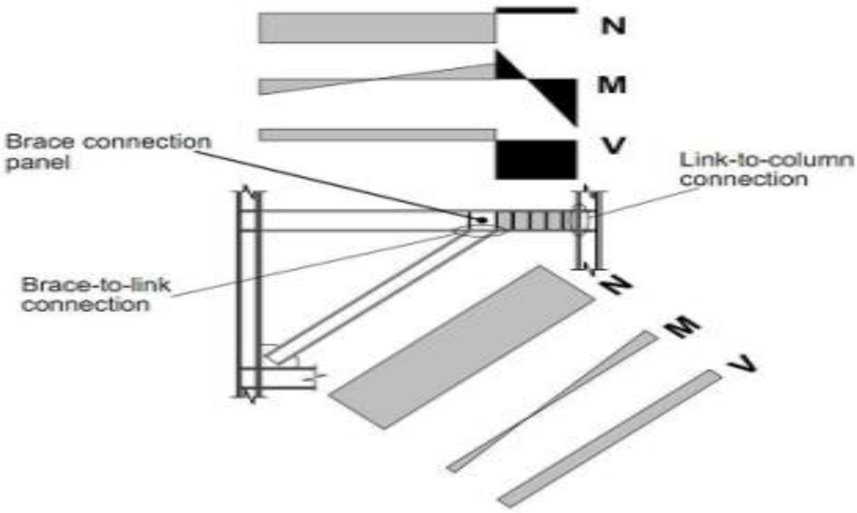


Figure 7. 3. Typical force distribution in an EBF (**Okazaki 2004).**

In order to increase the performance in terms of strength and stiffness, and by the same way, to prevent local web buckling of link element of shear link an attempt to achieve this goal is made through the installation of additional diagonal web stiffener on web to transverse ones.

By installing diagonal stiffener, a change in surface properties may occur. Also, the installation can prevent local buckling on link element. These changes include increase of inertia moment and shear surface.

It must be recalled that the normative requirement from the Eurocode 8 offers a method of establishing the intervals at which the intermediate stiffeners should be disposed along the link length, see chapter 5 for details.

7.4.5.2 Cases considered

In this analysis, stiffener material properties are taken similar to link web and flange material properties. The proposed configuration of transverse stiffeners is practical to construct. The chosen specimen of short links data was taken from previous study, as shown in Figure 7.4 **(Čaušević 2008).**

Note that the terminology used is similar to that of **(Čaušević 2008).**

Four basic specimens with an equal cross-section (HEA100) and equal length of e_s = 300 mm were selected and aac representing the actual distance between transverse stiffeners, depending on their numbers.

- Specimen of seismic link without stiffeners (a_{ac0} = 300*mm*)
- Specimen of seismic link with one couple of stiffeners (a_{ac1} = 150*mm*)
- Specimen of seismic link with two couples of stiffeners (a_{ac2} = 100*mm*)
- Specimen of seismic link with three couples of stiffeners (a_{ac3} = 75*mm*)

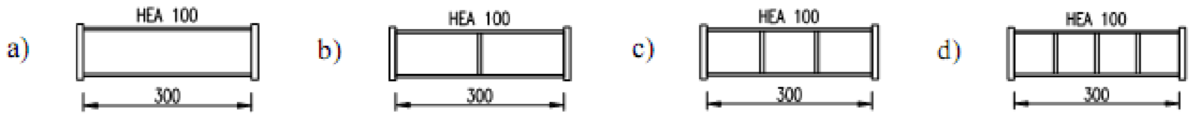


Figure 7. 4. Geometrical characteristics details of specimens **(Čaušević 2008)**

Web stiffeners are 5 mm and 10 mm thick plates are placed bilaterally. Boundary conditions for links are defined in a way as to simulate boundary conditions in the frame.

In this analysis, additional models were considered with the same geometrical dimensions equipped with diagonal stiffeners. For cases where diagonal stiffeners are used, the same geometric and material proprieties have been used as it is the case for the vertical stiffeners.

7.4.6 Characteristics of FEA models

To investigate the nonlinear behaviour of short beams (link), numerical models based on 3D are developed and implanted in the finite element software ABAQUS. These models are, in first time consisting of series of four isolated links belonging to shear links kind taken from previous research work, then other models selected by the author considering the effect of diagonal stiffeners on the nonlinear behaviour of shear (short) links. In this study, the whole data of numerical models are extracted from **(Čaušević 2008)** and the nonlinear behaviour of isolated seismic links is investigated.

The first stage concerns the development of the mathematical modelling which involves the representation of the entire structure by elements to which physical and material properties can be assigned. Numerical models based on 3D are developed using the finite element software ABAQUS were performed by considering several short segment beams (link) with the general considerations explained in the previous section.

- **General considerations**

- **Geometrical characteristics**: Four models of short seismic links were done, each having the same cross section (HEA100) and the same length (300 mm) but with different number of web stiffeners, i.e. without any stiffener on the seismic link length, with one, two and three couples of web stiffeners, with additional same shear links having diagonal and combined web-stiffeners.
- **Boundary conditions**: boundary conditions are the same as proposed by **(Richards and 2005)** which allow axial deformation of the link while preventing rotation at both ends.
- **Stiffeners**: Considering the effect of the number, the location and the configuration of web-stiffeners influence of stiffeners on the inelastic behaviour of short links was analysed by creating three different configurations of web stiffening: no stiffeners, one stiffener in the middle of the link and a third category with 2 stiffeners models) were evaluated by considering P-Δ (shear-force–link-displacements) response curves. In the case of the link model with 750

mm length an additional model with 3 stiffeners was considered. Two stiffener thickness values are used: t_s = 5 and 10 mm respectively.

- **Material**: The models account for material nonlinearities. The material is introduced as isotropic with elastic plastic strain hardening behaviour with Von-Mises criteria and Tresca yield criteria through large displacement for the illustration of local buckling and post buckling strength is octahedral shear stress theory, and maximum distortion energy theory, states that failure at a particular location occurs when the energy of distortion reaches the same energy for failure in tension. The plastic hardening was defined using a nonlinear kinematic hardening law, see Chapter 4.
- **Loading** Transverse loading is applied as vertical incremental prescribed displacement up to 40 mm as specified in **(Čaušević 2008)** to the right end nodes. By adding prescribed displacements on one end of the link, that is, by increasing the shear force, the plastification of seismic link web occurs. While the modelling of the active links, fixed support is assigned one end of the link and at the other end displacement is applied according to the test procedure **(Richards 2005)**.
- **Other parameters:** such as thickness of flange and thickness of web are kept constant in this analysis because these parameters cannot be modified in the field due to the applied HEA profile which is default from the factory. The nonlinear material behaviour was modelled using Von Mises and Tresca yield criterion.

Additional models, which are not included in **(Čaušević 2008)** examining the contribution of diagonal stiffeners in the nonlinear behaviour of seismic links are being modelled in the same manner to show the effect

All models account for material and geometric nonlinearities. Stress and strain response for the material was taken to be elements to which physical and material properties can be assigned.

- Actual Input

The global model contains approximately 80 000 solid elements. The boundary conditions used in the modelling prevent rotation at both ends of the link. On one side of the link all six conditions of freedom were prevented, while on the other side five conditions of freedom were prevented and displacement on the vertical axis was allowed. The seismic link must be free of any supports. Such supports can prevent occurrence of inelastic deformations in the link, which can lead to deformation of other system elements **(Kasai 1986b).**

Geometrical input used in the modelling

Specimens with transverse stiffeners as specified in (Čaušević 2008):

Hot-rolled HEA100: b = 100 mm; t_f = 8 mm; h_w =70 mm; t_w= 5mm.

Stiffeners: t_s = 5 mm and 10 mm.

Transverse stiffener locations: mid-span, third-span and quarter-span.

Length = 300 mm for all specimens

It must be reminded that parameters, such as thickness of flange and thickness of web are kept constant in this analysis because these parameters cannot be modified for HEA profile which is default from the factory.

Specimens with diagonal stiffeners: same geometrical properties with same positions with same thicknesses.

- **Mechanical properties input of steel used in the modelling (ABAQUS)**

The material is introduced as isotropic with elastic plastic strain hardening behaviour with Von-Mises criteria and Teresa's yield criteria through large displacement for the illustration of local buckling and post buckling strength. For all specimens including those with diagonal stiffeners.

Density, ρ = 7850 kg/m^3; f_y = 235 N/mm²; ε = 0; f_u = 321.74 N/mm²; ε = 0.3.
Young Module's E = 210000 N/mm²; Poisson's ratio ν = 0.3.
Type element used in meshing: 3D solid.

- **Meshing**

A shell is a three-dimensional body bounded by two curved surfaces. Meshing is defined as the process of representing a physical domain with finite elements creating a set of elements known as the finite element mesh. The applied Solid 3D elements, although much more complicated to model than plane elements, give more accurate results and are recommended for scientific numerical simulations of steel systems. Samples of complete models are shown in Figure 7.5.

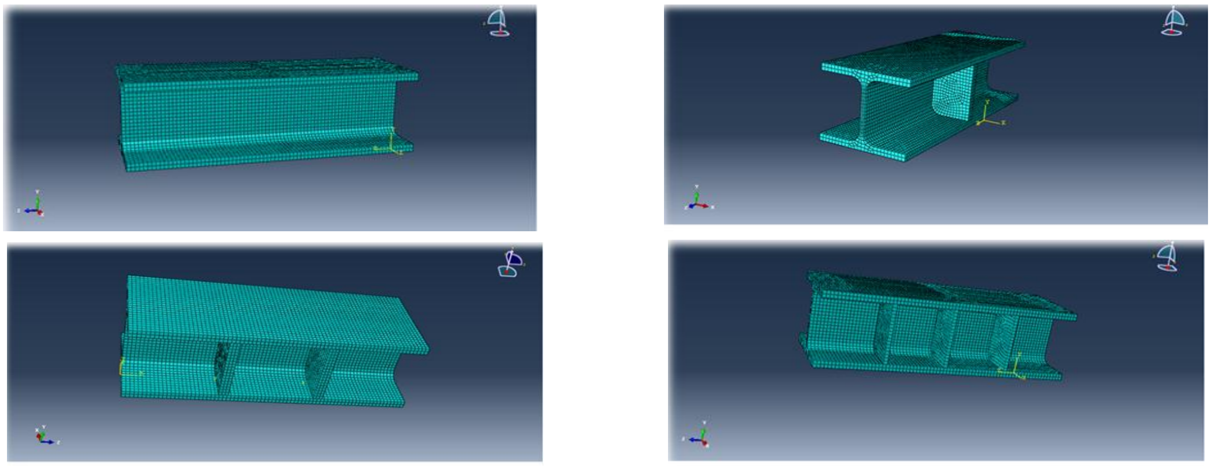


Figure 7. 5. Finite element mesh of the complete model with the basic geometrical layout of the specimens without and with stiffeners.

- **Boundary conditions**

In this particular investigation, the boundary conditions are the same as proposed by **(Richards and Utang 2005)** in the case of wide-flange links were used. These boundary conditions allow for axial deformation of the link while preventing rotation at both ends. The seismic link must be free of any supports. Such supports can prevent occurrence of inelastic deformations in the link, which can lead to deformation of other system elements **(Engelhard 1992)**.

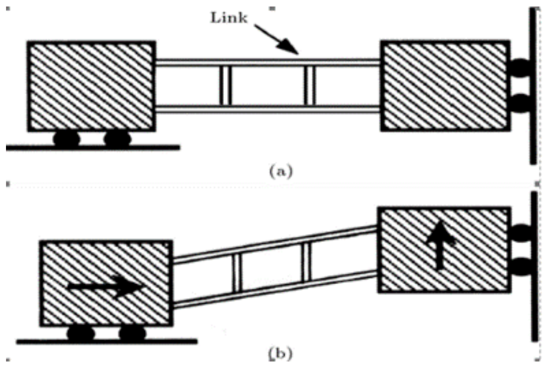


Figure 7. 6. Modelling boundary conditions according to **(Richards and Uang 2005)**.

7.4.7 Results and Discussion

7.4.7.1 Introduction

First of all, and before holding a discussion on the obtained results, it is worth to mention that the quality of the results obtained from FEA depends up on the density of the mesh, element type and the input properties of the element and these modelling aspects usually increases as the structural system become complex. The results become more accurate when the modelling shifts from one dimension to three. Some preliminary analyses were conducted to study the effect of mesh refinement, and to determine whether reduced integration elements could be used to improve computational time without loss of significant accuracy. Thus, most of the codes are presented without explanation, assuming the reader can input them properly.

Previous research studies show that the behaviour of shear links is quite complicated and affected by different parameters, and as a result significant amount of research interest has been directed towards both experimental and numerical determination of the nonlinear behaviour and cyclic energy dissipation characteristics especially for shear-links **(Hjelmstad 1983b; Roeder 1977; Malley 1984; Hjelmstad 1984; Engelhard 1992).** Following extensive research in the 1970s, 80s, and 90s of the twentieth's century, EBFs have become a widely accepted form of seismic force resisting system. Early research studies on EBF were conducted in attempt to solve problems faced in CBFs and MRFs, by Popov and his associates. Several investigators have studied web fracture in shear links using nonlinear finite element analyses as mention in the previous section **(Okazaki 2004).** Several investigators have studied web fracture in shear links using nonlinear finite element analyses including **(Okazaki 2004).**

7.4.7.2 Results

Several three-dimensional finite-element models using software package ABAQUS were developed for the nonlinear analysis of short links in order to investigate their performance. The nonlinearity of actual materials was introduced into the numerical model. Also, the nonlinear kinematic hardening plasticity material model available in ABAQUS was used in the finite element model.

It is worth to recall that the finite element models do not predict material failure and fracture, which generally occur in laboratory tests, causing loss of strength and consequently low rotation capacity.

- **Result presentations**

Results of the present investigation of nonlinear finite element analysis will be first presented and discussed in the terms of loading curve history that is shear forces vs. prescribed displacement curves as they better retrace the loading history for different configurations of shear links with and without stiffeners. The second part of the discussion will be devoted to the stress analysis by mean of the two available failure criteria implanted in ABAQUS: Von Mises and Tresca yielding criterion. It must be noticed that the plasticity model used in the analyses was based on a von Mises yield surface, which is octahedral shear stress yield criterion, also often called either the von Mises or the distortion energy criterion, represents an alternative to the maximum shear criterion and an associated flow rule. Failure criteria will be considered on the basis of values of stress and their application involves calculating an effective value of stress that characterizes the combined stresses, and then this value is compared to the yield or fracture strength of the material.

Broadly speaking, and as expected, it can be clearly seen that all load-deflection curves show two distinct branches: linear elastic and curved branches in the shear links considered. The parametric analyses in terms of the number of placed stiffeners shows the influence of the latter factor. The efficiency of the stiffeners, as they are usually required to control buckling effects, from shear stresses in steel members to ensure that the web panel is able to develop its shear strength capacity and shear buckling resistance, is demonstrated when the shear links behave in the post-elastic range, through an increase of the strength and stiffness capacities.

- **Loads-displacements relationships**

Results of the present study showing the effect of stiffeners in terms of thicknesses, configurations, numbers and orientations are presented in Figures 7.7 to 7.12. As can be from all depicted curves in Figures 7.7 to 7.12, show the respective relationships between Loads vs. Displacements through two curve branches: linear and nonlinear relationship, representing the linear elastic and inelastic behaviours of different models. The ultimate applied load values vs. corresponding displacement vary naturally with the general configuration of the studied models.

Thus, figures 7.7 to 7.12 are depicted in the following order:

- Case with transverse stiffeners

For Figures 7.7 to 7.9, load-deflection are depicted for models having the same features and with different number of stiffeners with t_f = 5 and 10 mm, respectively.

For Figures 7.10 and 7.11, display the results of added models made by the author not covered in the original work, concerning the effect of orientation of stiffeners with t_f = 10 mm

Figure 7.12 displays an overview comparison between all obtained results showed in 7.7 to 7.11 concerning stiffeners with tf = 10 mm.

In Figures 7.7 (a) and (b) the nonlinear Load-displacements curves are plotted for short links having parallel transverse web-stiffeners thickness with t_f = 5 mm and 10 mm respectively, representing the basic case for shear link equipped with stiffeners, which shows clearly the effect of the thickness of stiffeners on the increase of the ultimate load capacity for a single central transverse with a rise. The rise of the carrying capacity is roughly 10%.

Similar remarks can be made for the other cases, plotted in Figures 7.8 and 7.9, by changing configurations with an increasing of the number of transverse stiffeners, are modelled in terms of the thickness. The increase of the carrying capacity is ranging between 10% and 18%, which highlights the effect of stiffeners number.

Also, Figures 7.7 to 7.9 another geometrical parameter has been picked out which is stiffener's thickness 5 and 10 mm (a) and (b) respectively. Curves highlight the undeniable positive effect of this parameter along with the number, location and the configuration of stiffeners as they improve the carrying capacity of links by an additional the stiffness and strength in the inelastic response of shear links.

Figures 7.7 to 7.9, display the comparison of the obtained results of the three cases, namely: without any stiffener, being the reference case, and with one, two and three couples of transverse stiffeners. Also, it can be noticed that regardless the installation or not of transverse stiffeners their configurations, the behaviour of the seismic shear links in the linear branch of curves are all alike, and hence show sensibly identical behaviour in the elastic behaviour.

From the previous observations made on the effect of thickness on the global nonlinear behaviour of shear links, it has been decided to choose t_f = 10 mm for the remaining modelling, that is with diagonal stiffeners.

In Figure 7.9, of a fully equipped (maximum stiffeners number) shear link, link stiffness, defined as the secant stiffness is the ratio between the maximum lateral force and displacement at each stage of loading (load step)of and as far as the stiffness of isolated shear links is concerned, can be seems to not affect by the increase in stiffeners disposed on the link regardless their positions, number or even their configurations in the elastic range, as expected for a seismic loading. However, as found in other research works, when the shear links behave in inelastic manner this stiffness rises as the stiffener number arises.

As a summary, it was found, as discussed earlier, that adding transverse stiffeners, the shear- force capacity of the link increases for links with three couples of stiffeners. Certainly, as far as strength of the isolated links is concerned, which can be defined as the maximum lateral force (shear force) that can be undertaken by the link in each stage of loading, the Load vs. displacement curves in the corresponding Figures shows that case 4 (shear link with three couples of vertical stiffener of 10 mm thick) has obtained the highest strength in a lateral withstanding tension and compression compared to the cases 1, 2 and 3 respectively. This carrying capacity is 83 kN which represents about 33% more with t_s = 10 mm compared to the unreinforced basic case for which the yield force was 63 kN with 13% for the case of 5 mm stiffener compared the referential case. These results show clearly the impact of stiffeners thickness ton the resistance and stiffness of links with higher values for stiffeners having a thickness equal to 10 mm, as can be noticed from Figure 7.7(a) and (b) to 7.9(a) and (b). It was found that using intermediate stiffeners, the effect of web-stiffeners and their thicknesses and also their configuration are more pronounced, in the nonlinear branch curves, starting the yield point up to the ultimate load, as the strength capacities of links strengthen with transverse stiffeners, the shearing force capacity of the link increases for links with three couples of stiffeners to reach

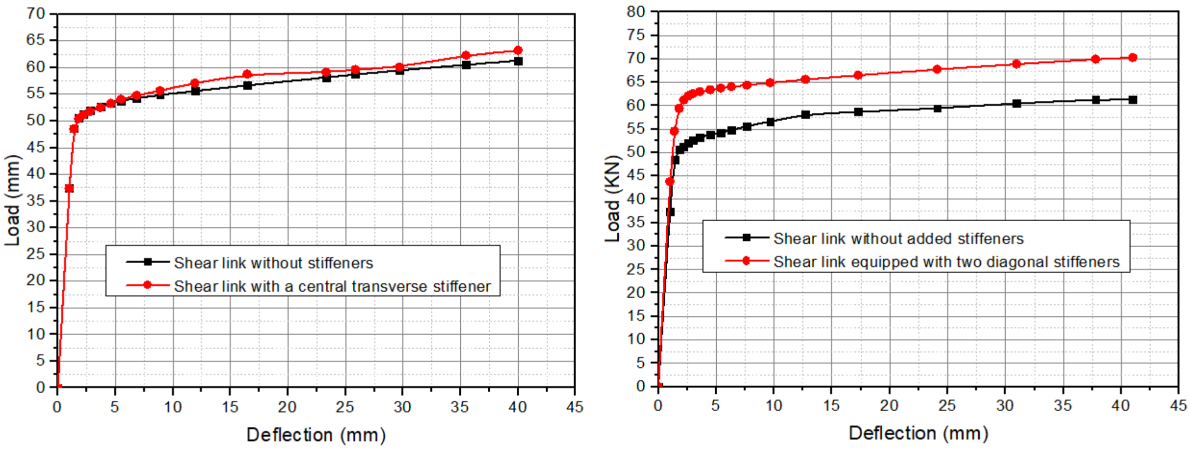


Figure 7. 7. Load vs. Displacement curves for links without and with one central stiffener (a) t_s = 5mm (b) t_s = 10 mm

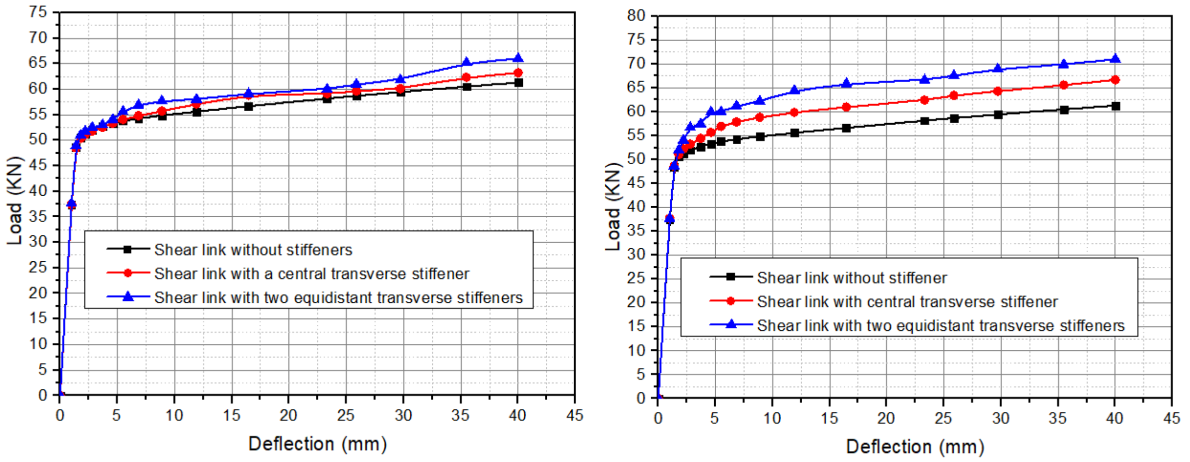


Figure 7. 8. Load vs. Displacement curves for links without and with one central and two equidistant stiffeners (a) t_s = 5mm (b) t_s = 10 mm

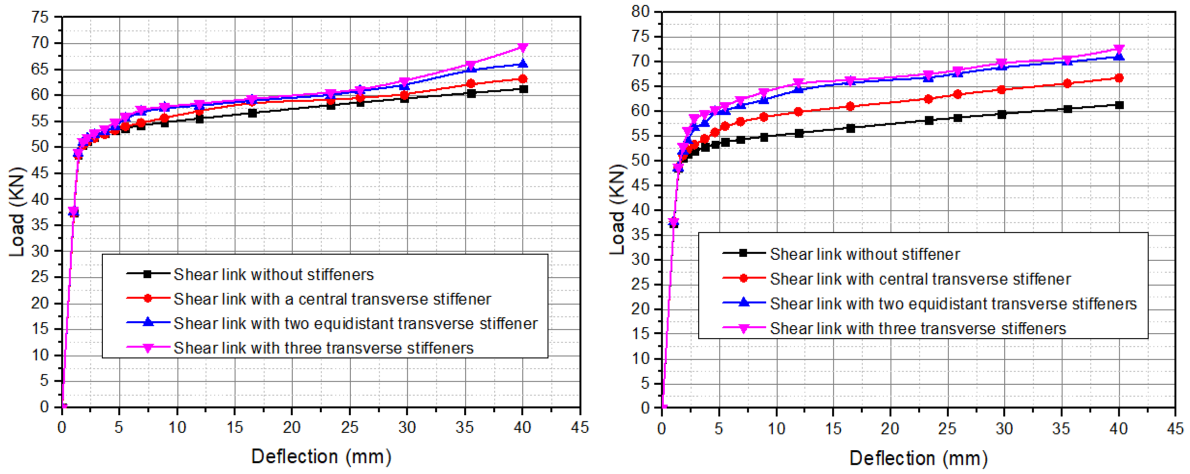


Figure 7. 9. Load vs. Displacement curves for links without and with one central and two and three equidistant stiffeners (a) t_s = 5mm (b) t_s = 10 mm.

- **Cases with combined stiffeners**

It is worthily to remind that the models treated in this section do not be covered in **(Čaušević 2008).**

Figure 7.10 deals with the cases of diagonal web-stiffeners and combined transverse - diagonal stiffeners. Figure 7.10(a) depictes the effect of two symmetrical diagonal stiffeners, being the reference case. Compared to Figure 7.7(b), one can notice that there is a small improvement of the carrying capacity of the shear link. When a combined configuration is being used as it is represented in Figure 7.10(b), better results are obtained, it was observed. An increase in the carrying capacity of slightly more 20%.

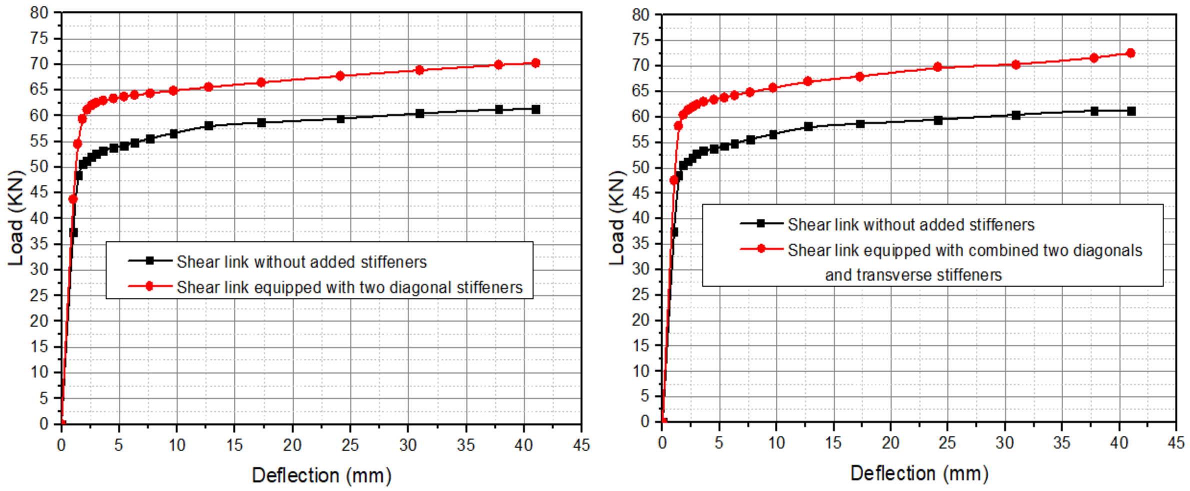


Figure 7. 10. Load vs. Displacement curves for links without and two diagonals (a) t_s = 5mm (b) t_s = 10 mm

As the number combined diagonals and transverse stiffeners increases, see Figure 7.11, further rise in the carrying capacity can be observed with the maximum for the case of four diagonals and three transverse stiffeners of more than 30%.

Figure 7.11 gives the overall performance of shear links having diagonal and combined transverse-diagonal stiffeners and clearly shows the effect of the orientation of web-stiffeners, by an increase in the strength, on the global inelastic behaviour.

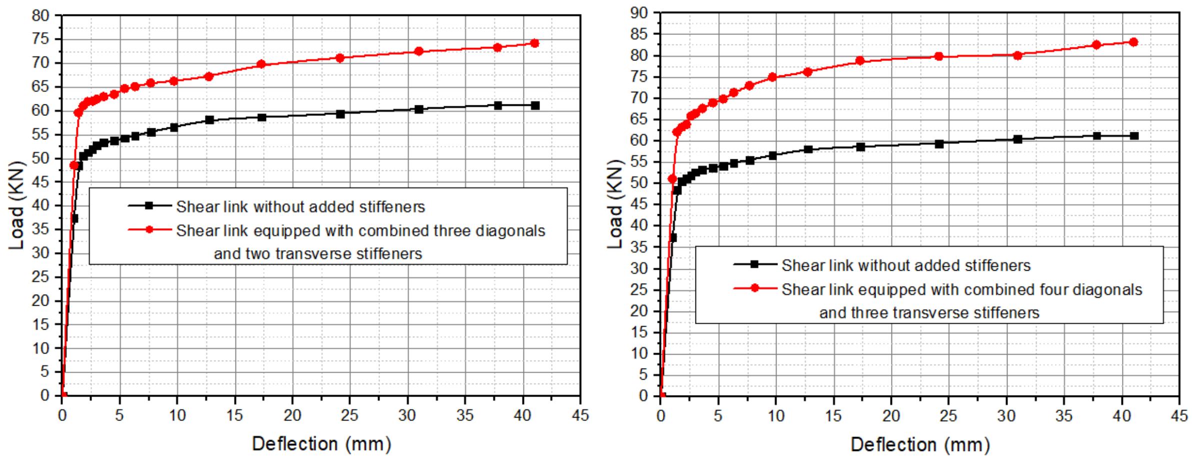


Figure 7. 11. Load vs. Displacement curves for links without and with combined three diagonals transverse and two transverse stiffeners (a) and (b) for four diagonals and three transverse stiffeners.

In Figure 7.12, an overall plot for all cases modelled having t_f = 10 mm is given. Additional investigation has been carried out to demonstrate, for the same specimens, the effect of diagonals stiffeners on the nonlinear behaviour of shear links. Figure 7.12 shows the effect of adding diagonal stiffeners to the transverse ones does manifest by an increase in the bearing capacity of the link around 20% increase.

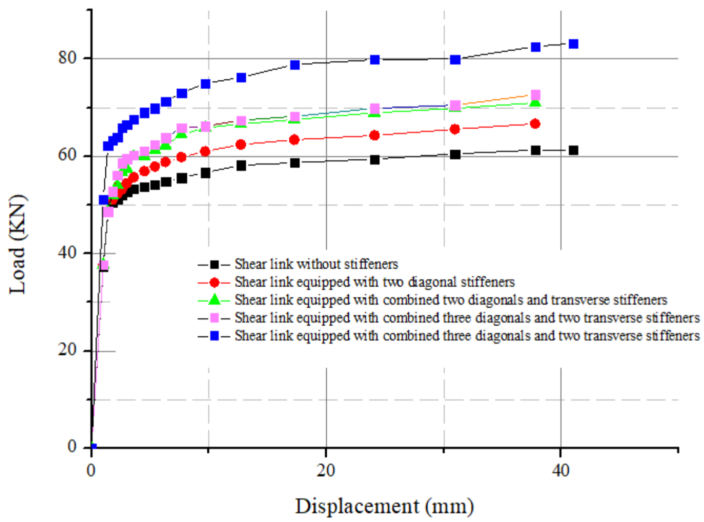


Figure 7. 12. Load vs. Displacement curves performance of shear link with different stiffeners configurations (t_s=10 mm).

Once again, it can be seen from Figure 7.12, that regardless the configuration of the shear link, all studied cases show sensibly identical behaviour in the elastic stage. However, as it can be noticed, when the link behaves in nonlinear manner, the effect of web-stiffeners and its thickness are more pronounced as the strength capacities of links strengthen with transverse stiffeners, the shearing force capacity of the link increases for links with three couples of stiffeners to reach 73 kN, for this particular case, which represents about 23% more when $t_s = 10$ mm compared to the without stiffeners for which the yield force was 63 kN which decrease to 13% for the case of 5 mm thickness. Changing parameters such as thickness has some significant effect on link performance. Increasing the thickness from 5 mm to 10 mm of the web stiffeners gives the link slightly more stiffness and additional strength and decrease the risk of premature web local buckling.

The comparison with the experimental results given in **(Čaušević 2008)** shows that similar curve shapes are obtained especially in elastic zone; the yielding forces are similar for the case of vertical stiffeners. However, for the post-elastic range, the slope of the curves is quite different with highest slope for the experimental results compared to numerical results obtained in this study. This is due essentially to the lack of information about tests conditions and also the real material characteristics used in the experimental works and can be attributed the true stress-strain curve used by **(Čaušević 2008).**

- **Stress and displacement contours**

  - **Preliminary on yield criterion used in ABAQUS**

The second part of discussion concerns the outcomes of plasticity theory deals dealing with yielding of materials under complex stress states. The concepts of effective stress and effective strain are necessary for analysing the strain hardening that occurs on loading paths other than uniaxial tension. The equivalent to assuming that plastic deformation causes no volume change **(Hosford 2005).**

There are two yield failure criteria implanted in ABAQUS: the maximum shear stress criterion (Tresca) and the maximum distortion energy criterion (Von Mises). The maximum shear stress criterion states that the material has failed when the shearing stress on the component has reached the yield shear strength of the material, which is derived from the tensile test of the material specimen. While the maximum distortion energy criterion indicates that material failure occurs when the distortion energy of a component reaches the energy for yielding (**Javidinejad 2015).**

Von Mises criterion was first stated by von Mises without a physical interpretation and it is now accepted that it expresses the critical value of the distortion (or shear) component of the deformation energy of a body. Based on this interpretation, a body flows plastically in a complex state of stress when the distortional (or shear) deformation energy is equal to the distortional (or shear) deformation energy in uniaxial stress (tension or compression). (**Meyers 2009).**

It must be mentioned that Von Mises criteria is worldly accepted for steel structures as per EC3, EC8, AISC, UBC etc.

The elastic deformation involves the stretching of chemical bonds disappears if the stress is removed. When the stress is removed, the deformation disappears. More drastic events can occur which have the effect of rearranging the atoms so that they have new neighbours after the deformation is complete. This causes an inelastic deformation that does not disappear when the stress is removed. Inelastic deformation that occurs almost instantaneously as the stress is applied is called plastic deformation, as distinguished, which occurs only after passage of time under stress **(Dowling 2012).**

The finite element models do not predict material failure and fracture, which generally occur in laboratory tests, causing loss of strength and consequently low rotation capacity.

- **Discussion**

It is worth to point out that in numerical studies, the earlier failure phenomenon cannot be picked up with computer software used. Due to the high ductility demand on short links, the flange area of the link surface HEA might experience buckling phenomena as shown in Figure 7.13 to 7.22 as the loading s were applied through prescribed displacements increase by 1*mm* increments on one end of the link and the changes in the history of equivalent stresses in both the web and flanges up to an ultimate displacement corresponding to 40 mm. The Von Mises and Tresca contours are representing the ultimate applied for four cases with transverse stiffeners. However, for the remaining structures, the beginning of plastification is represented.

ABAQUS outcomes of nonlinear finite element analysis also include the stresses contours, and deformed shape of specimens and will be presented respectively. Figures 7.13 to 7.22. display the FEA results of different configurations of models used in this analysis.

Figure 7.13 shows the stress contours extracted from Von Mises and Tresca criterion respectively, while Figure 7.14 displays displacement deformed configuration representing the last stage of loading for the reference shear link, that is without stiffeners or unreinforced link, which in fact, represents the reference case. As can be seen from Figure 7.13 (a) and (b), a total yielding of the web is remarkable.

The flanges suffer from intensive stress field as consequence of high-level shear stress combined with normal showing clearly the yielding of the upper and lower flanges. This is accompanied with high value of the end part nonlinear displacement as it is shown in Figure 7.14. The capacity of an unreinforced web to resist shear is taken to be that related to an average shear yield stress based on the Huber-Henckey-von Mises criterion of $f_y/\sqrt{3}$.

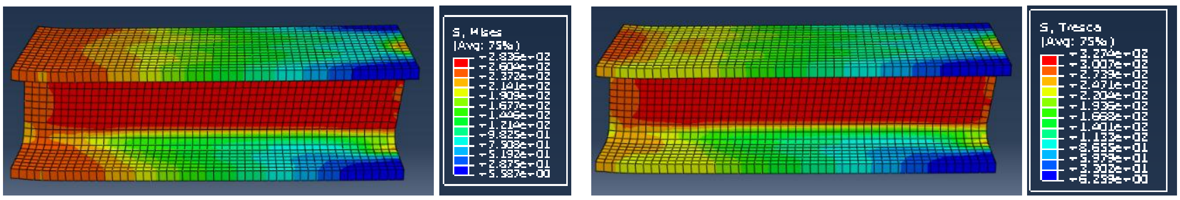


Figure 7. 13. Von Misses and Tresca stress contours respectively for short link without stiffeners**.**

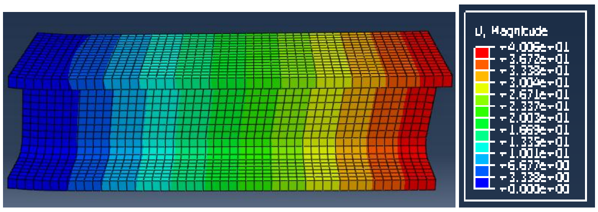


Figure 7. 14. Displacement of shear link for short link without stiffeners.

In model where a central couple of stiffeners is provided, Figure 15 (a) and (b) no major differences in the results can be noticed from the stress contour pattern, with however minor decrease in the value of maximum stress accompanied with better buckling resistance of the flanges. It can also be noticed that the central stiffener still behaves in elastic manner. The same remarks can be made for the deformation configuration.

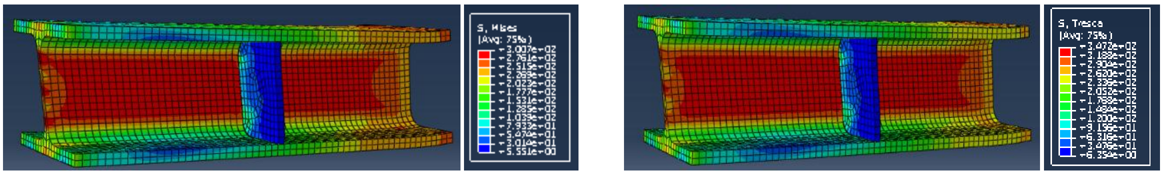


Figure 7. 15. Von Misses and Tresca stress contours respectively for short link with a coupled of central stiffeners.

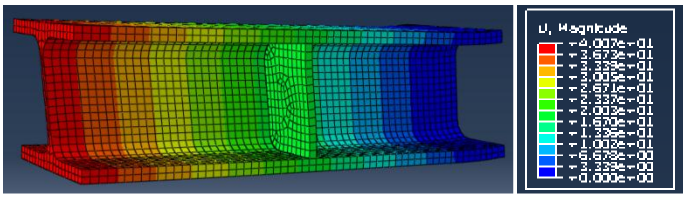


Figure 7. 16. Displacement of shear link with a couple of central stiffeners.

For the remaining cases having two and three pairs of stiffeners a greater region of material yielding may be observed which means that the web yielding in this region occurs without the buckling of the web plate, because of the presence of stiffeners, as shown in Figures 7.15(a) and (b) and 7.17(a) and (b). As the shear force is excessive, additional stiffening are placed to limit shear deformations. The effect on the inelastic behaviour of the shear links considered in this study with one central then two couples of transverse stiffeners and finely three couples of transverse stiffeners are shown in Figures 7.8 to 7.12 for both Von mises and Tresca criterion. By adding web these stiffeners, it can easily notice that the stiffness and the capacity resistance have been improved and the flanges of the cross section are being more stressed with a reduced area.

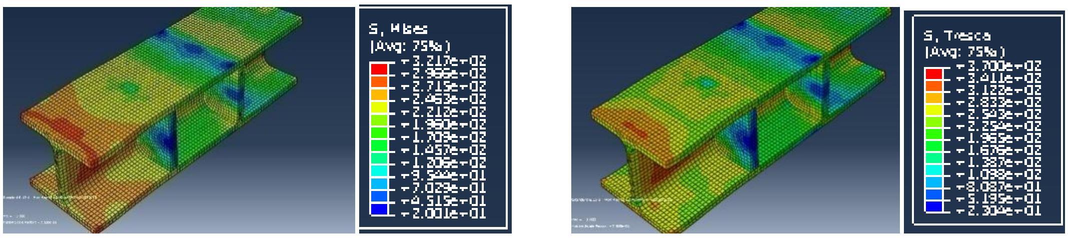


Figure 7. 17. Von Misses and Tresca stress contours respectively for short link with two equidistant couples of transverse stiffeners

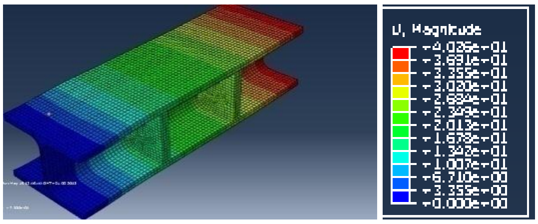


Figure 7. 18. Displacement of shear link with two equidistant couples of transverse stiffeners

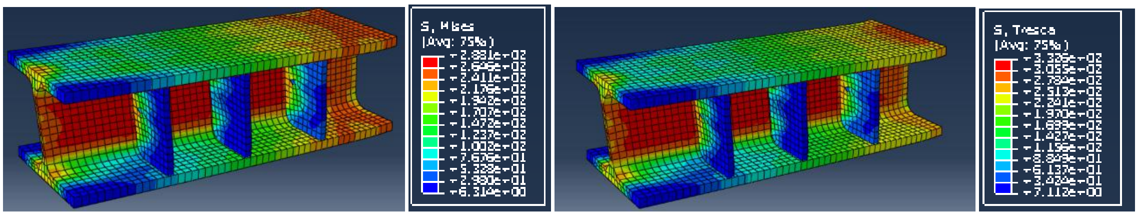


Figure 7. 19. Von Misses and Tresca stress contours respectively for short link with three equidistant couples of transverse stiffeners

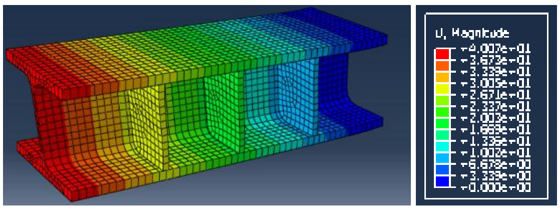


Figure 7. 20. Finite element model for ultimate displacement results of shear link with various types of stiffeners given in ABAQUS

Placing diagonal stiffeners has an impact on the nonlinear behaviour of shear links by increasing both its the strength and stiffness as was previously concluded and decrease the yield surfaces of both upper and lower flanges and able to undergo large deflections

In the additional models, not included in **(Čaušević 2008)**, which bring out the effect of diagonal stiffeners associated or not to transverse ones, are shown in Figure 7.13 and 7.18 for different configurations proposed by **(Čaušević 2008).** While Figures 7.17 and 7.18 depict depending on their configurations of diagonal stiffeners, the early stage of loading- history of the shear link to show that the first yielding occurs rather in diagonal stiffeners.

In the case of diagonal stiffeners confers more strength to the short link, associated with an increase in the capacity to resist the buckling phenomena, especially the flange local buckling and can delay the yielding of the web in both Von Mises and Tresca criterion patterns showing almost the same results patterns as far as stress field is concerned for all cases studied. In this type of stiffeners, the brace under the bottom flange could not prevent local buckling, and diagonal stiffeners must be placed at the top and bottom flanges.

As far as deformation state is concerned, an improvement can be seen from Figures 7.21 and 7.22 show clearly the favourable effect of diagonal stiffeners in reducing the general displacement of the model.

The most engaged areas in the stiffeners are located next to the right side of the shear link. This phenomenon is related to the yielding as a consequence of the redistribution of the stresses between the beam and the stiffeners.

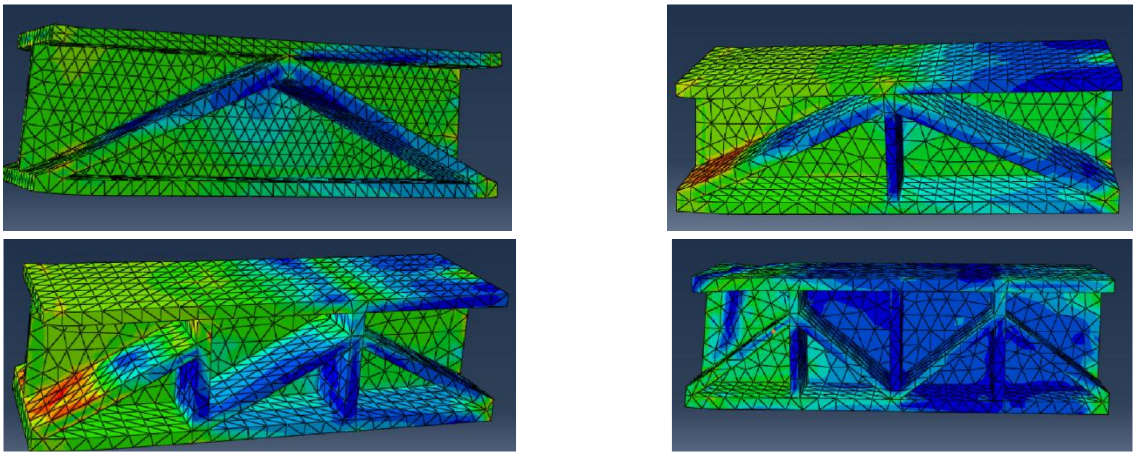


Figure 7. 21. Von MISES stress contour of web yielding for vertical, diagonal and combined stiffeners

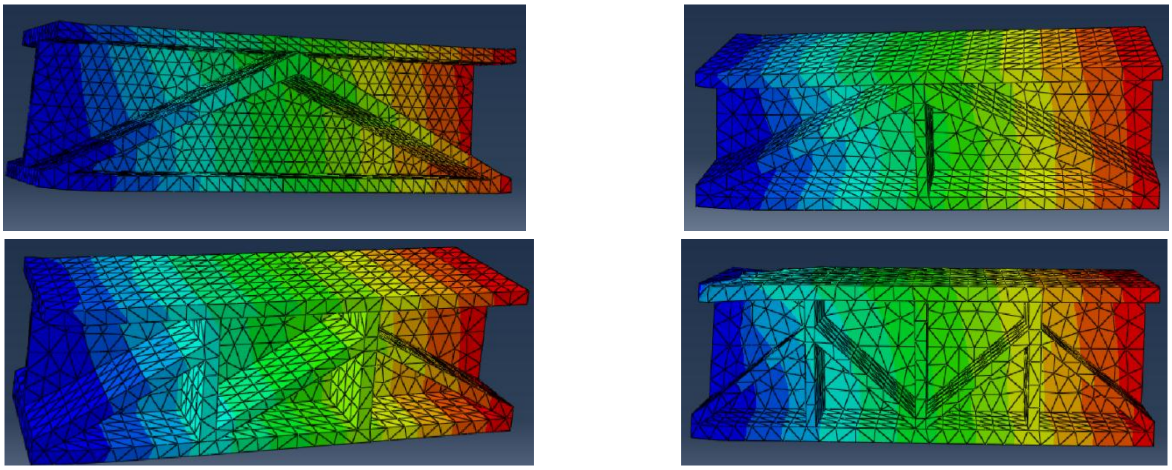
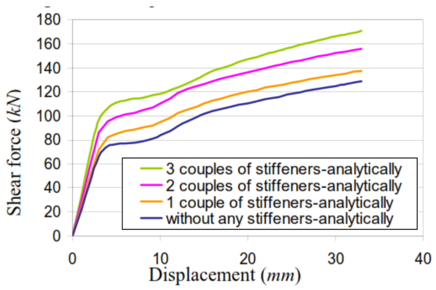


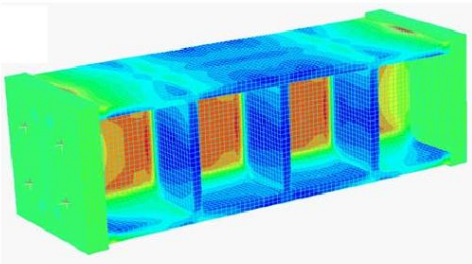
Figure 7. 22. Displacement of shear link equipped with diagonal and transverse stiffeners.

- **Comparison with Čaušević's results (Čaušević 2008)**

The comparison with the experimental results given in **(Čaušević 2008)** shows that a very good approximation is achieved and hence similar curve shapes were obtained from this study, especially in elastic zone; the yielding forces are similar for the case of transverse stiffeners as can be noticed from Figure 7.13 (a). Numerically obtained results of the present analysis in terms of stress distribution and deformation representing the drift of the right side of the segment were confronted to those in **(Čaušević 2008).**

However, for the post-elastic range, the slope of the curves is somewhat different with highest slope for the experimental results compared to numerical results obtained in this study. This is due probably due to the lack of information about tests conditions, especially the actual material characteristics including strain hardening being used in the experimental works as shown in Figure 7.23 (b).





(a) **(b)**

Figure 7.23 Force-Displacement Relationships for short seismic inks Finite element mesh after web plastification

Indeed, the elastic part of the loads- displacement curves shown in Figure 7.6 and 7.7 are roughly the same regardless the link has is reinforced or not with stiffeners. Also, the Von mises or Tresca's patterns shown in Figure 6.19, relative to the case of three transverse stiffeners, are similar to that displayed in Figure 7.23(b), with a total yield in the web. An increase of the shear force is about 11% when introducing diagonal stiffeners compared to the case without stiffener, which becomes about 15% for the case of combined stiffeners.

However, for models with two and three pairs of stiffeners a greater region of material yielding may be noted which means that the web yielding in this region occurs without buckling of the web plate and it that spacing and thickness of web stiffener has some significant effect on the nonlinear behaviour of the link. As far as stress contours are concerned predicted by Von Mises criteria, similar patterns were found from this numerical analysis and those displayed in **(Čaušević 2008).** However, some conclusions can be drawn from this work as summarized in the following.

- **Synthesis of results and conclusions**

In this preliminary work on seismic links, the main aim was to investigate the nonlinear behaviour of shear links in terms of strength and local stability (buckling) in the web and flanges. Another goal was to examine the effect of stiffeners, with different thicknesses and orientations, on the nonlinear behaviour of this type of link beam, with the possibility of increasing the strength and preventing the local stability (buckling) to occur in the web and flanges.

A large premature torsion buckling may occur on the web which in turn might cause lateral torsion buckling on the link especially for the link without stiffeners or vertical stiffeners due essentially to the loose of stiffness under inelastic deformations. When loading, the beam essentially behaves in elastic range for all configurations considered, with the attainment of the yield stress in both web and flange areas, the effect of stiffeners becomes more evident, namely, at the interface with the contact of stiffeners and the other parts of section elements: flanges and web.

The nonlinear analysis of short seismic links with HEA100 section, of class 1 as classified per EC3, was studied throughout the finite element method 3D model implanted in ABAQUS 6.13. Stress and strain response for the material was taken to be nonlinear and material properties. The parameters investigated were: thickness of transverse stiffeners, their numbers, configurations as diagonal or combined couples of stiffeners and the geometric model of the stiffener.

These models are, in first time consisting of series of four isolated links belonging to shear links kind taken from previous research work **(Čaušević 2008),** concerning the investigation of the nonlinear behaviour of isolated seismic links. The study was extended to include other models selected by the author to assess the effect of diagonal stiffeners on the nonlinear behaviour of shear (short) links are investigated.

Results of numerical analysis obtained in this study have shown the effect parameters which significantly affect the performance of shear link. It can be concluded, as a main conclusion, that the number and the type of the stiffeners have an incontestable effect on the nonlinear behaviour of short links. The installation of transverse stiffeners with a different thickness on the web of HEA profiles can improve the link performance in terms of strength, stiffness. Also, this study has proven that placing diagonal stiffeners can improve significantly the strength and the stuffiness of the shear links and can, due to the high ductility demand on short links, the flange area of the link surface HEA might experience buckling phenomena; at least, delay the apparition of buckling phenomenon in the link.

The comparison with the experimental results given in **(Čaušević 2008)** shows that similar curve shapes are obtained especially in elastic zone; the yielding forces are similar for the case of transverse stiffeners. Similar patterns of equivalent stress have been found as it is shown in Figures 7.19 and 7.23. However, for the post-elastic range, the slope of the experimental curves is different with highest slope for the experimental results compared to numerical results obtained in this study.

From results found in this analysis, it can be concluded that the installation of diagonal stiffeners will certainly provide the link with additional strength better it is in the transverse ones as they better undergo high shear forces. Also, it has been found that the idea of combining transverse and diagonal stiffeners focusing on some parameters which lead to improve the performance of shear links in terms of stiffness, strength and delaying the local buckling. The results from this analysis show the role of flanges in the nonlinear behaviour of shear link.

7.5 NUMERICAL ANALYSIS OF THE INELASTIC CYCLIC BEHAVIOUR OF SEISMIC LINKS

Based on the satisfactory and encouraging results obtained in this study, a more qualitative study will be undertaken in the foregoing section on seismic links under cyclic loading, which will be fully discussed in the next section when dealing with inelastic behaviour of links under cyclic loadings. Part of the obtained results has been published by AJCE in Oct. 2019 **(Labed 2019).**

7.5.1General

7.5.1.1 Behaviour of steel member under cyclic loading

First of all, it must be noticed that full details on the behaviour of seismic links can be found in the corresponding chapter, i.e. chapter 6. However, a concise recall will be given on the principles of design and the cyclic behaviour of seismic links in the following.

Steel structures are widely used in high seismic risk areas, due to their excellent performances in terms of strength and ductility. As steel frames are likely to be subjected to cyclic loads under earthquake and other forms of dynamic loadings will require the study of cyclic and hysteretic behaviours. All structural elements have limited strength and deformation capacities; and collapse safeties as well as damage control are depending on our ability to assess these capacities with some confidence.

The structural properties of a structure deteriorate when deformations reach the range of inelastic behaviour. A possible consequence of deterioration of the hysteretic behaviour of a structure is failure of critical elements at deformation levels that are significantly smaller than its ultimate deformation capacity.

7.5.1.2 Considerations design in EBF structures

As already stated in Chapter 6, it is a fact of life that earthquakes come in all magnitudes: small and large and the present state of the art does not permit an accurate prediction of the exact location of an earthquake or its size (**Tapan 2009).**

The main purpose of designing EBF is to restrict the inelastic action to the links and to design the framing around the links to sustain the maximum forces that the links can develop. Design using this strategy should ensure that the links act as ductile seismic fuses and preserve the integrity of the whole frame. For this reason, other components of the framing system (such as the diagonal braces, columns, and link connections) should be designed for the forces generated by the full yielding and strain hardening of the dissipative links. To this purpose, it is important to make explicit the distribution of internal actions in the EBF system and define a relationship between the frame shear force and link shear force, which depends only on the EBF configuration **(Gioncu 2013)**.

Design of EBFs habitually starts by selecting the length of links, e, at all levels based on seismic code criteria **(EC8 2005; AISC 2005; UBC 2005),** such as architectural constraints. After sizing the links, the selected length of link should be checked using material properties in order to satisfy code equations EC8 and AISC to determine which category of links belongs to.

7.5.1.3 Characteristics of seismic links

As reported in **(CISC 1992)**, short links will yield in shear prior to flexural hinging at the link ends, whereas long links will yield in flexure before shear. Either mode is acceptable, although short links are easier to design and have somewhat more stable and predictable post-yield behaviour **(Kasai 1986b, Engelhardt 1992).**

The link beam shall contain a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame. Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Long links must be Class I sections as flexural hinging is expected at link ends, whereas short links may have Class 2 flanges, provided the web is Class I **(Okazaki 2005).**

The link beam will normally carry high axial forces as well as high bending moments, and the axial forces cannot be neglected in the design. For short- and moderate-length links in particular, the web is expected to undergo severe cyclic inelastic action with straining well into the strain-hardening range. For this reason, discontinuities such as openings, splices, and stress raisers such as welded attachments (except stiffeners) must be avoided. Splices within the link are not acceptable and should also be avoided in the outer parts of the link beam near the link ends (with the exception of links attached directly to columns). The webs should be of uniform depth to maintain the same shear capacity throughout the link length, thus avoiding confined yielding.

7.5.1.4 Resistance of seismic links

The nominal resistances of the link are defined by taking into account the axial force, but this may be neglected if it is low. The interaction between bending moment and shearing force has been found to be negligible and is in fact neglected. The factored values of these resistances (nominal resistance times¢) are used when proportioning link beams for the factored load effects. Link lengths are defined for all link types. Very short links are proscribed, since they tend to undergo very high shearing deformations and develop very high and unpredictable forces.

The inelastic link rotation must be limited as specified in this Clause, to ensure that the ductile capacity of the link is not exceeded. Full-depth stiffeners on both sides of the web are required to clearly define the end of the link and to transfer the high shearing forces over the full web depth. Requirements for intermediate web stiffeners are based on physical test results and are needed to ensure the ductile performance of the link. For short links, stiffeners control shear buckling of the yielding web, while for long links, stiffeners required near the ends control flange buckling.

7.5.1.5 Buckling resistance

Here, stability loss or buckling is a phenomenon representing a system transition from one equilibrium state to another, that is when a structural member such as a plate, a stiffened plate, a stiffener, a column, etc., which are under thrust load deflect in an out-of-plane direction when the load reaches to a certain critical value. Buckling is essentially flexural behaviour.

When a certain structural member undergoes buckling, its load-carrying capacity decreases associated with a reduction in in-plane stiffness. This causes redistribution of internal forces in unbuckled structural members and increases the internal forces in these structural members, which may lead to the progressive occurrence of buckling failure of these structural members. If the load increases further, progressive buckling may result in the collapse of a whole structure.

The most dangerous form of a stability loss is the interactive buckling (coupled buckling) which usually causes the structure transition to the unstable equilibrium path, which leads to destruction of the structure with the load lower than the critical load corresponding to each mode separately. An interaction of different buckling modes occurs when the critical loads corresponding to different buckling modes are close to each other **(Kubiak 2013).**

Behaviour of a plate beyond the buckling point is called post-buckling behaviour. The post-buckling behaviour of structures depends on their type.

Eurocode 3 and CCM97 define four cross-section classes with reference to the local buckling risk. The parameter that governs what particular class a cross-section belongs to is the slenderness ratio of the individual plates of the cross-section mentioned above. The level of the slenderness ratio then governs the ability (or inability) for plastic rotational capacity, i.e. elongation at the tension side, and compression (with possible buckling risk) at the other side, for a girder subjected to a bending moment **(Akesson 2007).**

7.5.2 Numerical FEA models of links under cyclic loadings

7.5.2.1 Summary on seismic links in EBFs structures

- **General**

The seismic links are designed and expected to undertake large nonlinear deformations by an excellent dissipation of energy without losing resistance. There are substantial differences between the behaviour of short and long links **(EC8 2005; AISC 2005, UBC 2005)**. The ability of EBFs in dissipating energy strictly depends on the criteria adopted in the design: the plastic deformations are essentially located on link elements, dimensioned for yielding before beams, braces and columns that, otherwise, are proportioned using the forces generated by the yielded and hardened links in order to remain in the elastic field, according with the principles of capacity design **(Bruneau 2011).**

- **Previous studies**

Previous published analytical and experimental researches, available in literature, have demonstrated that when properly designed (EBF) systems can provide the ductility and energy dissipation capacity needed to serve as an effective lateral load resisting system to resist earthquake demands **(Hjelmstad 1983b; Malley 1984; Kasai 1986b; Uang 2001; Badalassi 2013)**. Although longer links provide more architectural freedom for openings, previous studies showed that the performance of short links is considerably better than that of long links under severe cyclic loadings in terms of strength and ductility. Also, previous research studies show that behaviour of links is fairly complicated and affected by various parameters, and as a result, significant amount of research interest has been directed towards both experimental and numerical determination of the nonlinear behaviour and cyclic energy dissipation characteristics of the links.

- **Code Design provisions and failure modes**

Design of EBFs habitually starts by selecting the length of links, e, at all levels based on seismic code criteria **(EC8 2005; AISC 2005; UBC 2005),** with the consideration of architectural constraints. After sizing the links, the selected length of link should be checked using material properties in order to satisfy code equations EC8 **(EC8 2005),** AISC **(AISC 2005)** to determine which category of links belongs to. Equations to determine the length ranges and allowable link inelastic rotation angles, for instance, have been developed for I sections as specified in EC8 in its chapter 8 and AISC Seismic Provisions. In these provisions, the length of link-beams affects the type of hinges and consequently the type of failure mechanism modes. As reported in many researches works, the ultimate failure modes of short links and long links are quite different.

- **Energy dissipation**

Generally speaking, the behaviour of link elements, and afterwards the way they dissipate energy, is related to their length (e): short links (i.e. characterized by a ratio between the plastic shear and the plastic moment smaller than 1.6 times the link length) generally develop high shear deformations, while long links (i.e. characterized by a ratio between the plastic shear and the plastic moment higher than 2.5 times the link length) mainly dissipate energy trough the formation of flexural deformations **(Okazaki 2007).**

For a short link, energy is dissipated primarily through inelastic shearing of the link web on the other hand for a long link; the energy is dissipated primarily through flexural yielding at the ends of the link. The inelastic response of a link is strongly affected by the link length and the (M_p/V_p) ratio of the link cross-section. Here, M_p = nominal plastic flexural strength of the link; V_p = nominal shear strength of the link. Considering simple plastic theory assuming no strain hardening and (M–V) interaction, it can be observed that e = ($2M_p/V_p$) is the theoretical dividing line between a shear link and a flexural link. The shear yielding energy dissipation mechanism is more efficient than the flexural plastic hinging mechanism **(Bruneau 2011)**.

- **Link stiffeners**

Owing the high ductility demand on links, the flange area of the link, surface of cross section of links might experience buckling phenomena; thus, it is required to install web stiffeners. If these web stiffeners are not installed, premature buckling may occur on the web which in turn might cause local buckling of parts of the section.

In a global analysis, a transverse stiffener shall be provided at the location of concentrated loads if patch loading resistance is exceeded. The out-of-plane buckling resistance of such stiffeners should be checked according to EC3 provisions.

To avoid eccentricity coming either from one-sided stiffener or from asymmetric stiffener should be accounted for in accordance with 6.3.3 and 6.3.4 of EN 1993-1-1. Therefore, stiffeners should be provided and placed symmetrically on both sides of the link web on both sides of the link web at the diagonal brace ends. Further, the link needs to be stiffened in order to delay the onset of web buckling and to prevent flange local buckling **(EC8 2005; Krawinkler 2009b; FEMA 1997)**. Also, previous researches, such as **(Hjelmstad 1983b, Badalassi 2013)** have determined a few simple requirements regarding web stiffener spacing with a maximum inelastic rotational angle (γ_p) to the beginning of web torsional buckling.

7.5.2.2 General considerations on models

- **Objectives**

The overall objective of the present study is to investigate the inelastic behaviour of stiffened panels with varying stiffener rigidities links under cyclic monotonic loading.

Accordingly, more than twenty models, detailed in Table 7.1, were considered with boundary conditions as shown in Figure 7.26, in which the nodes on the left end were restrained against all degrees of freedom except horizontal translation. However, nodes on the right end were restrained against all degrees of freedom except vertical translation. Loads were applied as displacement- controlled on the right end nodes. Models differ in their configurations by their length and the orientation of placed stiffeners.

- **Models configurations**

The basic geometrical features of long and short links are depicted in Figure 7.24. For each model analysed, five cases have been studied with geometrical properties summarised in Table 7.1, namely: the link without stiffeners being the reference case; links with two symmetrical transverse stiffeners 2T; links with three transverse stiffeners (one in mid-span) 3T, links with two transverse stiffeners associated to two diagonal stiffeners 2T+2D; and links with three transverse stiffeners associated to two diagonal stiffeners 3T+2D. All link segments were simulated as simply supported under constant monotonic cyclic loadings, made from hot-rolled IPE sections were employed as beam link members, of class1 as per EC3. Two IPE sections

(IPE 360, IPE 450) have been selected from chapter 6 and designed on the weak beams vs. strong columns philosophy **(EC8 2005),** and numerically modelled.

- **Definition of the models**

The overall objective of this research work was to study the influence of key parameters on the inelastic behaviour of links, namely the cyclic loading behaviour of stiffened panels with varying stiffener rigidities. Accordingly, more than twenty models detailed in Table 7.1 representing series of isolated links belonging to flexural (long) and shear (short) kind equipped with different stiffeners configurations. With two cross sections (IPE 360 and 450) which differ from their slenderness ratios. The length of each model has been designed to EC8 provisions. An additional two HEB beam segments were also modelled to study the effect of section shape on the inelastic behaviour of links.

- **Models input characteristics**

- **Material**

The only input that is needed concerning the material is a uniaxial stress-strain relation. Material model of the S235 structural steel grade is described with use of an elastic-plastic model with isotropic strain hardening and its uniaxial stress strain diagram follows that used in the material properties including, yield stress, ultimate stress and Young's modulus are identified from EC3 **(EC3 2005)** for S235 grade. The FEM material has following properties: Density ρ = 7850 kg/m^3, ε_{pl} = 0.25, f_y = 235 MPa, f_u = 360 MPa.

- **Geometry**

The geometrical characteristics of the links are provided below in Table 7.1. For each model analysed in this work, five cases have been studied, namely: the link without stiffeners; with two symmetrical transverse stiffeners; with three transverse stiffeners (one at mid-span), with two transverse stiffeners associated to two diagonal stiffeners; with three transverse stiffeners associated to two other diagonal stiffeners. It can be noted that the thicknesses of stiffeners are 8 mm and 9.5 for IPE360 and IPE450 respectively for all models (long or short links).

- **Stiffeners**

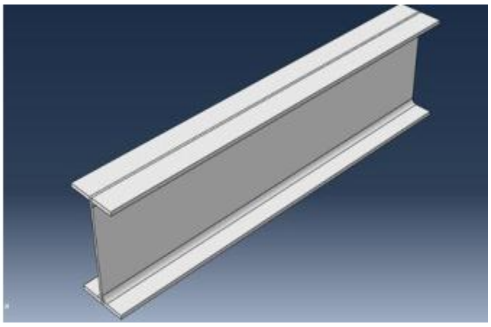
Since from the very early research works on specimens, local buckling was observed in the flange and the web of the cross section, it was decided to equip the links with stiffeners to prevent premature buckling. Considering the problematic effects of the stiffeners on the overall behaviour of EBF links, two kinds of stiffeners and their combination were being used in this study. In this study the design of the link and the different stiffeners is carried out made according to EC8 recommendations and is aimed at examining the influence the effect of geometrical parameters on the inelastic performance under the protocol cyclic loading by a displacement control of AISC 2005 cyclic loading protocol **(AISC 2005).** The link stiffeners were placed on both sides of the link according to EC8 provisions with 8 mm and 9.5 thickness of stiffeners in long and short links IPE360 and IPE450 respectively for all models.

7.5.2.3 Finite element models

Nowadays the finite element analysis is used not only for solving engineering problems but it is also used by scientists as a numerical experiment. By introducing new elements and mathematical techniques, the method has been still developing.

In this study, four short links made from IPE 360 and 450 profiles with a length of 740 and 815 mm were considered respectively. While for long links, lengths considered are 1410 mm and 1535 for IPE360 and IPE450 were studied respectively. In figure 7.25, a standard finite element for basic geometry for short link IPE360 equipped with different stiffeners configurations is shown.

The nonlinear computations were performed using the commercial finite element software package ABAQUS, as it has the ability to consider both geometric and material nonlinearities in a given model with large deflection, and large strain capability using the C3D8R. The C3D8R an 8-node linear brick, reduced integration, hourglass control), this special configuration where chosen from the ABAQUS element library due to the facility of applying and retrieving results with the minor errors. C3D8R is suitable for complex plastic buckling behaviour and has six degrees of freedom per node and provides accurate solutions to most relevant applications and also allow some triangular prisms (wedges) in transition regions. Flanges were modelled with 10 elements across the width besides 14 and 18 elements throughout the web height for IPE360 and IPE450 respectively.



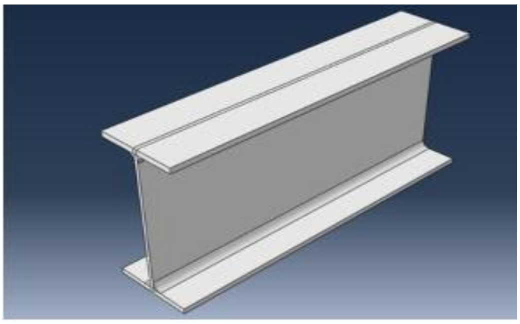


Figure 7. 23. Basic seismic link geometry configuration for long link and short link.

- **Identification of the studied cases**

The geometrical characteristics and the identification of models considered in this study are summarized in Table 7.1 bellow, in which the following notation is used:

L: is stating for long link section 1 and;

S: is referring to short link section 1, that is IPE 360, while letters with apostrophes are referring to section 2, that is IPE450.

T refers to transverse stiffeners and D for diagonal ones.

Table 7. 1. Identification of analysed models' characteristics of long (L) and short (S) links.

Model	LL	LL1	LL2	LL3	LL4	LL'	LL5	LL6	LL7	LL8	LL9
IPE1	1	1	1	1	1	2	2	2	2	2	2
Stiffeners	0	2T	2T+2D	3T	3T+2D	0	2T	2T+2D	3T	3T+2D	2T+1D
Model	SL	SL1	SL2	SL3	SL4	SL'	SL5	SL6	SL7	SL8	SL9
IPE 2	1	1	1	1	1	2	2	2	2	2	2
Stiffeners	0	2T	2T+2D	3T	3T+2D	0	2T	2T+2D	3T	3T+2D	2T+1D

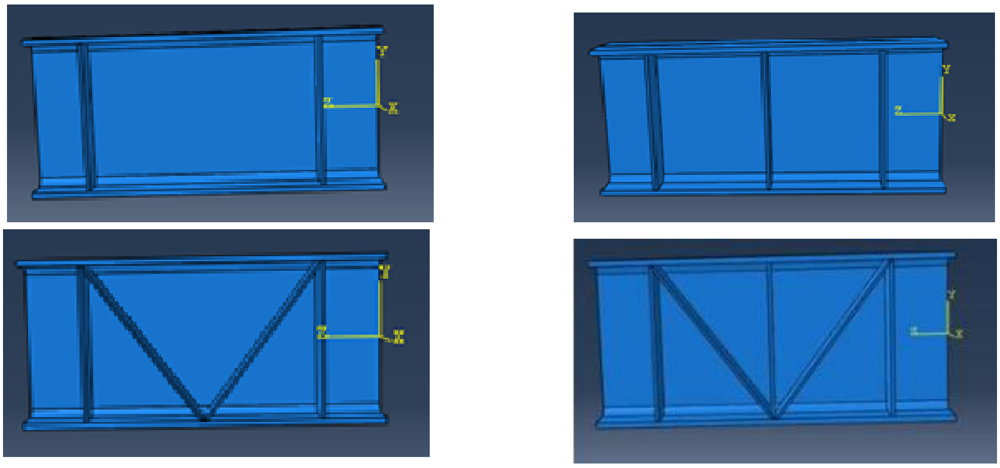


Figure 7. 24 Standard Finite element for basic geometry for short link IPE360 equipped with different stiffeners configurations.

- **Boundary conditions**

Boundary conditions considered are as shown in Figure 7.25, were nodes on the left end were restrained against all degrees of freedom except horizontal translation. However, nodes on the right end were restrained against all degrees of freedom except vertical translation.

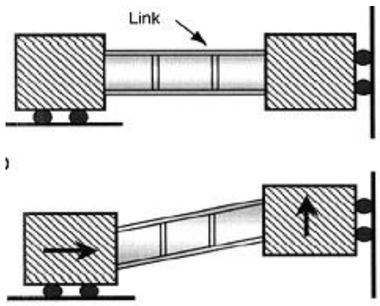


Figure 7. 25. FEM model boundary conditions **(Richards 2005)**

- **Applied loading procedure**

More details can be found in the relative chapter 5 of this thesis for cyclic loading and protocols with all recent references on the subject.

The objective of a cyclic seismic loading protocol is to simulate the number of inelastic cycles, cumulative inelastic demand, and peak displacement demand associated with a design seismic event **(Krawinkler 2009b)**. The cyclic loading protocol for the links was used to impose deformation demands consistent with earthquake loading effects.

In this study, the loading protocol was adapted from the AISC **(AISC 2005)** quasi-static cyclic deformation-controlled. Several existing loading protocols have been developed in the literature for different types of structural and non-structural components **(ATC-24 1992, FEMA 1997; EN-12512 2001; Imani 2015; Suswanto 2017,** which recommend slightly different loading histories, but in most cases, they differ more in details than in the concept. These protocols are used for quasi-static cyclic testing of structures and are based on recordings from regions of high seismicity. Hence, existing loading Protocols may over estimate seismic demands for regions of low to moderate seismicity. As the overriding issue is to account for cumulative damage effects through cyclic loading, the protocol of AISC 2005 **(AISC 2005)** has been used and applied to all models.

7.5.2.4 Discussion of results

- **Overall discussion of the obtained results**

The studied models are series of isolated links belonging to flexural and shear kind, made from two cross sections (IPE 360 and 450) which differ from their slenderness ratios. An additional two HEB beam segments were also modelled to study the effect of section shape on the inelastic cyclic behaviour of links. The length of each model considered herein is designed according to EC8 provisions.

It is worth to note that the results presented and discussed herein depend on the applied loading protocol **(AISC 2005)** as well as the selected links boundary **(Richards and Utang 2005)** and isotropic hardening **(Krawinkler 2009b).** The plastic deformation imposed on link beam segments is defined by the plastic rotation γ. This plastic rotation depends on the link length relative to the bay width. Therefore, by selecting the link length "e "and a beam section with capacities Mp and Vp, the designer can control both the type of yielding mechanism (shear or flexure) and the amount of plastic deformation imposed on the links for a given inelastic storey drift θ_p.

An overall discussion will be held for the all studied cases including long and short links made of IPE 360 and IPE450, and then a detailed discussion will be provided for each studied case. Eurocode 3, Part 1-5 also gives general rules for the shear buckling resistance of unstiffened and stiffened plates.

As already mention, links were subjected to the AISC 2005 loading protocol **(AISC 2005)** with 32 cycles for long links and 35 cycles for short links. In most studied cases, it was observed that flanges and webs suffer from local buckling, even those equipped with stiffeners, but at different level of loading intensity and at different increments as shown in Tables 7.2 and 7.3.

Local flange buckling has been noticed during the loading history in most models which is likely to be so because the folds of the web are less efficient in supporting tension fields than the flanges of a section with a flat web especially for short links. Also, during the loading history, a nonlinear buckling with geometrical nonlinearities large deflections was observed and determining the location of local buckling mode. It was not clear whether the buckling of the web or the flange is governing as results of the complexity of the interaction between different buckling phenomena. But for sure, flange section effectively contributes to the buckling resistance and post-buckling resistance was observed on all studied models, including those with unstiffened plates.

The results of this study carried out under cyclic loadings confirm, as it is stated in EC3 **(EC3 2003),** the ultimate state is a kind of plastic mechanism nearly formed in the flanges, caused by the tension field between the flanges which are designed to be fully effective and undergo local buckling.

It was noticed that for links where the bending moments can be neglected, the contribution of flanges can be added to the shear resistance of web. In other words, the contribution of flanges can be added to the shear resistance of web panels only when the flanges are not completely utilised in withstanding the bending moments. This was especially noticed for short links. Naturally, the thicknesses of flange and thickness of web are kept constant in this analysis because these parameters cannot be modified in the field due to the applied IPE hot-rolled profiles which is default from the factory unless changing the profile. For long links performance, the results of this study show that they are less effective than short links. In all the models studied here, a post critical strength is recognised for the local buckling of flanges and of web.

In general, the obtained results show a certain contribution of stiffeners in delaying premature buckling especially for short links. It was also observed that for all links, the stiffeners behave in elastic manner despite their configurations and the kind and the cross section of the link. The results show the determinant role of parameters considered and the contribution of stiffeners which in, particularly for shear links. A little contribution of stiffeners was noticed when the links were behaving in elastic manner. These stiffeners become more active as the links respond plastically and remain elastic during the loading history and delay premature local buckling.

It worth to point out that primary analysis of the FEA outcomes found in the present investigation reveal a better inelastic cyclic behaviour of links made from HEB than those of IPE, as they can undergo higher level of stresses and strains due chiefly to their geometrical shape and the reparation of material in the cross section.

Also, the primarily results demonstrate that changing the profile of segment beams from hot-rolled narrow flanges (IPE 360 and 450) to hot-rolled wide flange profiles of HEB 280 and 320 shows a better performance under inelastic cyclic loading of links, this is mainly due to a more rational matter distribution over the cross section. This conclusion needs of course to be confirmed by an intensive future research works.

- **Result presentations**

In the following, the results will be presented in the same manner for long and short links in terms of deformed model shapes and contour plots of field of equivalent stress. from the output database.

Contour plots display the variation of a variable across the surface of a model. Also, equivalent plastic strain PEEQ and PEMAG were included in the output because they are requests for this analysis. Accordingly, from left to right in each Figure, contour plots for Von Mises stress (a), the equivalent plastic strain (PEEQ) (b), the plot contour of the magnitude of plastic strain (c) and the magnitude of displacement at the last stage of loading are presented. See above for definitions of terms. Plot equivalent plastic strain contours on the cantilever beam.

N.B. Some clarification must be given to explain the physical meaning of terms used in the discussion below.

An equivalent tensile stress or von Mises stress, σ_{equi} is used to predict yielding of materials under multiaxial loading conditions using results from simple uniaxial tensile tests. As ABAQUS doesn't force to use any particular set of units. The output units depend on what units you used for the input and it is in N/mm² or MPa.

In ABAQUS, PEEQ and PEMAG are generally used since they correspond to the plastic strain value on the uniaxial stress-strain curve. PEEQ refers to the equivalent plastic strain is parameter refers to the equivalent plastic strain which describes the degree of work hardening in a material. It is essentially a scalar quantity measuring all the components of equivalent plastic strain at each position in the model, somewhat like Von Mises stress is a scalar measure of the shear stress at a point. PEMAG refers to the plastic strain magnitude of the accumulated plastic strain. Equivalent Plastic Strain (EPS) is a scalar quantity which describes the degree of work hardening in a material. Both are scalar measures. For proportional loading, the measures should be equal. However, for loading with reversals, PEEQ will continue to increase if the plastic strain rate is non-zero (regardless of sign plastic strain is used only if component values/orientation you are of interest.

- **Discussion**

- **Long Links**

The obtained results are summarized in Tables 7.2 and 7.3, for beam segments IPE360 and IPE450 respectively. Figures 7.27 to 7.31 show the final configuration of the IPE360 long links analysed, while Figures 7.32 to 7.33 are relative to long links made from IPE450.

Generally speaking, the stress and strain concentrations zones depend on the configurations of the link analysed. Same remarks can be made for PEEQ values, as the account for cumulative strains, which vary from 0.10 to 0.22% for IPE360 and remain constant and about 0.21% for IPE450. While for PEMAG values oscillate from 0.033 to 0.063 for IPE360 and once again are roughly constant for IPE450 as shown in Table 7.2. It was also observed that for all links, the stiffeners behave in elastic manner despite their configurations and the cross section of the link, IPE 360 or IPE450.

For long links made from IPE360, see Table 7.2 for the summary of the results, it can also be seen from Figure 7.26(a), the unstiffened LL link presents a clear plastic hinge at approximately mid-span which means that the failure is likely to be flexure yielding accompanied with a noticeable local bucking of the flanges at the same location. Adding stiffeners with different configurations gives certainly more rigidity to the links, and delays, nevertheless does not suppress the local buckling of both flanges and web. Therefore, the buckling zone moves from mi-span for LL to other locations for stiffened links. These zones are located in the area between the transverse stiffeners and move to the left- hand side, when the diagonal stiffeners are being used, while the remaining part are not affected by any local buckling, as it is shown in Figures 7.26 to 7.30.

As far as long links made from IPE 450 are concerned, see Table 7.2 for the summary of the results, however, inelastic cyclic behaviour for links made of IPE450 seems to be quite different. In fact, LL' which represents the unstiffened link, the plastic hinge is less visible, and the Von-Mises stress is higher as it was for LL as shown in the relative Figures. Equipping links with stiffeners with different configurations, see Figures 7.31 and 7.32, has, once again gives more resistance, but does not suppress to the section against the buckling phenomenon, where less stress concentrations were observed during the loading history. In the contrary to IPE360, the local buckling occurs approximately in the same area, that is not far from the mi-span, or at least in the area delimited by the stiffeners, despite the configuration of the stiffeners. For LL6 and contrary to LL3, the buckling of the top and bottom flanges occurs in both sides of the central area, which mean a rather symmetrical inelastic behaviour of LL6. This is probably due to the geometrical properties of each cross section, i.e. the ratio (I_y/I_z) being grater for IPE 450, and thus playing a role and leads to a more predominating flexure yielding e as it was for links with IPE360.

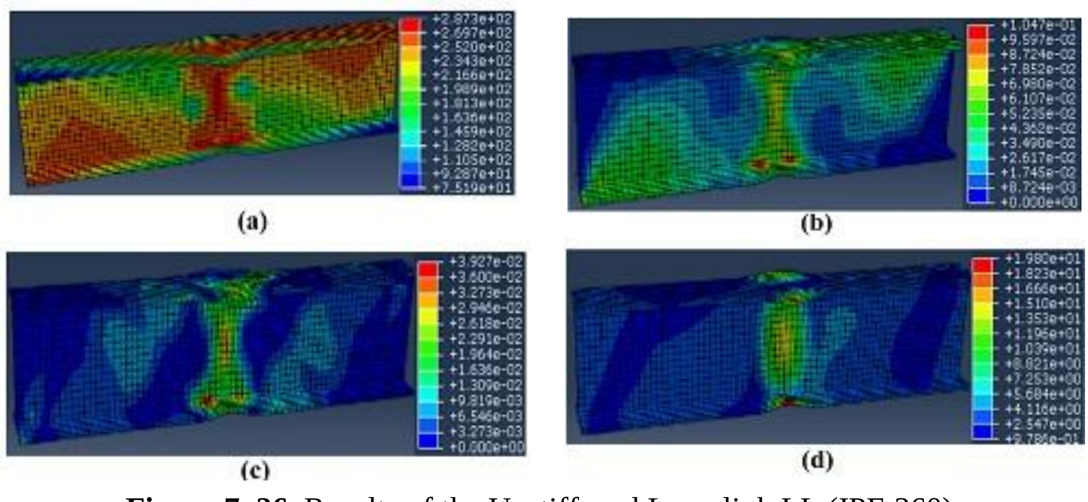


Figure 7. 26. Results of the Unstiffened Long link LL (IPE 360)
a) Von Mises Stress Contour b) Equivalent Plastic strain Distributions
c) Magnitude Plastic strain Distributions d) Magnitude Displacement Distributions

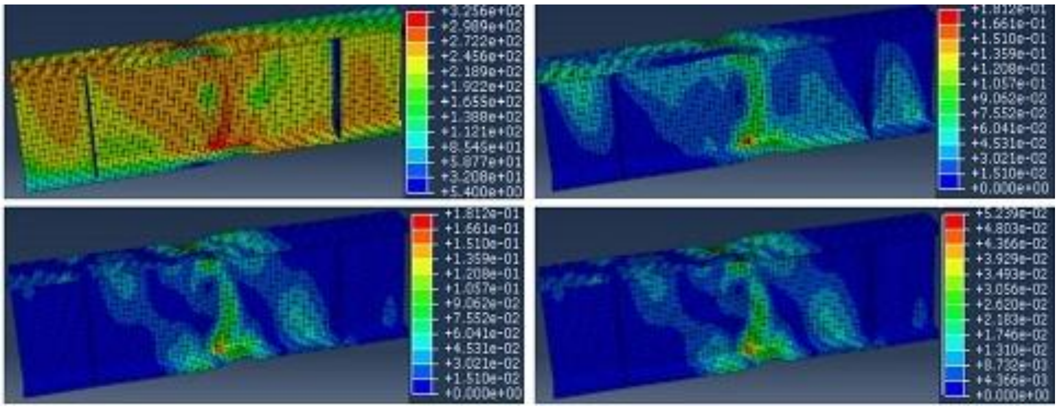


Figure 7. 27. Results of IPE 360 long link segment LL1 equipped with two transverse stiffeners

Table 7. 2. Summary of results for stiffened long links with IPE360.

Link	Elements number	Nodes Number	Increments Number	Mises MPa	Tresca MPa	PEEQ %	PEMAG %	Displ. mm	LB increment
LL1	5448	8840	199	325.6	370	0.18	0.052	19.5	130
LL2	6090	10412	216	342	381	0.21	0.063	22.6	136
LL3	5616	9264	197	294.5	337	0.12	0.033	17.3	140
LL4	6340	10800	210	346	384	0.22	0.062	21.5	148

Table 7. 3. Summary of results for stiffened long links with IPE450**.**

Link	Elements number	Nodes Number	Increments Number	Mises MPa	Tresca MPa	PEEQ %	PEMAG %	Displ. mm	LB increment
LL5	12728	20728	220	338	388	0.21	0.05	26	152
LL6	13460	21966	243	360	404	0.3	0.057	24.2	166
LL7	12932	20728	220	338	388	0.21	0.05	26.5	157
LL8	13662	22504	220	338	388	0.21	0.05	26.5	168

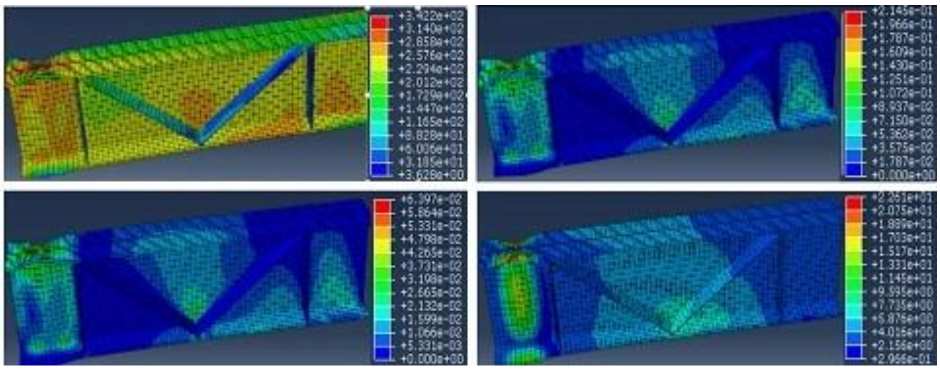


Figure 7. 28. Results of IPE 360 long link segment LL2 equipped with combined stiffeners

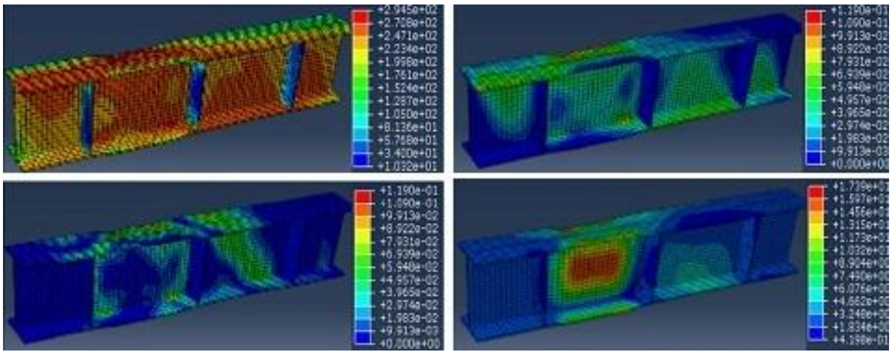


Figure 7. 29. Results of IPE 360 long link segment LL3 equipped with three transverse stiffeners

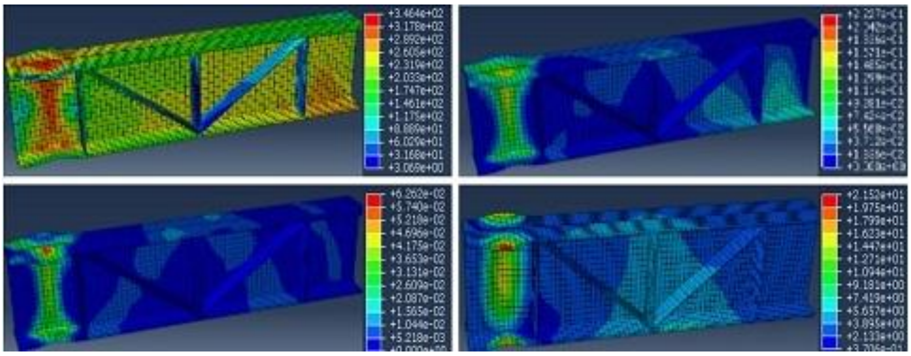


Figure 7. 30. Results of IPE 360 long link segment LL3 equipped with two diagonal and three transverse stiffeners

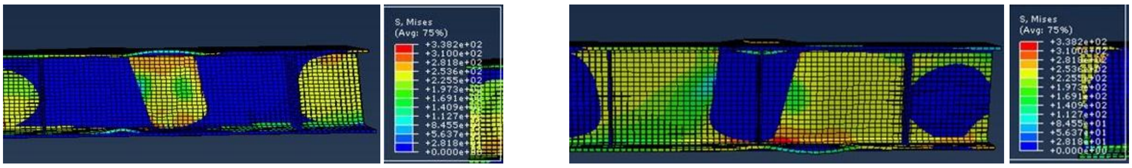


Figure 7. 31. Von Mises stress contour of long link LL (IPE 450)

a) two transverse stiffeners

(b) three transverse stiffeners respectively.

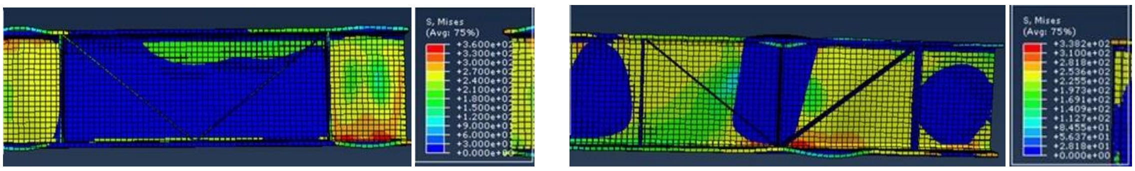


Figure 7. 29. Von Mises Stress Contour of long link LL (IPE 450)

a) two transverse stiffeners+ two diagonals and three transverse stiffeners

(b) two diagonals.

- **Shear links**

An examination of the inelastic cyclic behaviour of short links is an essential preliminary step toward the understanding of the overall inelastic behaviour of the whole structure. When structural members composed of slender elements, such as the flanges and webs of many steel shapes, such as IPE sections, are loaded axially, the overall member capacity can be limited by the capacity of the individual cross-sectional elements. Yielding in the beam link may develop in the form of inelastic shear deformations in the web of the beam, plastic rotation at the ends of the links, or a combination there of, depending on the length of the link "e" relative to the ratio of the link plastic moment capacity Mp to the link plastic shear capacity Vp as can be seen from Figure 7.33. This phenomenon is known as local buckling and is closely related to classical plate-bucking theory. Figures 7.33 to 7.39 represent the numerical results for short links IPE360 and IPE450. Accordingly, as it was the case of long links, in each figure contour plots for von Mises stress (a), the equivalent plastic strain (PEEQ) (b), the plot contour of the magnitude of plastic strain (c) and the magnitude of displacement at the last stage of loading are presented. Tables 7.4 and 7.5 displays the summary of the obtained results for shear links of IPE 360 and IPE 450 respectively.

In this particular study, and as well known, the primary yielding observed of the short links was the shear yielding. As stated in several Steel Codes, i.e. EC3, the initial mode of buckling in pure shear, which takes the form of a half-wave in the tension direction and at least one full wave in the compression direction undergoes a change in the advanced post-buckling range and eventually takes on the form of a family of diagonal folds and dominance of this tension-field behaviour. This was observed for IPE 360 SL short-unstiffened link, it was observed during the loading history that no noticeable buckling occurred.

However, because of the absence of noticeable bending moment, the flanges seem to undergo more plastic stresses and strains, and effectively contribute to the total resistance of the cross section, as for the web area in which the maximum stresses appear to act diagonally at the junction of the flange–web section and as it was the case for LL.

Indeed, owing to geometric non-linearity axial end restraints produce the development of a tension axial force, as far as the transverse relative displacement between the end sections is increased. This axial force might appreciably modify the shear buckling and post-buckling behaviour. The occurrence of this diagonal complies with the EC3 statement.

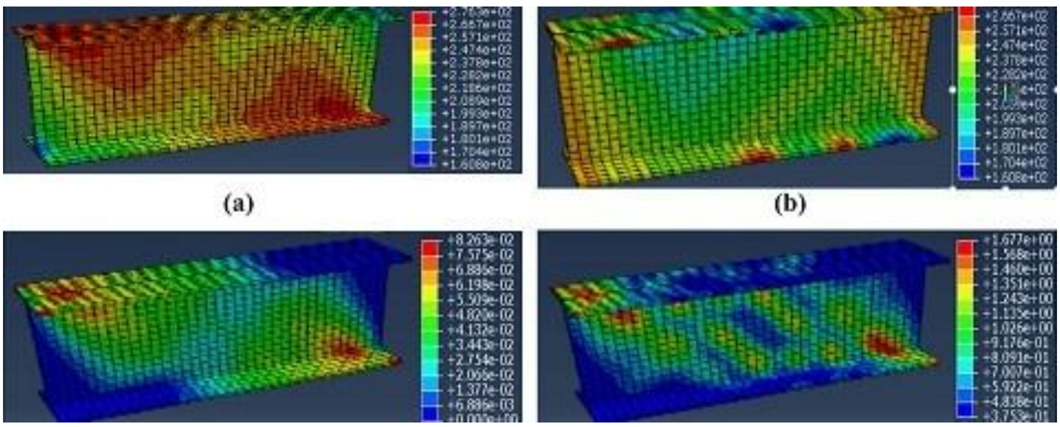


Figure 7. 30. Results of IPE 360 unstiffened short link segment. (a) von Mises stress contour. (b)Equivalent plastic strain distributions. (c) Magnitude of plastic strain distributions. (d) Magnitude displacement distribution

Table 7. 4. Summary of results for stiffened short links with IPE360.

Link	Elements number	Nodes Number	Increments Number	Mises MPa	Tresca MPa	PEEQ %	PEMAG %	Displ. mm	LB increment
SL1	3004	5074	110	275	301	0.08	0.015	2.9	-
SL2	3428	6130	121	276	301	0.08	0.010	2.4	-
SL3	3174	5502	108	275	297	0.08	0.006	1.6	-
SL4	3704	6822	107	282	303	0.09	0.005	1.8	-

Table 7. 5. Summary of results for stiffened short links with IPE450.

Link	Elements number	Nodes Number	Increments Number	Mises MPa	Tresca MPa	PEEQ %	PEMAG %	Displ. mm	LB increment
SL5	3736	6286	121	280	313	0.09	0.02	4.6	91
SL6	4240	7540	114	283.5	318	0.09	0.02	5..4	110
SL7	3964	6846	147	300	340	0.13	0.03	4.6	99
SL8	4468	8094	112	285.5	305	0.10	0.01	3	-
SL9	4468	8094	132	300	324	0.14	0.01	5.2	-

With the presence of stiffeners, this diagonal area, which is a high stress concentration zone, will appear between the limiting area of the stiffeners and exceeds roughly the elastic yielding stress (235 MPa), but less than the ultimate stress (360 MPa). In most cases with stiffeners, the web is fully yielded showing with high values of stresses. For PEEQ values it can be observed from Table 7.4 that there is an almost constant value of 0.08 for all stiffened models with the exception of SL4 for which it rises to 0.09. The values PEMAG drop from 0.015 to 0.005 from SL1 to SL4, respectively. This shows clearly the role of stiffeners in reducing the buckling and giving more strength and rigidity to the link. It has been noticed that all stiffeners behave in an elastic manner despite their configurations.

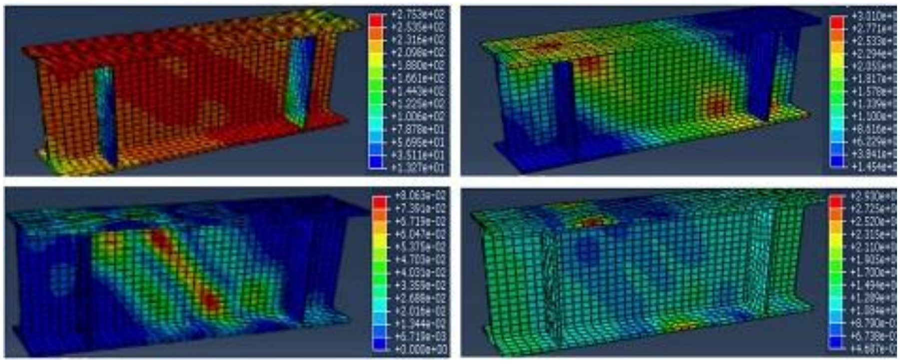


Figure 7. 31. Results of IPE 360 short link segment SL1 equipped with two transverse stiffeners

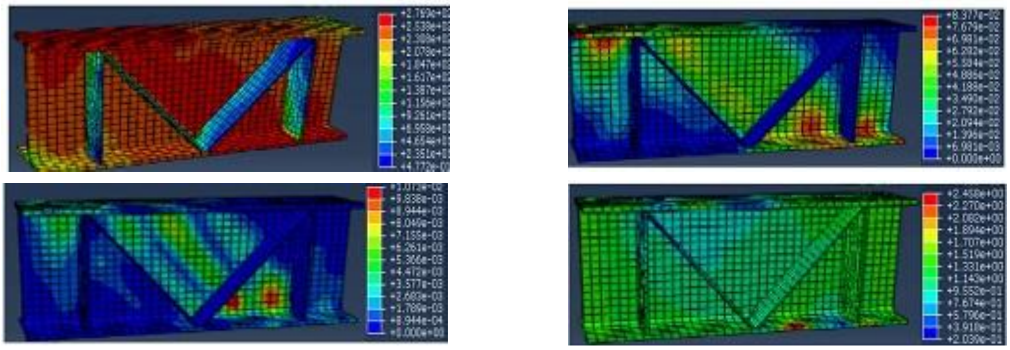


Figure 7. 32. Results of IPE 360 short link segment SL2 equipped with combined stiffeners

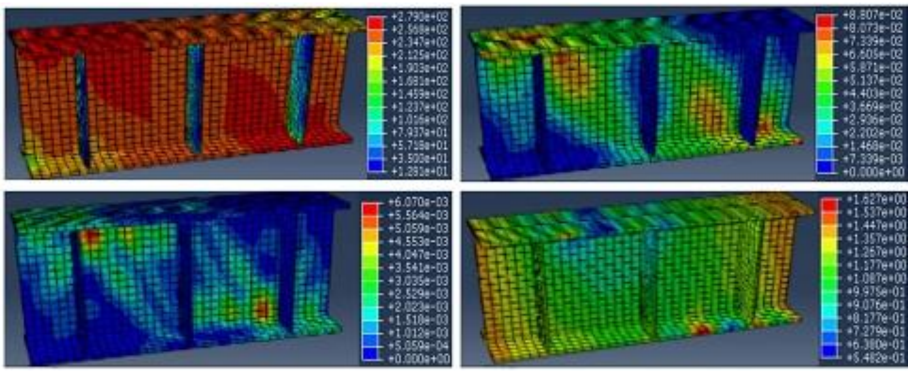


Figure 7. 33. Results of IPE 360 short link segment SL3 equipped with three transverse stiffeners

Figure 7. 34. Von Mises Stress Contour of short link SL (IPE 450) a) two transverse stiffeners and two transverse stiffeners + single diagonal respectively.

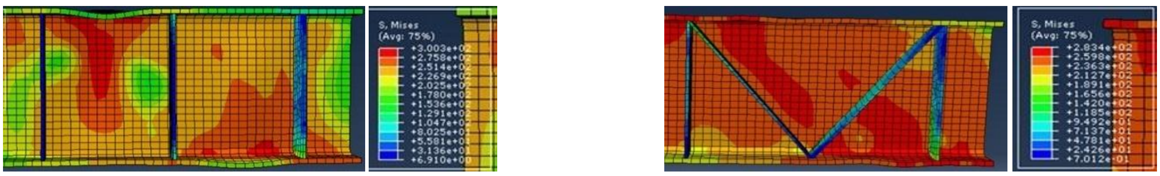


Figure 7. 35. Von Mises Stress Contour of short link SL (IPE 450) a) three transverse stiffeners and two transverse stiffeners + two diagonals respectively.

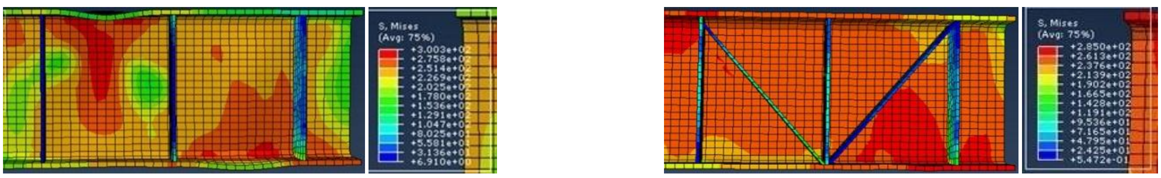


Figure 7. 39. Von Mises Stress Contour of short link SL (IPE 450) a) three transverse stiffeners and three transverse stiffeners + two diagonals respectively.

Summary

When subjected to cyclic inelastic loading, steel develops an isotropic strain-hardening response that results in increasing yield strength with additional cycles. Shear deformations are basically plane deformations of the cross-section web of the link, without any significant tendency towards lateral torsional buckling. To achieve the require flange buckling which was perceived as advantageous in that it distributes damage in the members and extends fatigue life, but high strains concentrating at the flange buckles can eventually lead to cracking of the flange's material. In many test results, flange local buckling (FLB) in inelastic cyclic tests is generally accompanied by web local buckling (WLB. Local buckling is influenced by the width-to-thickness ratio of the beam flanges (b_f/t_f) and web(h_w/t_w), and Lateral–torsional buckling is essentially governed by the lateral slenderness of the beam.

The flange local buckling is delayed by specifying width to thickness ratio, while the web local buckling will be prevented by adding number of transverse stiffeners along the web of the link. When changing the cross section of the link from IPE 360 to IPE 450, the behaviour of the latter shows some differences, namely the vulnerability to buckling phenomenon, at different increments in all modelled studied with the exception of SL8 and SL9, where no buckling was observed. For IPE 450 SL5 the short-unstiffened link, a severe buckling of both flanges and with less importance in the web is notice-able during the loading history. This may be, in part, due to the ratio of (I_y/I_z) and other geometric properties such as the J_z that causes lateral torsional bucking (LTB) of the flanges.

As it was previously said for IPE 360, and with the presence of stiffeners, in these diagonal stiffeners which have high stress concentration zones will appear between the limiting area of the stiffeners, by exceeding the elastic yielding stress (280 MPa) but less than the ultimate stress (360 MPa). For PEEQ, the values rise from 0.09 to 0.14 for SL6 to SL9, respectively. The PEMAG values drop from 0.02 to 0.01 from SL5 to SL9, respectively. Once again, it seems that the thicknesses of stiffeners are sufficient, as they behave in an elastic manner despite their configurations.

- **HEB profiles results**

Broadly speaking, the primary results obtained demonstrate that the HEB 280 and 320 hot-rolled wide flange profile links seem to behave in a better way in inelastic cyclic than hot-rolled narrow flanges (IPE 360 and 450), which is in concordance with EC8 **(EC8 2005)** and EC3 **(EC3 2003)**, in which HEB (Figure 7.41)sections have been defined as having sufficient local ductility for use in high-ductility structures. This will need obviously more attention in future works to come up with more realistic conclusions.

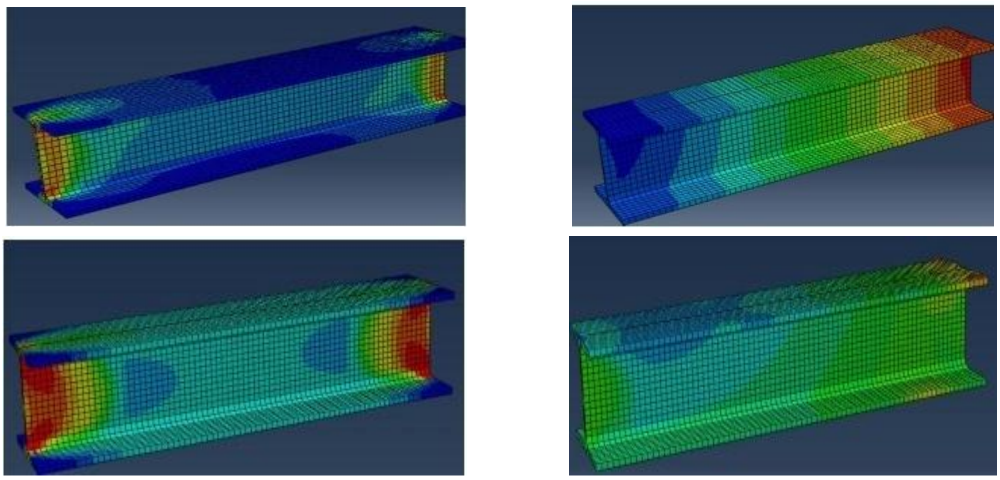


Figure 7. 36. Results for basic model made from HEB section.

7.5.2.5 Conclusions

One of the objectives of this study was to have an insight into the effect of some key parameters which are believed to affect the inelastic cyclic behaviour of links designed and modelled through FE. A series of models representing a large range of long and short links subjected to the AISC (2005) loading protocol were investigated through detailed finite element modelling implanted in ABAQUS. ABAQUS code which was selected for its capabilities of performing cycling loading analysis, taking into account both geometric and material nonlinearities with large defection and large strain capability.

Steel compression or flexural members typically employ cross-sectional shapes which may be idealized as a composition of slender elements, or flat thin1plates. Limiting the width-to-thickness ratios of elements that are vulnerable to local buckling in the inelastic range of steel structures, must not be impaired by local buckling ensuring the required plastic rotation is achieved. Assessment of link rotation capacity must therefore account for this buckling response and its impact on strength degradation and strain demand.

Inelastic response in EBFs is constrained to ductile link beam segments created at the beam ends, next to the columns when using single-diagonal members, or at the beam midspans when a chevron bracing configuration is employed. Buckling of the web is more critical in shorter, shear-dominated links whereas flange buckling is more an issue in longer, flexure-critical links. Both phenomena are accentuated when γ is increased.

It is worth pointing out that the results presented and discussed herein depend on the applied loading protocols as well as the selected link boundary conditions and isotropic hardening. The long and shear links considered in this study were expected to behave in an inelastic manner and were reinforced with stiffeners with different configurations to improve their stiffness, plastic deformation capacity and resistance against any premature buckling. Typical deformed shapes of links under cyclic loading are presented in Figures 7.27 to 7.32, long links for IPE 360 and IPE 450 respectively. Figures 7.34 to 7.40 for short links and fully discussed in the previous section.

The influence of web stiffeners on links made of European hot rolled shapes was numerically investigated with different configuration for both long and short links. The obtained results show that intermediate stiffeners are also required in links with flexure-dominant behaviour. This study shows once again, the results obtained by the comprehensive investigation by Engelhardt and Popov on long links revealed that, unlike shear links, local buckling of flanges will not necessarily cause strength degradation in stiffened long links.

The present study confirms the results obtained in **(Della 2013)**, and owing to geometric non-linearity axial end restraints produce the development of a tension axial force, as far as the transverse relative displacement between the end sections is increased. This axial force might appreciably modify the shear buckling and post-buckling behaviour. Also, based on the finite element analysis, the results have shown, once more as in recent investigations **(Imani 2015; Suswanto 2017)** that the length of a link segment (e) along with the cross-sectional properties is the key parameter that controls the stiffness, strength, and ductility. Substantial differences in performances under inelastic behaviour were observed throughout this study of short and long links having the same cross section. The apparition of buckling phenomenon, either in the flange or in the web or in both, is noticeable when the section was no more behaving in the elastic range. As the obtained results were fully discussed in the previous section, some concluding remarks can be drawn to sum up the finding of this study:

• In all models studied, and during the loading history, it was noticed that the interaction between different kinds of buckling phenomena has a clear impact on the link segment resistance, especially in the post-elastic regime. Notwithstanding, a post-buckling resistance was observed on all models, even for unstiffened links, as the von Mises equivalent stresses reach the yielding limit, but does not exceed the ultimate stress specified in EC3, which is also true for unstiffened links.

• In all studied cases, it was observed, when highly loaded, flanges and webs suffer from local buckling with distinct intensity level and at diverse locations under different loading increments. Also, the deterioration in strength due to plastic local buckling under cyclic loading was detected during cyclic loading.

• A post-buckling resistance was observed on all studies including the unstiffened link models, as the von Mises equivalent stresses reach the yielding limit but does not exceed the ultimate stress specified in EC3.

• It was also observed that under cyclic loadings, especially for long links the ultimate state is a kind of plastic mechanism which is nearly formed in the flanges, caused by the tension field between the flanges which undergo local buckling as stated in EC3 for static loading. The ratio (bf/tf) of slenderness is then of prime importance.

• As the bending moments can be neglected in shear links, the contribution of flanges can be added to the shear resistance of web, especially for short links which will obviously be influenced by the geometry of the web.

• During loading history, when the links were still behaving elastically, it was noticed that the contribution of web stiffeners seems to not be effective until the segments reach yielding and, at that time, the effects of stiffeners are more observable in short links than for long links.

• Links equipped with stiffeners, with different configurations, will surely give more rigidity to the links and can delay, but does not totally suppress, the local buck-ling of both flanges and web to a plastic mechanism that should occur on the link becomes unattainable especially on long links that cause links to behave like beam. This appears to be more evident when using diagonal stiffeners associated with the transverse stiffener's configuration.

Conclusively, it is worth recalling that the links used in this study were parts of multi-storied structures designed, in previous authors' works, according to RPA99 provisions.

The authors seriously think that the time has come for RPA in its future version to include the EBF structures as seismic lateral resisting system and to adopt provisions of links from international seismic codes EC8, AISC, as their provisions are very close to each other.

CONCLUSIONS AND PERSPECTIVES

CONCLUSIONS AND PERSPECTIVES

Based on the obtained results in this thesis, several conclusions may be drawn to express the necessity of improving the future revision of the RPA code provisions to meet with the advances made in seismic engineering, as the current provisions of RPA99 are based upon the knowledge of the eighties and nineties of last century.

In the following, only important conclusions will be cited.

1. In chapter four, seismic methods of analysis described in Chapter 3 have been used through study cases of multi-storeys steel frames performed starting from basic cases of multi-storeys MRFs frames, to investigate the effect of bracing systems when associated to MRFs, then the behaviour of MRFs-EBF dual structures (not covered by RPA99) in an elastic behaviour to more complicated ones dealing with the nonlinear analyses of regular and irregular steel structures.

2. In section 4.5, the elastic behaviour of a series of MRFs frames was undertaken in order to highlight the effect of some parameters, namely the aspect ratio (l/h) which has been found to have a definite influence on the overall elastic seismic behaviour of the structures in terms of forces and displacements, depending of the storey number and aspect ratio, with conservative results of RPA99 compared to EC8's, which places RPA99 predictions in the safe side.

2. In section 4.6, which is devoted to study the influence of concentrically full or alternative bracing systems configurations in the elastic behaviour of multi-storeys steel structures. Once again, the effect of aspect ratio (l/h) and the structure slenderness $\lambda = (H_t / L)$ along with the configuration of concentrically bracing system (X or IV) have been shown to be decisive in the elastic behaviours of structures in both seismic codes RPA99 and EC8 (Type 1 and 2).

It appears worthy to place braces (X or IV) associated to MRFs structures with alternate concentric bracing for low-rise which yields in a considerable reducing of the displacements and then has a significant economical repercussion. The use of one or other of the alternate concentric bracings is not particularly important. However, as the structures become slender their effect decreases significantly and does not reduce the displacement and the inter-storey drift and consequently the criteria of inter-storey drift are not satisfied and a check of P-delta effect is necessary.

Also, it has been found that, when the ratio l/h increases, this will give supplementary stiffness to the structures and better elastic behaviour is noticed. The performance of Dual-CBFX structures is shown to be better than Dual-CBFIV structure as stated in RPA9 seems to be correct. For all examined structures, for both seismic codes, the obtained values from LFM are always higher than those predicted by Spectral Method. An important finding of this section is that the RPA99 predictions in terms of displacements, base shear, seismic storeys forces etc.

are always laying between EC8's of Type1 and type 2 earthquake, regardless the applied analysis which gives some confidence in RPA99 as far as the elastic analyses are concerned.

3. The first insight of Dual-EBFs, not covered yet by RPA, of elastic is made. This attempt is made by considering several multi-storeys frames with different link length along with other parameters described in the above sections. According to first results obtained in this study which is, of course preliminary and limited, showed that the use of the dual structures in MRFs-EBFs can be defensible when the seismic links remain elastic under pure shearing, i.e. the EBF1or in pure bending with EBF2. The results given by LFM and SMA confirm that and, at least for the structures with small slenderness ratio, the criteria of the RPA concerning the drift-displacement can be satisfied. Therefore, the behaviour of EBF1 is acceptable and can be compared to CBF with more ductility and adopting the same value of R coefficient = 4, although this unique value does not take into account the degree of ductility of structures (MRF and CBFIV or even EBF).

4. A natural and enlargement of the previous study concerning the linear and the nonlinear global behaviours of multi-storeys frames with different bracing system configurations including Dual-EBFs structures was considered.

According to the obtained results through this particular study, the most important conclusion which can be deducted is that the use structures designed to meet RPA99 provisions with concentrically braces applied to EBFs bracing systems can be justified and therefore applicable for different lengths of seismic links: short, medium and long.

Details of the design of seismic link can be done by using either the EC8 provisions or American codes as they are all based upon the works of Popov and his colleagues, and the nonlinear pushover analysis can be safely used, with loading patterns from RPA, as it gives very encouraging results compared to EC8 ones as, the pushover curves or capacity curves obtained are similar to each other's for two loading patterns, and gives an overstrength ratio exceeding the value limited by EC8. It was found, by mean of comparison, to confirm conclusions of in many research publications, that the global inelastic behaviour of EBFs appears to be very close to CBFs for shorter links, and nearer to UB for long links. For all EBFs structures, the shear force in the seismic links exceeds sometimes the limit values indicated in EC8 because of absence of transverse stiffeners necessary to withstand the inelastic deformation in links highly excited by namely major earthquake regardless the number of storeys.

As expected, the Dual-EBFs structures show, as for other typologies, that the values of base shear forces increase or decrease depending on the length of the link. For slender structures of 8 and 10 storeys, the presence of a diagonal of bracing in CBF or EBFs does not seem to be necessary since the efforts in these diagonals always remain very low, which is not the case for low-rise structures where diagonals are highly stressed and a plastic hinge in the compressed diagonals of the lower floors. is commonly formed in lower levels of the frames. In almost the same chronology of plastic hinges appear in both Dual-EBFs or CBFs, stating from bottom storey to upper storeys. Also, the first plastic hinges occur in the beams in lower storeys regardless the bracing configurations.

5. A particular aspect of the structure skeleton is being investigated dealing with vertical stiffness irregularity in multi-storeys frames which may lead to soft weak-storey mechanism, are not explicitly covered by RPA99. In view of the obtained results by the linear and nonlinear static pushover analyses of series of 2D structures models by changing the ground storey height from 3, 4.5 and 6m, the general primary conclusion may be deducted concerning the influence of irregular ground storey which is found to be remarkable.

The soft-storey irregularity effects on the studied structures are observed to not be very pronounced in elastic range, although all of the models are designed according to the current design codes where the weak beam-strong column concept is satisfied and which reveals once again the advantages of such concept, which is unfortunately not clearly stated in RPA99. As an expected behaviour, the force and displacement demands are concentrated at this level. The significant variation of column stiffness along the height causes a 'soft-storey’ in the irregular frame; large drifts are observed at the second floor. Another important conclusion is that even if the shear frames do not reveal the soft storey behaviour during the elastic range, however in the inelastic range soft storey behaviour may be expected and can easily be observed and collapses of these models always occur due to this behaviour if subjected to major earthquake.

To reduce this effect, some arrangement of reinforcing bracing system of the uprights is placed at the ground floor, that is to say where the soft weak-storey is potentially attended and consequently, model results show a higher stiffness by braces give better results in low-rise buildings, i.e. four down storeys, than it is for higher frames, i.e. six and up storeys relating, regardless the typology of braces, to deformability which is a characteristic of a “soft-floor” type of collapse mechanism that may lead to global instability due to second-order effects.

6. Firstly, the author has analysed the nonlinear behaviour of isolated short seismic links in terms of strength and local stability (buckling) in the web and flanges, taking into account the effect of the number of stiffeners their configuration and spacing data was taken from literature (Čaušević 2008).

Additionally, in the purpose to achieve the possibility of increasing the strength and preventing the local stability (buckling) to occur in the web and flanges, another study with the same data has been undertaken to assess the effect of diagonal associated or not with transverse stiffeners on the nonlinear behaviour of this type of link beam, the results show some improvement in the loading capacity of the links is undertaken.

The comparison with the experimental results given in **(Čaušević 2008)** shows that similar curve shapes are obtained especially in elastic zone; the yielding forces are similar for the case of transverse stiffeners. However, for the post-elastic range, the slope of the experimental curves is different with highest slope for the experimental results compared to numerical results obtained in this study. It can be concluded that the installation of diagonal stiffeners will certainly provide the link with additional strength.

Also, the present analysis focused on vertical diagonal and combined stiffeners where the parameters, such as the stiffeners thickness, could easily be modified to increase performance of shear link in terms of stiffness, strength and delaying the local buckling due to the high level of loading. Also, the results from this analysis show the role of flanges in the nonlinear behaviour of shear link.

7. Secondly, based on the satisfactory and encouraging results obtained in the previous section, a more qualitative study has been undertaken in the foregoing section on seismic links under cyclic loading, which will be fully discussed in the next section when dealing with inelastic behaviour of links under cyclic loadings. Part of the obtained results has been published by AJCE in Oct. 2019 **(Labed 2019).**

Results of the present study confirms the results obtained in (Della 2013), and owing to geometric non-linearity axial end restraints produce the development of a tension axial force, as far as the transverse relative displacement between the end sections is increased. This axial force might appreciably modify the shear buckling and post-buckling behaviour. Also, based on the finite element analysis, the results have shown, once more as in recent investigations (Imani 2015; Suswanto 2017) that the length of a link segment (e) along with the cross-sectional properties is the key parameter that controls the stiffness, strength, and ductility.

Substantial differences in performances under inelastic behaviour were observed throughout this study of short and long links having the same cross section. The apparition of buckling phenomenon, either in the flange or in the web or in both, is noticeable when the section was no more behaving in the elastic range.

An interaction between different kinds of buckling phenomena has a clear impact on the link segment resistance, especially in the post-elastic regime and the von Mises equivalent stresses reach the yielding limit, but does not exceed the ultimate stress specified in EC3, which is also true for unstiffened links. Flanges and webs suffer from local buckling with different intensity levels and at diverse locations under different loading increments. Also, the deterioration in strength due to plastic local buckling under cyclic loading was detected during cyclic loading.

- **Concluding and suggestions**

By their very nature, structural codes are subject to periodic revision and amendment. From time to time, existing design rules are modified to incorporate significant research findings or new rules are introduced to allow for more advanced or novel methods of analysis and construction. In this thesis, an attempt has been made to outline the fundamental principles and to highlight their application in RPA99. Moreover, a qualitative evaluation of the RPA99 code clauses has been presented. It is hoped that, during the next version of RPA99 when it is issued in a few years time, will help fuel further growth and give more attention required to steel structures in the proper and effective use of steel in construction in Algeria. RPA99 has been produced by the combined efforts of a large number of experts and should be, in its future version, sufficiently clear, transparent and comprehensible for practicing engineers.

Finally, the main concluding remarks resulting from this study can be summarised as follows:

The main conclusion from this long study, including global and local analyses of multi-storeys steel frames, is that the time has come to RPA99 to revise, renew and update provisions regarding steel frames and to consider advances in earthquake engineering detailed in chapters 1, 2 and 3 of this thesis, like most of the developing countries, i.e. Turkey, Iran and Indonesia for instance.

RPA99 Seismic Code should **imperatively** in its future version states the use of strong columns vs. weak beams **SCWB** as basic principle for designing steel structures, allow the use of nonlinear analysis (Pushover analysis) for assessing nonlinear behaviour of steel structures, and give more details on the conditions of avoiding soft-storey mechanism as it is stated in UBC97 and, Turkish seismic code, EC8 etc.

RPA99 Seismic Code should permit the use of EBFs frames with the same provisions proposed in many seismic codes as they give very similar requirements based upon on the works of Popov and his colleagues.

RPA99's future version should include a more developed provisions for classifying structures ductility on the base of International seismic codes with, of course, RPA99's philosophy.

RPA99's current provisions as far as the elastic behaviour of multi-storeys frames, yield in conservative values in terms of seismic outcomes, and hence are in the safe side, however being uneconomical, compared to EC8's. This may be due to the values of R factor whose value can be unified like that recommended in EC8 for all typologies (R = q = 4) regardless the typology and including EBFs structures or at least proposing R value for EBFs equal to the case of MRFs. Also, the use of a design spectrum instead of elastic horizontal, as it is for EC8, may has it impact of the seismic output, giving higher values for RPA.

Also, in incoming future RPA version, an elastic horizontal spectrum must be proposed instead of the design spectrum as being more economical.

Perspectives

RPA99 has been produced by the combined efforts of a large number of experts. It is based on limit state philosophy and probabilistic safety concepts and has been produced in a format which should be sufficiently clear, transparent and comprehensible for practicing engineers.

The earthquake is a permanent danger for a country like Algeria with the advent of the last tremors occurring in the summer of this year (2020) in Mila in the northeast of Algeria, consequently it is a must for authorities to encourage the young generation of researchers in the field of earthquake and seismic engineering to carry out studies on the following topics:

- Future research should be conducted toward developing for RPA code an elastic horizontal spectrum along with another taking into account the vertical action of earthquake.
- Further studies will focus on the use of more accurate elastic and inelastic dynamic analyses, that is Time-history, should be carried out on series of multi-storey -storeys frames in order to incorporate its use in a future RPA's provisions.
- Semi-rigid connections, in use in many codes, should be analysed in the context of RPA's philosophy of design in order to be incorporated in the Algerian Seismic Code.
- Comprehensive investigation to review the conclusive and adequate values of R coefficient to be given to different typologies, including EBF structures.
- Similarly, studies on based-restrained structures must be encouraged to be used in high risk seismic areas.
- Slender steel structures or high-rise buildings located in low and medium seismic areas in linear and nonlinear dynamic behaviours should have a particular interest.
- Study of the buckling post-elastic effect on members of frames on the performance and failure of the whole structures.

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