الجمـهـوريـــــة الجـــزائريـة الديمـــــــقـراطيـة الـشـــــــــعـبيـة

People's Democratic Republic of Algeria

وزارة التـــــعـليـــم العــــالي والبـــحث العــــلـمـــي

Ministry of Higher Education and Scientific Research

جـــــــــــــــــامعة العربــــي التبســــي- تــبســـــــــــــة

Larbi Tébessi University– Tebessa –

Faculty of Sciences and Technology

Departement of Civil Enginnering

THESIS

Submitted in fulfilment of the requirements for the degree of
Master

In: civil engineering

Speciality: Structural Engineering

By: Brahmi Rania

NUMERICAL INVESTIGATIONS OF SEMI-RIGID CONNECTIONS IN BEAM TO COLUMN STRUCTURES UNDER MONOTONIC AND CYCLIC LOADING

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2020/2021

ABSTRACT

The theme addressed in this Academic Master research work was aimed to the improvements of the existing recommendations in the future version of CCM97 relating to steel structures connection with the insertion of the semi-rigid connection, for which the beam to column connections are assumed to be rigid or pinned. This dissertation is dealing with the behaviour of the beam to column connection under monotonic and cyclic loading. Actually, the study undertaken in this dissertation stems from two different parts. Firstly, a substantial effort has been made understand the topic of the behaviour of semi rigid connections with a literature review was performed on the general subject of beam to column connections including the theoretical background as well as the code's prescriptions. The second part of the undertaken research work concerns the validation of experimental and theoretical results of 3D modelling with ABAQUS under monotonic loading. Taking into account material and geometric nonlinearities, of a beam to column structure, then a parametric analysis was performed of the significance of some design parameters: steel grade, bolt grade and the contribution of web- stiffeners. While no significant difference in behaviour was observed between the connections with different bolt grades, the steel grades seems to have a significant effect. To assess the deterioration of the structural properties of a structure when deformations reach the range of inelastic behaviour during a seismic event. A more complex of beam to column connections analysis under cyclic loading behaviour through 3D models implanted in ABAQUS software using the SAC 2000 loading protocol was carried out to assess the hysteresis loops and energy dissipation at high deformation levels. Several difficulties have arisen during the 3D modelling including the mesh refining, the boundary and contact conditions the specially the introduction of the loading protocol. Some encouraging results have been obtained to understand the beam to column connections in terms of the hysteresis energy dissipation and the contribution of each component of the connection especially the panel zone and the effect of web stiffeners in the global behaviour of the structure.

Key words: semi-rigid; beam to column; monotonic; cyclic; loading protocol; energy dissipation.

RESUME

Le thème abordé dans ce travail de recherche de Master Académique visait d'améliorer les recommandations existantes dans la future version du CCM97 relatives à l'assemblage des structures *en acier avec l'insertion de la connexion semi-rigide, en plus des assemblages rigides et articulées. Cette thèse traite le comportement de l'assemblage poutre-poteau sous chargement monotone et cyclique. En fait, l'étude prise dans cette thèse s'articule autour de deux parties différentes. Tout d'abord, un effort substantiel a été fait pour comprendre le sujet du comportement des assemblages semi-rigides avec une revue de la littérature sur le sujet général des assemblages poutre-poteau comprenant le contexte théorique ainsi que les prescriptions du code. La deuxième partie des travaux de recherche concerne la validation des résultats expérimentaux et théoriques de la modélisation 3D avec ABAQUS, sous chargement monotone prenant en compte les non-linéarités matérielles et géométriques, d'une structure poutre-poteau, puis une analyse paramétrique a été réalisée pour étudier l'influence de certains paramètres de conception : nuance d'acier, nuance de boulon et contribution des raidisseurs d'âme. Bien aucune différence significative de comportement n'ait été observée entre les assemblages avec différentes nuances de boulons, les nuances d'acier semblent avoir un effet significatif. Pour évaluer la détérioration des propriétés structurelles d'une structure lorsque les déformations atteignent l'étape de comportement inélastique lors d'un événement sismique. Une analyse plus complexe des connexions poutre-poteau sous un comportement de chargement cyclique grâce à des modèles 3D implantés dans le logiciel ABAQUS à l'aide du SAC Le protocole de chargement 2000 a été réalisé pour évaluer les boucles d'hystérésis et la dissipation d'énergie à des niveaux de déformation élevés. Plusieurs difficultés sont apparues lors de la modélisation 3D dont le raffinement du maillage, les conditions aux limites et de contact et notamment l'introduction du protocole de chargement. Les résultats obtenus dans cette étude ont permis de mieux appréhender le comportement des assemblages poutre-poteau en termes de dissipation d'énergie d'hystérésis et la contribution de chaque composant de la connexion en particulier la zone du panneau et l'effet des raidisseurs d'âme dans le comportement global de la structure.*

Les mots clés : semi-rigide ; poutre-poteau ; monotonique ; cyclique ; protocole de chargement ; dissipation d'énergie.

الملخص

يهدف الموضوع الذي تم تناوله في هذا العمل البحثي للماجستير الأكاديمي إلى تحسين التوصيات الحالية في الإصدار المستقبلي من CCM97 المتعلقة بالهياكل الفولاذية المتصلة بإدخال الوصلة شبه الصلبة، والتي يُفترض أن تكون وصلات الحزمة إلى العمود جامدة أو مثبتة. تتناول هذه الرسالة سلوك الحزمة إلى وصلة العمود تحت تحميل رتيب ودوري. في الواقع، الدراسة التي أجريت في هذه الرسالة تنبع من جزأين مختلفين. أولاً، تم بذل جهد كبير لفهم موضوع سلوك الروابط شبه الصلبة مع مراجعة الأدبيات التي تم إجراؤها على الموضوع العام لتوصيلات الحزمة إلى العمود بما في ذلك الخلفية النظرية بالإضافة إلى وصفات الكود. يتعلق الجزء الثاني من العمل البحثي الذي تم إجراؤه بالتحقق من صحة النتائج التجريبية والنظرية للنمذجة ثلاثية الأبعاد باستخدام ABAQUS ، في ظل تحميل رتيب مع مراعاة المواد وغير الخطية الهندسية ، من حزمة إلى هيكل عمود ، ثم تم إجراء تحليل حدودي لأهمية بعض معايير التصميم: درجة الفولاذ ودرجة الترباس ومساهمة أدوات تقوية الويب. بينما لم يلاحظ أي اختلاف كبير في السلوك بين الوصلات مع درجات مختلفة من البراغي، يبدو أن درجات الصلب لها تأثير كبير. لتقييم تدهور الخصائص الهيكلية للهيكل عندما تصل التشوهات إلى نطاق السلوك غير المرن أثناء حدث زلزالي، تحليل دوري تحليل أكثر تعقيدًا من الحزمة إلى العمود تحت سلوك التحميل الدوري من خلال النماذج ثلاثية الأبعاد المزروعة في برنامج ABAQUS باستخدام SAC تم تنفيذ بروتوكول التحميل 2000 لتقييم حلقات التخلفية وتبديد الطاقة عند مستويات عالية من التشوه. نشأت العديد من الصعوبات أثناء النمذجة ثلاثية الأبعاد بما في ذلك تحسين الشبكة والحدود وظروف الاتصال وخاصة إدخال بروتوكول التحميل. تم الحصول على بعض النتائج المشجعة لفهم وصلات الحزمة إلى العمود من حيث تبديد طاقة التباطؤ ومساهمة كل مكون من مكونات الاتصال خاصة منطقة اللوحة وتأثير تقوية الويب في السلوك العالمي للهيكل.

الكلمات الرئيسية: شبه صلبة؛ كمرة-عمود؛ مونوتوني؛ دوري؛ بروتوكول التحميل؛ تبديد الطاقة.

Acknowledgements

First and foremost, praises and thanks to the God, the Almighty, for his showers of blessings throughout my research work to complete the research successfully.

I would like to give my deep and sincere thanks to my honorific supervisor, Dr.abderrahim LABAD, whose immense knowledge, understanding, and patience contributed to

my development as a graduate student. Without his persistent help, continuous support, guidance, motivation, and encouragement, this research would not be possible. It was a great privilege and honour to work and study under his guidance.

Besides my supervisor, I would like to thank the committee members, Dr. Redha SOLTANI and Dr.H'mida HAMIDANE, for their time and contributions on this thesis. Their critical comments and suggestions enabled me to notice the weaknesses of my thesis and make the necessary improvements according to their comments.

Special thanks are also given to Dr. abd el Halim Madjour for taking the time to answer my questions and giving me important advices.

Finally, I must express my very profound gratitude to my parents, my sisters, my brothers and to my friends for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of researching and writing this thesis. This accomplishment would not have been possible without them.

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LIST OF SYMBOLS AND ABBREVIATIONS

3D	Three Dimensional
AISC	American Institute of Steel Construction
FE	Finite element
FEA	Finite element analysis
FEM	Finite element model
FEMA	Federal Emergency Management Agency
SAC	Scientific Advisory Committee
SCWP	Strong-column Weak-panel zone
WCSB	Weak-column Strong-beam
As	cross-sectional area
CAE	Computer-aided engeneering
Es	Young's modulus
FR	Fully restrained
f_y	The yield stress
f_u	the ultimate stress
HS	high strength
$K_{yy,}K_{zz}$	interaction factors
ki	the stiffness coefficient
$M_{\mathbf{ED}}$	Design moment
$M_{c,y,Rd}$,$M_{c,z,Rd}$	the in-plane bending resistances about the major and minor axes respectively

M-θ	Moment-rotation
M	Bending Moment
M_s	moment at service loads
$N_{\mathbf{ED}}$	Design axial force
Θ_s	rotation at service loads
θ_u	rotation capacity
PR	partially restrained
P_U	ultimate load
SLS	Serviceability Limit States
Sj.ini	initial rotational stiffness
ULS	Ultimate Limit States
μ	the stiffness ratio ($S_{j,ini}$ /S_j)
Δ	Displacement

GENERAL INTRODUCTION

- **GENERAL**

– Beam to column structures

Members subjected to flexure and axial forces are commonly identified as beam-columns. Beam-columns may act as if isolated, as in the case of eccentrically loaded compression members with simple end connections, or they may form part of a rigid frame. Beam-columns are structural members which combine the beam function of transmitting transverse forces or moments with the compression (or tension) member function of transmitting axial forces. Theoretically, all structural members may be regarded as beam-columns, since the common classifications of tension members, compression members, and beams are merely limiting examples of beam-columns. As far as beam-columns are concerned, fourtypesof structural elementsexist in any beam-to-column connection:beams,columns,a panelzone, andjoining mediathatconnectbeamstothecolumn.The stresses anddeformation demandattheconnectiondependstrongly ontherelativestrength, stiffness and ductility of each elements. Consequently, thesedifferent parameters will certainly affectthelocalbehaviouroftheconnection.

– Joints in steel MRFs structures

Despite these facts, the great majority of joints doesn't exhibit such idealized behaviour. In most steel frame designs the beam to column connections are assumed to be rigid or pinned. Rigid joints, where no relative rotation occurs between the connected members, transfer not only substantial bending moments, but also shear and axial forces. On the other extreme, pinned joints are characterized by almost free rotation movement between the connected elements that prevent the transmission of bending moments. A substantial effort has been made in recent years to characterize the behaviour of semi rigid connections. Most design codes included methods and formulas to determine both their resistance and stiffness. EC3 and EC4, for instance, allow the use of springs attached to the end of the beams at both sides of the joints.

– Semi-rigid in ended-plate connections

Two extreme cases regarding the actual performance of beam-to-column connections have been idealized in traditional analysis and design of steel frame structures. One extreme is known as rigid-joint connection while the other one is referred to as pinned-joint connection. Nevertheless, both of such idealized models do not accurately present the actual behaviour since most of the connections demonstrate a semi-rigid behaviour. Moreover, non-conservative predictions regarding the structural drift or frame stability could result from such approaches. Thus, real connections in steel frames should be treated as 'semi-rigid' ones. Semi-rigid connections are connections that have a dependable and known moment capacity intermediate in degree between the rigidity of rigid connections and the flexibility of simple shear connection.

– Monotonic loading behaviour of joints

Semi rigid connections are also designed like rigid connections, able to transfer moments and end reactions (shear and normal force). However, the capability to transfer moments at joints is less than rigid connections. Besides that, the member end joints are permitted to rotate but it is limited. The value of connection rotation is smaller than the simple connections. These connections are designed to provide a predictable degree of interactions between the members based on actual or standardized design moment- rotation ($M-\phi$) characteristic of the joints.

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.

– Cyclic behaviour of joints

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. 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. 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. The cyclic loading protocol is 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.

• MOTIVATION AND AIMS

Structural engineers have always tried to find new methods to improve the design and construction performance of steel and composite buildings in order to increase strength and reduce overall cost. Members subjected to flexure and axial forces are commonly identified as beam-columns. Beam-columns may act as if isolated, as in the case of eccentrically loaded compression members with simple end connections, or they may form part of a rigid frame. Despite these facts, the great majority of joints doesn't exhibit such idealized behaviour. The beam-column problem is generally approached from the standpoint of the Methods of the strength of materials, which drastically simplify the more precise Methods of the theory of elasticity and plasticity.

In most steel frame designs the beam to column connections are assumed to be rigid or pinned. Rigid joints, where no relative rotation occurs between the connected members, transfer not only substantial bending moments, but also shear and axial forces. On the other extreme, pinned joints are characterized by almost free rotation movement between the connected elements that prevent the transmission of bending moments. 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. 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 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 . Nonlinear study of the behaviour of semi-rigid connection in beam to column under both static monotonic and cyclic using ABAQUS is an interesting field involving lot of structural analysis fields and very good theoretical and code backgrounds.

- **OBJECTIVES**

Several objectives have been planned but with the lack of time given to achieve this work has reduce these objectives. Firstly, a validation of nonlinear 3D model implanted in ABAQUS of end-plate semi-rigid connection with some experimental and theoretical results taken from literature was successfully done. A parametric study was carried out to highlight the effect of some parameters by means of a finite element (FE).

- Literature review was done in order to deeply understand the structural behaviour of beam to column structures.
- Literature review was done in order to deeply understand the classification of joints and the concept of semi-rigid connections.
- The heavy task of learning ABAQUS software which takes some times on the first models.

- Modelling the first actual model to validate the experimental and theoretical results which has been successfully executed.
- Some design parameters effects on the global behaviour of the connection via numerical models.

- Then the most complicated task to evaluate the behaviour of cyclic response of a semi-rigid connection in a beam to column structure. This task was obviously proceeded by an intensive literature revue on the subject on the importance of the cyclic loadings on steel structure.

- **THESIS ORGANISATION**

This dissertation has been structured into a general introduction, five separate chapters, and conclusions as follows:

- A general introduction which provides a brief information on the work undertaken in this study and its objectives.

- Chapter 1: This chapter presents an overview on beam to column steel structures with the necessary theoretical background. The state of the art inevitably starts with an inventory with some relative recommendations on beam to column design of European and American codes are also presented.

- **Chapter 2:** This chapter is aimed at giving a brief account with the main characteristics of steel joints, their classification highlighting the concept of semi-rigid connection and end-plate types of connection. Once again, some relative recommendations on joints design of EC3 and AISC are also presented.

- **Chapter 3:** This chapter is divided into two main topics: First, a presentation of some generalities concerning ABAQUS program capabilities, which properly were used in this study. Secondly, a presentation of some studied cases under monotonic loadings of beam to column ended-plate connection are described and fully discussed implanted in ABAQUS software taking into account both nonlinearities. Some interesting conclusions have been drawn to highlight this first and preliminary.

- **Chapter 4:** Brief background on cyclic loading is provided. Loading Protocols, their physical significance and their several types are presented along with some theoretical background.

- Chapter 5: The objective of this chapter is to develop and describe numerical models that is able to simulate as accurately as possible the cyclic behaviour of bolt connection under AISC loading protocol. Lot of difficulties were encountered during the modelling. Several tries were necessary to find out the most representing model. The outcomes of such study were fully discussed.

- **Conclusions and suggestions for further work**: Main conclusions are drawn with some suggestions for further work.

CHAPTER 1

INTRODUCTION TO BEAM TO COLUMN

IN STEEL STRUCTURES

1.1 INTRODUCTION

The most general statements within the scope of theory of structures are obtained when the internal and external force and deformation variables are rigorously associated in the form of work-associated variables. Structural behaviour is expressed in the form of internal and external force and deformation variables: loads and stresses plus displacements and strains.

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 environmental loads, especially during earthquakes. Nonlinear structural analysis in civil engineering is not a new topic, but the existing method used for calculating the nonlinear behaviour of civil engineering structures is often by changing the structural member stiffness.

Due to the fabrication process, the nominal values of the yield strength f_y and also of the ultimate strength f_u depend on the thickness of the elements. It should be noted that different values for the relation between the material thickness and the material strength are given in the delivery condition specified in the product standards (for example EN 10015) and in EC 3.

1.2 GENERAL ON STEEL MEMBERS

Structural members can be classified as tension or compression members, beams, beam-columns, torsion members, or plates (Figure 1.1), according to the method by which they transmit the forces in the structure (figure 1.2). The behaviour and design of beam-column is summarised in this chapter, for full details, the reader is referred to references are **[Trahair et al 2008].**

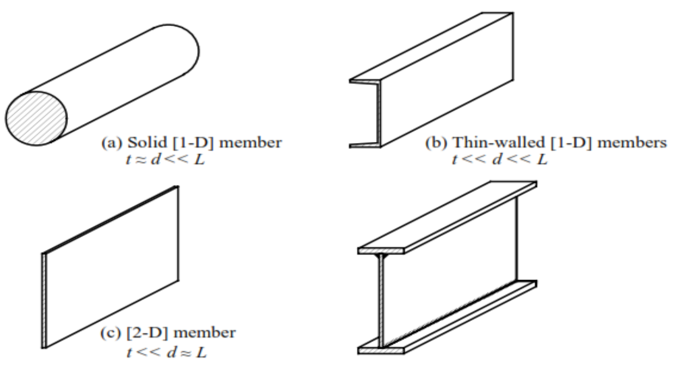


Figure 1.1 Types of structural steel members **[Trahair et al 2008].**

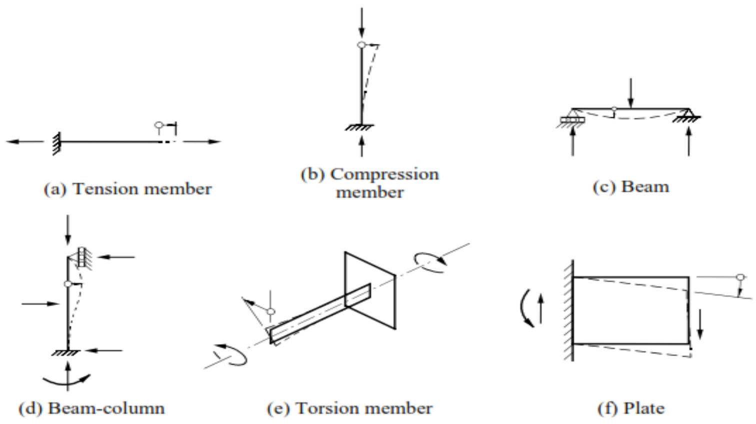


Figure 1.2 Load transmission by structural members **[Trahair et al 2008].**

1.3 MEMBER AND STRUCTURE BEHAVIOUR OF BEAM TO COLUMN

1.3.1 General

Beam-columns or beam to columns are structural members which combine the beam function of transmitting transverse forces or moments with the compression (or tension) member function of transmitting axial forces (Figure 1.3). Theoretically, all structural members may be regarded as beam-columns, since the common classifications of tension members, compression members, and beams are merely limiting examples of beam-columns.

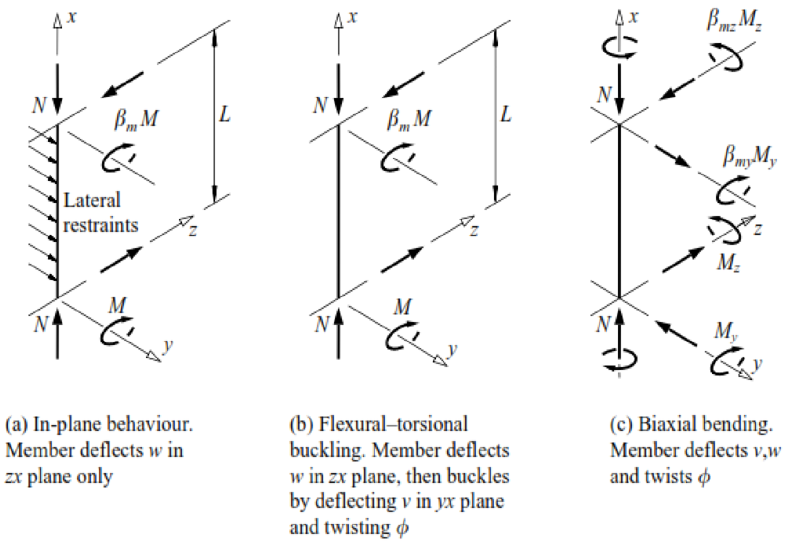


Figure 1.3 Beam-column behaviour **[Trahair et al 2008].**

1.3.2 Member behaviour

Structural steel members may be one-dimensional as for beams and columns (whose lengths are much greater than their transverse dimensions), or two-dimensional as for plates (whose lengths and widths are much greater than their thicknesses), as shown in Figure 1.1c. While one-dimensional steel members may be solid, they are usually thin-walled, in that their thicknesses are much less than their other transverse dimensions.

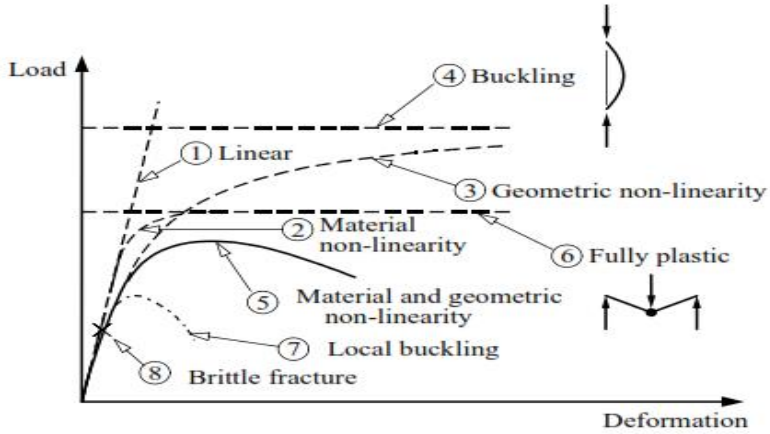


Figure 1.4 Member behaviour **[Trahair et al 2008].**

The member may have the linear response shown by **curve 1** in Figure 1.4, at least until the material reaches the yield stress. The magnitudes of the deformations depend on the elastic moduli *E* and *G*. Theoretically, a member can only behave linearly while the maximum stress does not exceed the yield stress f_y and so the presence of residual stresses or stress concentrations will cause early non-linearity. However, the high ductility of steel causes a local redistribution after this premature yielding, and it can often be assumed without serious error that the member response remains linear until more general yielding occurs.

The member behaviour then becomes non-linear **(curve 1)** and approaches the condition associated with full plasticity **(curve 6)**. This condition depends on the yield stress f_y.

The member may also exhibit geometric non-linearity, in that the bending moments and torques acting at any section may be influenced by the deformations as well as by the applied forces. This non-linearity, which depends on the elastic moduli *E* and *G*, may cause the deformations to become very large **(curve 3)** as the condition of elastic buckling is approached **(curve 4)**. This behaviour is modified when the material becomes non-linear after first yield, and the load may approach a maximum value and then decrease **(curve 5).**

The member may also behave in a brittle fashion because of local buckling in a thin plate element of the member **(curve 7),** or because of material fracture **(curve 8).** The actual behaviour of an individual member will depend on the forces acting on it. Thus, tension members, laterally supported beams, and torsion members remain linear until their material non-linearity becomes important, and then they approach the fully plastic condition. However, compression members and laterally unsupported beams show geometric non-linearity as they approach their buckling loads. Beam-columns are members which transmit both transverse and axial loads, and so they display both material and geometric non-linearities.

1.3.3 Structure behaviour

The behaviour of a structure depends on the load-transferring action of its members and joints. This may be almost entirely by axial tension or compression, as in the triangulated structures with joint loading. Alternatively, the members may support transverse loads, which are transferred by bending and shear actions. The load-transferring action of the members of a structure depends on the arrangement of the structure, including the geometrical layout, the joint details and on the loading arrangement.

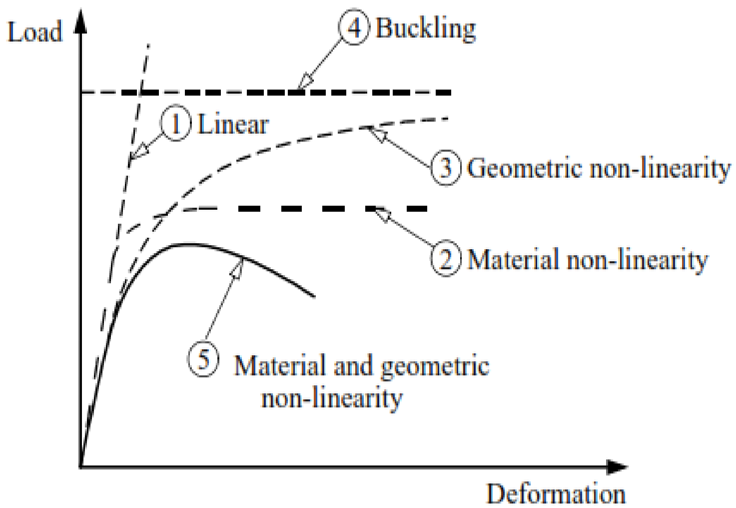


Figure 1.5 Structure behaviour **[EC3.2004].**

Most steel structures behave non-linearly near their ultimate loads, unless they fail prematurely due to brittle fracture, fatigue, or local buckling. This non-linear behaviour is due either to material yielding (**curve 1** in Figure 1.5), or member or frame buckling (**curve 4**), or both (**curve 5).** In axial structures, failure may involve yielding of some tension members, or buckling either of some compression members or of the frame, or both. In flexural structures, failure is associated with full plasticity occurring at a sufficient number of locations that the structure can form a collapse mechanism. In structures with both axial and flexural actions, there is an interaction between yielding and buckling (**curve 5** in Figure 1.4), and the failure load is often difficult to determine. The transitions shown in Figure 1.4 between the elastic and ultimate behaviour often take place in a series of non-linear steps as individual elements become fully plastic or buckle.

1.4 BEAM-COLUMN STRUCTURES

1.4.1 General

Beam-columns are structural members which combine the beam function of transmitting transverse forces or moments with the compression (or tension) member function of transmitting axial forces. Theoretically, all structural members may be regarded as beam-columns, since the common classifications of tension members, compression members, and beams are merely limiting examples of beam-columns.

Members subjected to flexure and axial forces are commonly identified as beam-columns. Beam-columns may act as if isolated, as in the case of eccentrically loaded compression members with simple end connections, or they may form part of a rigid frame.

They are frequently encountered in routine design when:

- the axial force is eccentric with reference to the cross-section centroid;
- the compressed element is also subjected to transverse load inducing flexure (typically, beams in simple frames loaded by gravity loads but also interested by axial forces due to the effects of horizontal forces);
- the vertical elements, which belong to a rigid or to a semi-continuous frame, are loaded at their ends by bending moments transferred by beams;
- thin-walled elements are subjected to axial load on the centroid of the gross section, which does not coincide with the one of the effective cross-sections.

‘The connections between members should be capable of withstanding the forces and moments to which they are subject without invalidating the design assumptions. If, for instance, the structure is designed in ‘simple construction’, the beam-column joint should be designed accordingly to accept rotations rather than moments. A rigid joint would be completely wrong in this situation, as it would tend to generate a moment in the column for which it has not been designed.

1.4.2 Theoretical background

The behaviour and design of beam–columns are covered thoroughly by Chen and Atsuta (1976).

1.4.2.1 General Considerations

The beam-column problem is generally approached from the standpoint of the Methods of the strength of materials, which drastically simplify the more precise Methods of the theory of elasticity and plasticity. Instead of taking a small volume as the elementary volume for investigating internal stresses and strains in a body, the methods of the strength of materials take full cross-sectional area of a beam-

Column segment of unit length as the basic building block.

The unit segment may be subjected to axial compression and bending moments. These loads are called generalized stresses of the problem. The corresponding generalized strains are elongation and curvatures. Solutions will be sought to the generalized strains and deflections, produced by generalized stresses. Hence the three-dimensional elasticity and Plasticity problem is reduced to a one-dimensional problem in a generalized sense.

Solutions for any beam-column problem, just as in the theory of elasticity and plasticity, must satisfy the equations of equilibrium in the generalized stress sense, the conditions of geometry or compatibility, and the generalized stress-strain relations. The equations of equilibrium and conditions of geometry are familiar and straightforward. The new element is the development of the relations between generalized stress and generalized strain. The methods of the strength of materials differ from the more precise methods of the theory of elasticity and plasticity in that a number of simplifying assumptions of a kinematic and geometrical nature must Be made. These simple but powerful assumptions then enable the forms for generalized stress vs. generalized strain, stress vs. generalized stress, and strain vs. generalized strain to be derived for a beam-column segment on the basis of uniformity along the unit length of the segment.

1.4.2.2 The Stability Problem

When a load is applied to a structure at a certain point, the structure will generally deform in such a manner that the load point moves in the general direction of the applied load. This is the deflection of the structure and in general the effect of this deflection upon the overall geometry can be ignored.

In regard to stability problems, two different approaches are commonly used in analysis. They furnish two different kinds of information, both valid and both important. All stability problems may be approached from the standpoint of deflection. This is known as load-deflection approach which attempts to solve a stability problem by determining its load-deflection behaviour throughout the entire range of loading including the descending (or unloading) branch of the curve. Calculations of deflections in the elastic-plastic range of materials are difficult in general, and numerical procedures are often necessary for a solution. Most part of the work described in this book attempts to solve the stability problem of beam-columns by this approach.

The general shape of the load-deflection curve of a stability problem is shown in Figure 1.6. It consists of an ascending branch OAC and a descending branch CD With a definite apex C which defines the maximum load-carrying capacity of the Member. This load is called stability limit load or stability load.

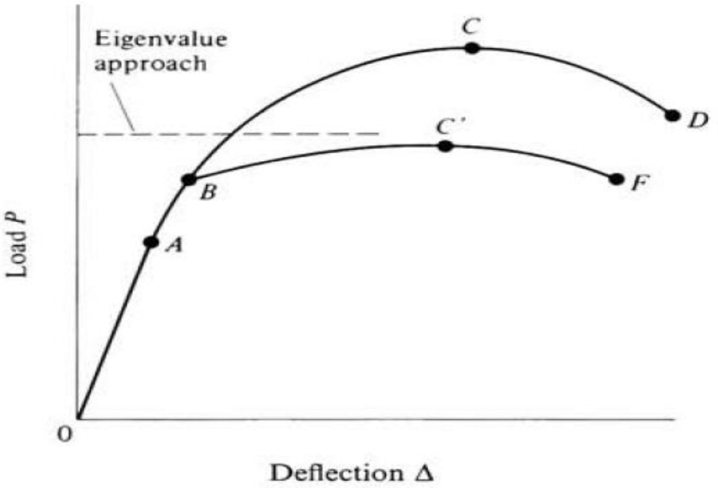


Figure 1.6 Load-deflection relationships for stability problems **[Chen and Atsuta 1976].**

The initial portion of the curve is approximately linear and the stiffness of line OA is almost a constant Value. When the deflection becomes larger, the curve starts to deviate from the linear Portion as the result of geometric change of the member and/or yielding of the Material. In some cases, loss of stiffness occurs suddenly as shown by the curve BC'F in Fig. 1.6. This is called stability problem with bifurcation or buckling. This Phenomenon may be caused also by the fracture of the material. There is a basic Difference however between the two failure phenomena. In the case of material Fracture, the curve OBC'F is the only equilibrium state for the system, while in the case of buckling. Both curves OBCF and OBCD represent equilibrium states. For a slender Compression member, there are two possible directions for a member to deflect at the Point B of Fig. 1.6. This point is called bifurcation point or bifurcation load. The Maximum load point C is known as the buckling load or more accurately, as the Post-buckling strength of the member.

1.5 GENERAL ON THE DESIGN PHILOSOPHY FOR BEAM-TO-COLUMN CONNECTIONS

1.5.1 Introduction

The task of the structural engineer is to design a structure, which satisfies the needs of the client and the user. Specifically, the structure should be safe, economical to build and maintain, and aesthetically pleasing. The engineering design process can often be divided into two stages:

(1) a feasibility study involving a comparison of the alternative forms of structure and selection of the most suitable type and

(2) a detailed design of the chosen structure **.**

As far as beam-columns are concerned, four types of structural elements exist in any beam-to-column connection: beams, columns, a panel zone, and joining media that connect beams to the column. The stresses and deformation demand at the connection depend strongly on the relative strength, stiffness and ductility of each elements. Consequently, these different parameters will certainly affect the local behaviour of the connection.

Structural design of steel beams and joists primarily involves predicting the strength of the member. This requires the designer to imagine all the ways in which the member may fail during its design life. It would be useful at this point, therefore, to discuss some of the more common modes of failure associated with beams and joints **.**

1.5.2 Types of design philosophies

In the design process of beam-column; different philosophies can be used:

1.5.2.1 Strong-Column Weak-Beam (SCWB) Design

In this design philosophy, the beams are detailed to be weaker than the adjoining columns and are designed to be the critical elements that undergo inelastic deformations (developing a plastic hinge) and the joining media (the connection) are called upon to transfer the beam bending strength with appropriate strain hardening to the column.

For a welded beam flange joint this implies the transfer of (a) very high horizontal normal stresses generated by axial yielding and strain hardening of the beam flange, (b) high shear stresses due to concentration of shear in the beam flange near the column face, and (c) possibly high normal stresses due to local bending of the beam flange in the region along the cope hole.

1.5.2.2 Weak-Column Strong-Beam (WCSB) Design

In this design philosophy, the columns are the critical elements that dissipate the earthquake input energy. This situation normally arises in low-rise buildings where the beam design is dominated by gravity loads and drift requirements while that of the column is governed by earthquake forces. Since the column is the weak element, it will develop a plastic hinge and the beam may remain in the elastic range even under severe earthquake loading. For a welded beam flange joint this implies very high vertical normal stresses and strains in the column flange near the weld root.

It is well known that weak-column-strong-beam (WCSB) frames concentrate the inelastic deformation into individual stories of a building and as a consequence seismic demands are much larger for these local parts of the structure.

1.5.2.3 Strong-Column Weak-Panel Zone (SCWP) Design

The beams, columns, and connections are the same as those for SCWB design philosophy. However, joint panel zones are designed to be the weakest elements in the frame (i.e. the vulnerable elements). Most of the inelastic action is expected to take place in the joint panel zones which have stable and ductile restoring force characteristics. If the panel zones are weak in shear compared to the bending strength of the beams, the excessive panel zone shear distortions (and eventually severe plastic hinging in columns) can have a very detrimental effect on the behaviour of welded beam flange joints. Therefore, protecting the weld from excessive beam stresses (i.e. moving the potential plastic hinge away from the beam end) may not be a solution to the weld fracture problem if panel zones or columns are the weak elements. Hence, the concept of sharing of inelastic deformations between beams and panel zones is a very desirable one and should be implemented whenever feasible. This concept cannot be implemented at an elastic force level even when factored loads are used. It needs to be implemented at the structure strength level and with estimates of the expected strengths of the elements and not nominal strengths.

Since the panel zone is the weak element and it yields in shear, neither the beam nor the column outside the panel zone may reach their yield bending strength and structure strength is controlled by a mechanism formed by plastification in the panel zones. This mechanism leads to a reduction in strength but not necessarily to a worsening in seismic performance. Its advantage is that it avoids the formation of story mechanisms.

1.6 DESIGN OF BEAM TO COLUMN TO EC3 AND AISC

1.6.1 Design According to the European Approach according to EC3

1.6.1.1 General

European provisions deal with the most common cases in design practice and in particular provide design rules for members with bi-symmetrical cross-sections. Rules to evaluate both strength and stability of some of the most common cases typical of routine design are discussed in the general part of EC3 (EN 1993-1-1).

Two different formats of the interaction formulae are provided in EN 1993–1–1, called Method 1 (Annex A of EN 1993–1–1) and Method 1(Annex B of EN 1993–1–1). The main difference between them is the presentation of the different structural effects, either by specific coefficients in Method 1 or by one compact interaction factor in Method 1. This makes Method 1 more adaptable to identifying and accounting for the structural effects, while Method 1 is mainly focused on the direct design of standard cases (Boissonade *et al*, 1006).

- **Method 1 (Annex A of EN 1993–1–1)** contains a set of formulae that favours transparency and provides a wide range of applicability together with a high level of accuracy and consistency.
- **Method 1 (Annex B of EN 1993–1–1)** is based on the concept of global factors, in which simplicity prevails against transparency. This approach appears to be the more straightforward in terms of a general format.

Both methods use the same basis of numerically calculated limit load results and test data for calibration and validation of the different coefficients. In this respect, the new interaction equations follow the format of those in the previous Eurocode 3 in principle. Both methods follow similar paths, namely the adaptation of the flexural-buckling formulae to lateral-torsional buckling by modified interaction factors calibrated using the limit-load results.

1.6.1.2 Design rules

- **Cross-section resistance**

EC3 requires beam-columns to satisfy both cross-section resistance and overall member buckling resistance limitations. The cross-section resistance limitations are intended to prevent cross-section failure due to plasticity or local buckling. The general cross-section resistance limitation of EC3 is given by a modification of the first yield condition of following equation:

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \tag{1.1}$$

in which N_{Ed} is the design axial force and M the design moment acting at the cross-section under consideration, $Nc_{Ed,Rd}$ (obtained using the effective cross-sectional area A is the cross-section axial resistance Af_y for slender cross-sections in compression), and $M_{c,Rd\ eff}$ is the cross-section moment resistance (based on either the plastic, elastic or effective section modulus, depending on classification).

- **Member resistance**

The general in-plane member resistance limitation of EC3 is given by a simplification of equations:

For major axis buckling

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy}\frac{M_{y,Ed}}{M_{c,y,Rd}} \leq 1 \tag{1.2}$$

For minor axis buckling,

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zz}\frac{M_{z,Ed}}{M_{c,z,Rd}} \leq 1 \tag{1.3}$$

In which k_{yy} and k_{zz} are interaction factors whose values may be obtained from Annex A or Annex B of EC3 and $M_{c,y,Rd}$ and $M_{c,z,Rd}$ are the in-plane bending resistances about the major and minor axes respectively. The former is based on enhancing the elastically determined resistance to allow for partial plastification of the cross-section, whilst the latter reduced the plastically determined resistance to allow for instability effects. Lengthy formulae to calculate the interaction factors are provided in both cases.

1.6.2 Design According to the US Approach

AISC 360-10 specifications address design rules for members subjected to flexure and axial force in its Chapter H. This chapter actually contains provisions for 'Design of members for Combined Forces and Torsion', therefore, its scope is more general. AISC 360-10 provides specific rules for the following cases:

(a) Doubly and singly symmetrical members subjected to flexure and compression.

(b) Doubly and singly symmetrical members subjected to flexure and tension.

(c) Doubly symmetrical rolled compact members subjected to single axis flexure and

Compression.

(d) Unsymmetrical and other members subjected to flexure and axial force.

(a) Doubly and singly symmetrical members subjected to flexure and compression.

1.7 RESISTANCE OF A BEAM COLUMN STRUCTURE

1.7.1 Cross section resistance

The cross- section resistance is based on its plastic capacity (class 1 or 1 sections) or on its elastic capacity (class 3 or 4 cross sections). When a cross section is subjected to bending moment and axial force ($N + M_N + Mz$ or even $N + My + M$), the bending moment resistance should be reduced, using interaction formulas. The interaction formulae to evaluate the elastic cross section capacity are the well-known formulae of simple beam theory, valid for any type of cross section. However, the formulae to evaluate the plastic cross section capacity are specific for each cross-section shape.

For a cross section subjected to $N + M$, a general procedure may be established to evaluate the plastic bending moment resistance M, reduced by the presence of an axial force N. This method, applied to a cross section with a generic shape and gross area A, composed by a material with a yield strength f_y involves the definition of an area and A are equal to:

$$A_c = N / f_y \tag{1.4}$$

$$(A_1=A_2=(A-N/f_v)/2\ ) \tag{1.5}$$

The reduced plastic bending moment resistance $M_{11\,y}\ N_{,Rd}$ is given by the product of the force $Ft = A_1.\ f_y$ equal to $Fc = A_1.\ f_y$ and the distance d between the centroid of areas $Ayt11$ and A, as shown in Figure 1.7.in compression, located in such a way that the areas $A\ y$ (Figure 1.7).

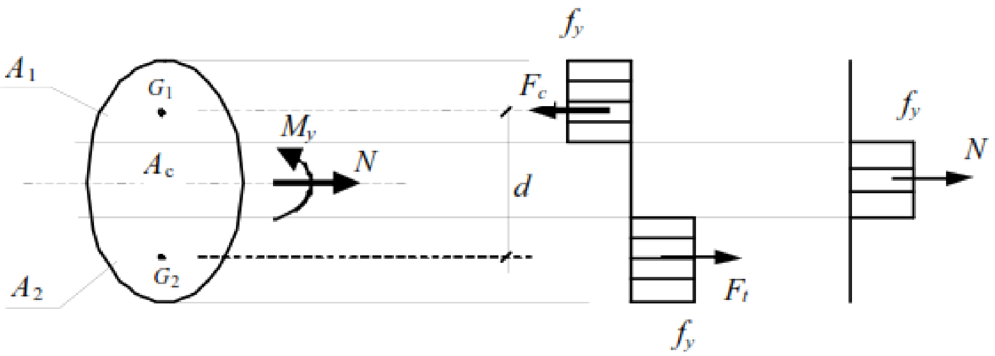


Figure 1.7 Bending moment-axial force plastic interaction (**Simoes et al. 2016)**

Members subjected to bi-axial bending and axial compression (beam–columns) exhibit complex structural behaviour. First-order bending moments about the major and minor axes (M and M, respectively) are induced by lateral loading and/or end moments. The addition of axial loading $N_{z,Ed\ b;Rd}$ clearly results in axial force in the member, but also amplifies the bending moments about both principal axes (second-order bending moments). Since, in general, the bending moment distributions about both principal axes will be non-uniform (and hence the most heavily loaded cross-section can occur at any point along the length of the member), plus there is a coupling between the response in the two principal planes, design treatment is necessarily complex.

1.7.2 Fully plastic beam-columns

An upper bound estimate of the resistance of an I-section beam-column bent about its major axis can be obtained from the combination of bending moment M and axial force N which causes the cross-section to become fully plastic. A particular example is shown in Figure 7.8, for which the distance $z\ y,r$ from the centroid to the unstrained fibre is less than $(h - 1tf\ n)/1$.

This combination of moment and force lies between the two extreme combinations for members with bending moment only (N = 0), which become fully plastic at:

$$M_{pl} = f_y b_f t_f (h - t_f) + f_y t_w \left(\frac{h - 2t_f}{2}\right)^2 \tag{1.6}$$

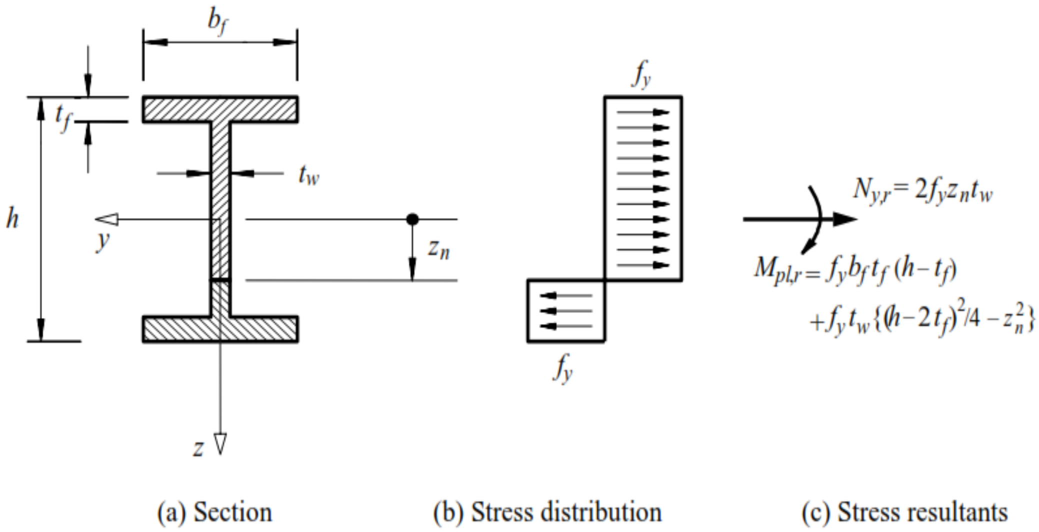


Figure 1.8 Fully plastic cross-section **[Trahair et al 2008].**

An analysis may be made of an I-section beam-column bent about its minor axis. In this case, a satisfactory approximation for the load and moment at full plasticity is given by :

$$\frac{M_{pl,r,z}}{M_{pl,z}} = 1.19\left[1 - \left(\frac{N_{y,r}}{N_y}\right)^2\right] \leq 1.0. \tag{1.7}$$

1.7.3 Ultimate resistance

1.7.3.1 General

An isolated beam-column reaches its ultimate resistance at a load which is greater than that which causes first yield, but is less than that which causes a cross-section to become fully plastic as developed in the previous section. These two bounds are often far apart, and when a more accurate estimate of the resistance is required, an elastic–plastic analysis of the imperfect beam-column must be made. Two different approximate analytical approaches to beam-column strength may be used, and these are related to the initial crookedness and residual stress methods allowing for the effects of imperfections on the resistances of real compression members. Headlines are of the two approaches are given below, details on these methods can be found in **[Trahair et al 2008].**

- **Elastic–plastic resistances of straight beam-columns**

The resistance of an initially straight beam-column with residual stresses may be found by analysing numerically its non-linear elastic–plastic behaviour and determining its maximum resistance.

- **First yield of crooked beam-columns**

In the second approximate approach to the resistance of a real beam-column, the first yield of an initially crooked beam-column without residual stresses is used. For this, the magnitude of the initial crookedness is increased to allow approximately for the effects of residual stresses. One logical way of doing this is to use the same crookedness as is used in the design of the corresponding compression member, since this has already been increased so as to allow for residual stresses.

1.8 MODELLING OF BEAM-TO-COLUMN JOINTS / CONNECTIONS

1.8.1 General

The methods for predicting the beam-to-column joint behaviour can be divided into:

1. Mathematical (analytical) models or curve fitting.

2. Finite element models.

3. Mechanical models.

Broadly speaking, each of the different techniques of modelling the actual response of the connections has its advantages and drawbacks. As the aim of the joint modelling is to account for the joint rotational behaviour in structural analysis, it is evident that the prediction of the joint behaviour by means of one of the above methods has to be generally accompanied by a mathematical representation of the moment-rotation curve which is necessary to be used as input data in computer programs for the structural analysis of semi-rigid frames.

1.8.2 General on the mathematical Models

Mathematical models are based on the curve fitting so that they are able to represent only the cyclic behaviour of beam-to-column joints for which an experimental test is available. The range of application of such models is, therefore, limited to the structural details tested. In other words, the mathematical model is just a tool to account for the actual cyclic behaviour in estimating the seismic inelastic response of structures.

1.8.3 Finite Element Models

According to FEMA-355C, 2000 h and k, modelling and analysis of the actual behaviour of a joint thorough a finite element idealization can be done. FE modelling presents several advantages, that is:

1. It can be conducted to solve both linear and nonlinear problems.

2. It can handle any system that has complicated geometrical shape.

3. The boundary conditions can be managed easily. However, FE method suffers of many disadvantages such as:

- It is time consuming for complicated structures and very expensive in terms of effort and is therefore not suitable for a design procedure.
- It needs high skills and experience to produce model which accurately represents the actual structure.
- It needs high performance computers.

CHAPTER 2

MOMENT RESISTING JOINTS

BOLTED JOINTS

2.1 INTRODUCTION

Building frames consist of beams and columns, usually made of H or I shape that assemble together by means of connections showed in Figure **2.**1. These connections are between two beams, two columns, a beam and a column or a column and the foundation **[Ivanyi et al 2000].**

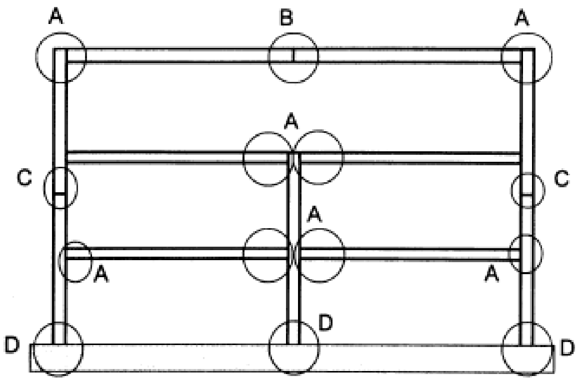


Figure 2.1 Different types of connections in a building **[Ivanyi et al 2000].**

The design of joints may, as for any other cross section, be performed on an elastic or a plastic basis. In a pure elastic approach, the joints should be designed in such a way that the generalised Von Mises stress nowhere exceeds the elastic strength of the constitutive materials. To achieve it, and because of the geometrical complexity of the connection elements, a refined stress analysis will be required, often requiring sophisticated numerical approaches like FEM ones. In this process, the presence of residual stresses or of any other set of self-equilibrated stresses which could, for instance, result from lack-of-fit, is usually neglected, but would have normally to be integrated. As a result, the elastic resistance of the joint may be evaluated but generally appears to be relatively low.

2.2 THE CONCEPT OF JOINT REPRESENTATION

During many years, the research activity in the field of joints mainly concentrated on two aspects:

- The evaluation of the mechanical properties of the joints in terms of rotational stiffness, moment resistance and rotation capacity;
- The analysis and design procedures for frames including joint behaviour.

But progressively it has been understood that there were intermediate steps to consider in order to integrate in a consistent way the actual joint response into the frame analysis; this is known as the joint representation.

The joint representation includes four successive steps respectively named:

- The joint characterisation i.e. the evaluation through appropriate means of the stiffness, resistance and ductility properties of the joints (full M-θ curves or key values).
- The joint modelling i.e. the way on how the joint is physically represented in view of the frame analysis.
- The joint classification i.e. the tool providing boundary conditions for the use of conventional types of joint modelling (e.g. rigid or pinned).
- The joint idealisation i.e. the derivation of a simplified moment-rotation curve so as to fit with specific analysis approaches (e.g. linear idealisation for an elastic analysis).

2.3 JOINTS OR CONNECTIONS

2.3.1 General

Connections or joints are used to transfer the forces supported by a structural member to other parts of the structure or to the supports. They are also used to connect braces and other members which provide restraints to the structural member. Although the terms connections and joints are often regarded as having the same meaning, the definitions of EC3-1-8 are slightly different, as follows:

A *connection* consists of fasteners such as bolts, pins, rivets, or welds, and the local member elements connected by these fasteners, and may include additional plates or cleats.

A *joint* consists of the zone in which the members are connected, and includes the connection as well as the portions of the member or members at the joint needed to facilitate the action being transferred.

2.3.2 Definition Moment-resisting joints according to EC3

Moment-resisting joints is a general term used to cover all joints which transfer significant bending moments between the connected members, but also shear and/or axial forces. These joints may be rigid or semi-rigid, in terms of stiffness, and may exhibit a full or a partial strength resistance level. The evaluation of their mechanical design properties is a key aspect to which several pages of EN 1993-1-8 are devoted. As already said, it is based on the application of the component method, that requires the mechanical properties of the active components to be computed before the components are assembled, to derive the global response of the whole joint in a specific loading situation. The assembly procedure problem is addressed, successively in terms of resistance, stiffness and deformation capacity.

The application of the component approach to specific joints will be presented. The following joint configurations are considered:

- Steel beam-to-column, beam-to-beam and beam splices;
- Steel column splices;
- Column bases.

2.3.3 Design of joints

Generally speaking, the process of designing building structures has been up to now made up of the following successive steps:

- Frame modelling including the choice of rigid or pinned joints;
- Initial sizing of beams and columns;
- Evaluation of internal forces and moments (load effects) for Ultimate Limit States (ULS) andServiceability Limit States (SLS);
- Design checks for ULS and SLS criteria for the structure and the constitutive beams andcolumns;
- Iteration on member sizes until all design checks are satisfactory;
- Design of joints to resist the relevant member end forces and moments (either those calculated, or the maximum ones able to be transmitted by the actual members); the design is carried out in accordance with the prior assumptions (frame modelling) on joint stiffness.

This approach was possible since designers were accustomed to considering the joints to be either pinned or rigid. In this way, the design of the joints became a separate task from the design of the members. Indeed, joint design was often performed at a later stage, either by other members of the design team or by another company.

Recognising that most joints have an actual behaviour which is intermediate between that of pinned and rigid joints, EN 1993 offers the possibility to account for this behaviour by opening up the way to what is presently known as the semi-continuous approach. This approach offers the potential for achieving better and more economical structures.

The basic elements to compose joints in steel structures are mechanical fasteners like bolts or pins. Chapter 2 of Eurocode 3 Part 1-8 provides design rules for such mechanical fasteners.

In steel construction, the most typical mechanical fasteners to connect plates or profiles are bolts, or more precisely: bolt assemblies (sets) including the bolt itself, a nut and one or more washers. The bolts may be preloaded to improve serviceability performance or fatigue resistance. Joints made with preloaded bolts normally may have a slightly higher stiffness, but this effect is not taken into account in the design rules. However, preloading requires a controlled tightening which leads to additional work during erection.

2.4 CONNECTIONS CLASSIFICATION

Two extreme cases regarding the actual performance of beam-to-column connections have been idealized in traditional analysis and design of steel frame structures. One extreme is known as rigid-joint connection while the other one is referred to as pinned-joint connection. Nevertheless, both of such idealized models do not accurately present the actual behaviour since most of the connections demonstrate a semi-rigid behaviour. Moreover, non-conservative predictions regarding the structural drift or frame stability could result from such approaches. Thus, real connections in steel frames should be treated as 'semi-rigid' ones.

In both specifications for structural steel buildings, ANSI/AISC 360-10 (ANSI/AISC 360-10 2010) and Eurocode 3 Part 1-8 specification (Eurocode 3 - Part 1-8 2005), three types of connections are classified: Type 1-rigid connection; Type 2-simple connection; and Type 3-semi-rigid connection, Figure 2-2 shows the comparison between connection types **[Faridmehr et al.2016]**.

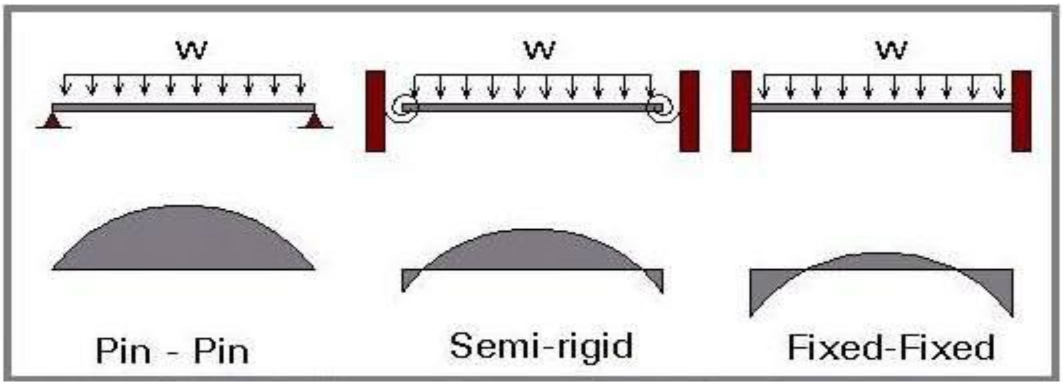


Figure 2.2 Comparison of Semi-Rigid Connections vs. Pinned and Fixed Connections

[Faridmehr et al.2016].

The fundamental criteria considered in categorizing connections is that the most significant behavioural appearances are exhibited by a moment-rotation (M-θ) curve. From this point of view, such classifications directly explain strength, stiffness and ductility of connections. The secant stiffness, KS, at service load is considered as an index property of connection stiffness (ANSI/AISC 360-10 2010),

$$K_S = M_S/\theta_S \qquad \textbf{(2.1)}$$

Where,

M_s= moment at service loads, (kn-m)

Θ_s= rotation at service loads, rad

Devising criteria suitable for serviceability and ultimate limit states design is regarded as one of the main difficulties in the provision of a classification system. For serviceability, deformation and other stiffness-related characteristics of the connections are known to be the prime considerations. Yet, in the case of ultimate limit states, strength parameters would be the major considerations. The maximum moment developed by a connection, Mn, is known as the strength of a connection Figure 2.3. Ductility, maximum rotation capacity, θu, and energy absorption are believed to be the critical factors for structures located in seismic areas, however.

The classification systems of connections have been presented by many authors: Bjorhovde et al. (1990), Goto et al. (1998), Nethercot et al. (1998), Eurocode 3 (CEN, 2005), and ANSI/AISC 360-10(ANSI/AISC 360-10 2010) **[Chen et al.2011]**.

In prior studies, researchers defined the connection classification index, which was mainly extracted from moment-rotation (*M-θ*) curves. The findings from these studies make major contributions to the current ANSI/AISC 360-10, and Eurocode 3 Part 1-8 specification. However, some differences exist among these two specifications in terms of connection classification schemes, although the findings are somewhat contradictory. Literature reviews indicated that there are no controlled studies that compare connection classification criteria between these two specifications. The current study attempts to investigate an adequate beam to column connection classification index from ANSI/AISC 360-10 (ANSI/AISC 360-10 2010), and Eurocode 3 Part 1-8 specification (Eurocode 3 -part 1-8 2005) through test results of FEP for semi-rigid connections**[Faridmehr et al.2016].**

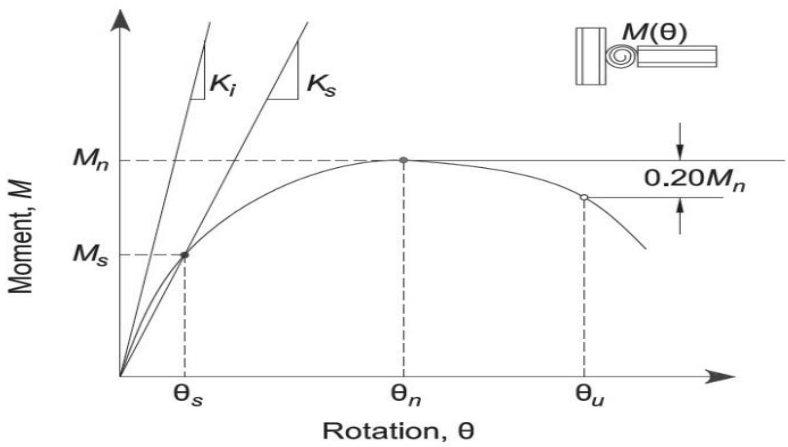


Figure 2.3 Strength, stiffness and ductility characteristics of the moment-rotation response of a partially restrained connection. **[Faridmehr et al.2016]**

- **Eurocode 3 part1-8 Classification System [Faridmehr et al.2016]**

In this specification, connections are classified by their stiffness and strength. A joint may be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness, by comparing its initial rotational stiffness, Sj.ini, with classification boundaries is given in Figure 2.4. Beam-to-column connections categorized as fully-rigid are supposed to have adequate rotational stiffness to consider analyses based on fully-rigid. In Figure 2.4, zone 1 represents as rigid connection and defined as in Equation (2):

$$S_{j.i.n.i} \geq \frac{K_b E I_b}{L_b} \qquad \textbf{(2.2)}$$

Where,

Kb is taken as 8 for structures with lateral displacement of frames by at least 80%

Kb is taken as 25 for other frames

A nominally pinned joint should be capable of transmitting the internal forces, without developing significant moments that might adversely affect the members or the structure as a whole. According to Figure 2.4, connections are considered as nominally pinned, zone 3, if:

$$S_{j.i.n.i} \leq \frac{0.5\, E I_b}{L_b} \qquad \textbf{(2.3)}$$

The beam-to-column connections that do not address the criteria for FR connections or a simple connection shall be classified as a partially restrained (PR) or semi-rigid connections, zone 2. PR connections provide an anticipated deformation between connected members, based on the (M-θ) curve features of the connections. PR connections are supposed to convey the shear forces as well as bending moments.

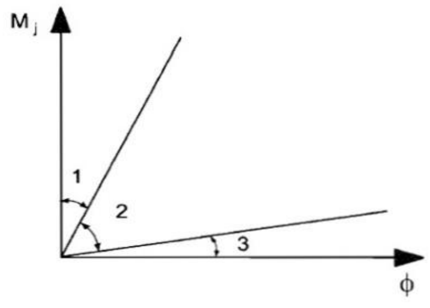


Figure 2.4 Classification of joints by stiffness according to Eurocode 3.

basic components; each is represented by an elastic stiffness coefficient, ki. The initial rotational stiffness, Sj.ini, of a beam-to-column connection may be calculated with sufficient accuracy from:

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{K_i}} \qquad \textbf{(2.4)}$$

Where,

kiis the stiffness coefficient for basic joint component i;

Z is the lever arm

μ is the stiffness ratio ($S_{j,ini}$ /S_j)

Notice that the initial rotational stiffness, $S_{j,ini}$, of connections is given by expression (2.4) with

μ = 1. The basic components that should be taken into account for bolted end-plate connection are given in Table 2.1 in accordance with EC3.

Table 2.1: Basic components of end-plate connections **[Faridmehr et al.2016]**.

Beam to Column Connection with Bolted End-plate Connections	Number of Bolt-rows in Tension	Stiffness Coefficient ki to be Taken into Account
Single-side	One	K1 ; k2; k3 ; k4 ; k5 ; k10
	Two or more	K1 ; k2; keq

A joint may be classified as full-strength, nominally pinned or partial strength by comparing its design moment resistance, Mn, with the design moment resistances of the members that it connects. A joint is categorized as simple if its design moment resistance, Mn, is not higher than 0.25 times the design moment resistance required for a fully-rigid connection and also addresses the adequate rotation capacity. The design capacity of a fully-rigid connection should not be less than the connected beam. A connection is classified as FR connections if it addresses the following equation:

$$M_n \geq M_p \qquad \textbf{(2.5)}$$

Where, M_p is the design plastic moment resistance of the beam.

- **ANSI/AISC 360-10 Classification System [Faridmehr et al.2016]**

Connection classification by AISC specification is conducted through modelling the most significant behavioural characteristics of the connection using a moment-rotation (M-θ) curve.

According to the AISC guideline, the (*M-θ*) curve is defined as being a part of the column and beam as well as the connecting components. This is because the connection rotation in a physical test is basically identified over a length that takes not only the connecting elements contributions, but also connected beam and the column shear panel zone. In general, based on AISC specifications, such classifications explain stiffness, strength and ductility of the connections.

The secant stiffness, KS, at service loads is considered a fundamental criterion of the connection stiffness as defined in Equation (2.1).

- If $K_sL/EI \geq 20$, the connection is considered to be fully-rigid or FR connections (be able to preserve the rotation between members).
- If $K_sL/EI \leq 2$, the connection is classified as simple (it rotates without increasing moment).
- If the connection stiffness is between these two boundaries than it will be categorised as a partially restrained or semi-rigid connection, and the strength, stiffness and ductility of the connection should be taken into account in the analysis.

The maximum moment can be carried out by connection introduced as Mn, as shown in Figure2.5. If the (M-θ) curve response does not demonstrate a peak moment, the moment at a rotation of 0.02 rad is considered the maximum strength of connection. Connections that convey less than 20% of the plastic moment of the connected beam, Mp, at a rotation of 0.02 rad, is supposed to have no flexural capacity for analysis. It is worth mentioning that for an FR connection, strength less than the beam strength is anticipated. Yet, it is also probable for a PR connection to provide a moment capacity higher than the connected beam.

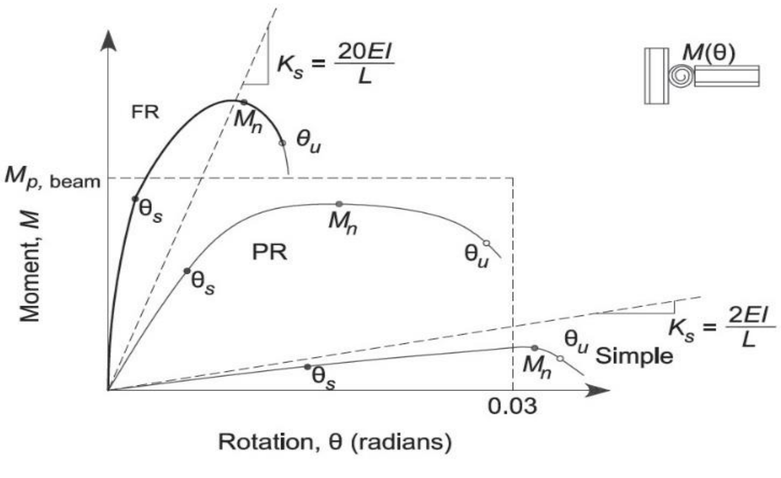


Figure 2.5 Classifications of moment-rotation response of fully restrained (FR),

partially restrained (PR) and simple connections based on strength (ANSI/AISC 360-10 2010).

[Faridmehr et al.2016]

The required ductility of a connection is a function of its application. For instance, a lower ductility is required for a structure in a low-seismic zone compared to those located in high seismic zones. The structural system plays an important role in rotation ductility requirements for seismic design. Once the connection strength capacity considerably exceeds the plastic moment of the connected beam, the ductility of the whole structure is measured by the beam, and the connection is supposed to remain elastic. The connection may experience severe inelastic deformation where its strength capacity is marginally higher than the plastic moment of the connected beam. However, deformations may concentrate within the connection component if the beam flexural capacity surpasses the connection strength. According to Figure 8, the rotation capacity, θ_u, identified as the particular point where either the resisting moment has decreased to 0.8Mn or the connection has experienced deformation beyond 0.03 rad. This second principle is reliable for connections with no obvious decrease in strength capacity until a very large deformation occurs. An evaluation should be made among the rotation resistance, θ_u, and the required rotation strength where a 0.03 rad rotation resistance is considered acceptable. This amount is equal to the minimum connection capacity in conformity with seismic provisions for special moment frames.

2.5 SEMI-RIGID CONCEPT

2.5.1 General

Semi-rigid connections are connections that have a dependable and known moment capacity intermediate in degree between the rigidity of rigid connections and the flexibility of simple shear connection **[El-Abidi .2012]**.The in-plane behaviour of connections can be represented by the moment-rotation (M–θ_r) curves illustrated in Figure 2.6.

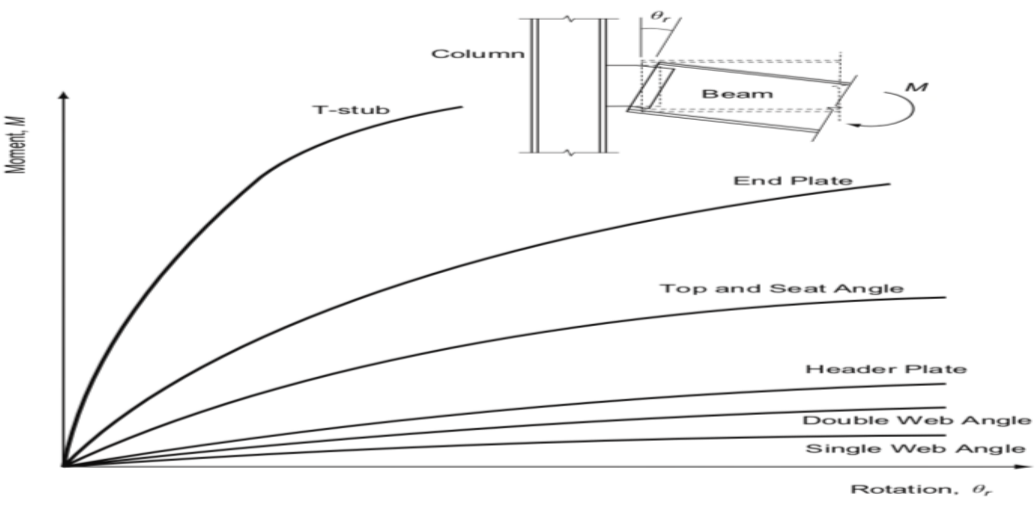


Figure 2.6 Moment-rotation curves of semi-rigid connections **[Chen et al.2011].**

Semi rigid connections are also designed like rigid connections, able to transfer moments and end reactions (shear and normal force). However, the capability to transfer moments at joints is less than rigid connections. Besides that, the member end joints are permitted to rotate but it is limited. The value of connection rotation is smaller than the simple connections. These connections are designed to provide a predictable degree of interactions between the members based on actual or standardized design moment- rotation ($M - \phi$) characteristic of the joints**[El-Abidi 2012].** (Figure 2.7) shows an example of semi rigid connection. Most of the present regulation codes regarding constructional steelwork allow for the semi-rigid concept as model for connection behaviour [**Bojrhovde et al .1995].**

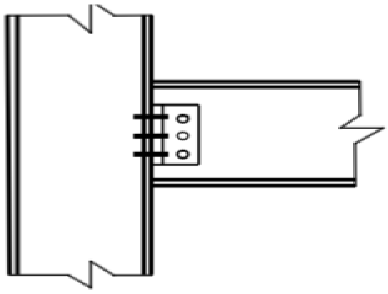


Figure 2.7 Semi rigid connection **[El-Abidi .2012]**.

2.5.2 Historical background

In 1917, Wilson and More were the first who published their results of riveted joints tests. Following this research was conducted in the thirties in Great Britain (Batho and Rowan -bolted connections, 1930), Canada (Young and Jackson-welded a riveted connection, 1934) and USA (Rathbun -riveted joints, 1936). It was observed that M-Φ relationship was nonlinear. Attempt was made, according to calculating possibilities at that time, to consider the flexibility of joints in structural analysis.

Batho and Rowan (1931) presented a graphical method for predicting the moment in a joint, in the case when the M-Φ curve is known (beam-line method). Figure 2.8 shows the nature of this method. Point P in which experimentally obtained M-Φ curve intersects the line connecting fixed moment in beam M_F and rotation Φ_w of simply supported beam, indicates bending moment and rotation in the joint. Modification of the classical methods: slope-deflection and moment-distribution were proposed by Baker and Rathbun in 1936.These methods did not find application in design practice because of poor calculation tools at the time **[Kozlowski.1996].**

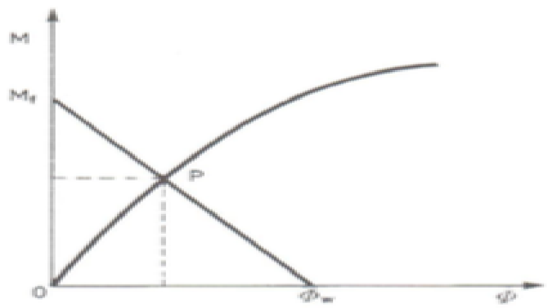


Figure2.8 Beam-line method **[Kozlowski.1996].**

In 1970, a semi-rigid composite connection was first proposed by Barnard. He continued some of the slab reinforcement across the column with enough shear studs to ensure full composite connection. Since then, extensive research has been carried out to investigate the behaviour of isolated semi-rigid composite connections and the effect of a composite connection on the behaviour of composite structures. (Figure2.9) shows an example of a semi-rigid composite connection**[AL-aasam.2013].**In 1970-s together with computer development, matrix stiffness method and FEM were established in the analysis of frames. From this time, many experimental tests were conducted and joint behaviour models were developed by various researchers all over the world.

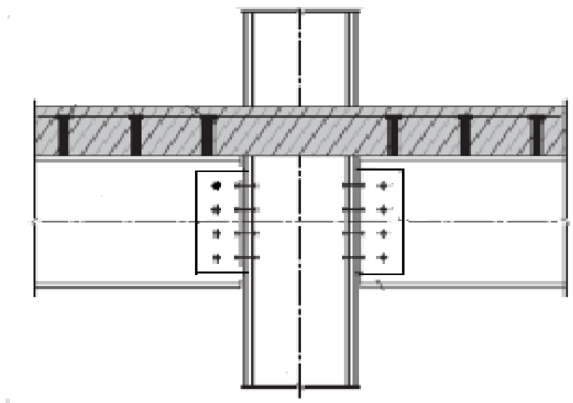


Figure 2.9 Example of a semi-rigid composite connection**[Kozlowski.1996].**

2.6 TYPES OF JOINTS

2.6.1 Introduction

The arrangement of a joint is usually chosen to suit the type of action (force and/or moment) being transferred and the type of member (tension or compression member, beam, or beam-column) being connected. The arrangement should be chosen to avoid excessive costs, since the design, detailing, manufacture, and assembly of a joint is usually time consuming; in particular the joint type has a significant influence on costs. For example, it is often better to use a heavier member rather than stiffeners since this will reduce the number of processes required for its manufacture.

2.6.2 General about bolted-joints

A joint is designed by first identifying the force transfers from the member through the components of the joint to the other parts of the structure. Each component is then proportioned so that it has sufficient strength to resist the force that it is required to transmit. General guidance on joints. The main information given below is taken out from **[Bickford 2008].**

Bolted joints come in two flavours, depending on the direction of the external loads or forces acting on the joint. If the line of action of the forces on the joint is more or less parallel to the axes of the bolt, the joint is said to be loaded in tension and is called a tension or tensile joint. If the line of action of the load is more or less perpendicular to the axes of the bolt, the joint is loaded in shear and is called a shear joint.

2.6.3 Categories of joints

The distinction between tensile and shear joints is important, because the two types differ in the way they respond to loads, the ways in which they fail, the ways in which they are assembled, etc.

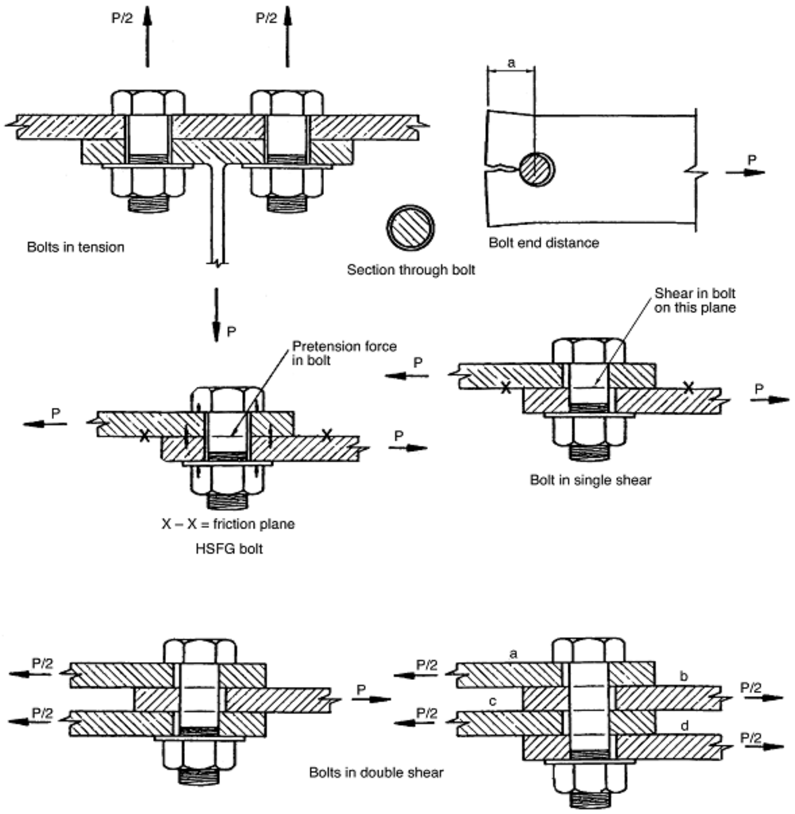


Figure 2.10 Type of bolted joints **[Bangash .2000].**

In general, the tensile joint is the more complex of the two as far as behaviour and failure are concerned and it's the more common type of joint. Figure 2.10 illustrate some types of joints.

The purpose of a bolt or group of bolts in all tensile and in most shear- joints is to create a clamping force between two or more things, which we'll call joint members. In some shear joints the bolts act, instead, primarily as shear pins, but even here some bolt tension and clamping force is useful, if for no other reason than to retain the nuts.

- **Tensile joints**

Specifically, in tensile joints, the bolts should clamp the joint members together with enough force to prevent them from separating or leaking. If the joint is also exposed to some shear loads, the bolts must also prevent the joint members from slipping.

Coincidentally, the tension in the bolt must be great enough to prevent it from self-loosening when exposed to vibration, shock, or thermal cycles. High tension in the bolt can also make it less susceptible to fatigue (but sometimes more susceptible to stress cracking). In general, however, we usually want the bolt in a joint loaded in tension to exert as much force on the joint as it and the joint members can stand.

There are two important facts you should keep in mind when dealing with tension joints. First, the bolt is a mechanism for creating and maintaining a force, the clamping force between joint members.

Second, the behaviour and life of the bolted joint depend very much on the magnitude and stability of that clamping force.

- **Shear joints**

The bolt's main job in a shear joint is to keep the joint from slipping or from tearing apart in the slip direction. If the joint must also support some tensile load, the bolt must resist that too. In some shear joints, as already mentioned, the bolts resist slip by acting as shear pins, and joint integrity is determined by the shear strength of the bolts and joint members. There are a number of reasons why we will often want to tension these bolts, as we'll see, but the exact amount of tension, or of the energy stored in them, is not a critical factor. In other shear-loaded joints, slip is prevented by friction restraint between joint members. These friction forces are created by the clamping load, which in turn is created by heavily tensioned bolts. Here again, therefore, the bolt is a mechanism for creating and maintaining a force, and the magnitude and life of that force depend on the potential energy stored in the bolts during assembly. Even here, however, we're usually less concerned about creating an exact amount of tension in the bolts during assembly than we are when we're dealing with tensile joints, because service loads don't affect bolt tension and clamping force in other shear-loaded joints, slip is prevented by friction restraint between joint members. These friction forces are created by the clamping load, which in turn is created by heavily tensioned bolts. Here again, therefore, the bolt is a mechanism for creating and maintaining a force, and the magnitude and life of that force depend on the potential energy stored in the bolts during assembly. Even here, however, we're usually less concerned about creating an exact amount of tension in the bolts during assembly than we are when we're dealing with tensile joints, because service loads don't affect bolt tension and clamping force in shear joints.

- **Failure modes**

The main reason we want to control or predict the results of the assembly process and the in-service behaviour of the joint is to avoid joint failure. This can take several forms. A joint will obviously have failed if its bolts self-loosen, shake apart, or break. Self-loosening is a complicated process. In general, however, it's caused by vibratory or other cyclical shear loads which force the joint members to slip back and forth. A major cause of self-loosening is too little preload, and hence too little clamping force. Both tensile and shear joints are subject to this common mode of failure.

Bolts in both types of joints can also break because of corrosion, stress cracking, or fatigue all of which are also covered in later chapters and two of which are encouraged by the wrong preload. Stress cracking occurs when bolts are highly stressed; fatigue is most apt to occur when there's too little tension in the bolts. Even corrosion can be indirectly linked to insufficient preload, if a poorly clamped joint leaks fluids that attack the bolts. If the bolts fail for the reasons just cited or if they exert too little force on the joint, perhaps because of the assembly or in-service conditions discussed earlier, the shear joint may slip or the tension joint may separate or leak. Each of these things means that the joint has failed.It's obvious that a leak is a failure, but what's wrong with a little slip or with separation of a joint that doesn't contain fluid? Slip can misalign the members of a joint supporting shear loads, thereby cramping bearings in a machine, for example. Or it can change the way a structure absorbs load, perhaps overstressing certain members, causing the structure to collapse. Slip can lead to fretting corrosion or to fatigue of joint members. As already mentioned, cyclical slip can lead to self-loosening and perhaps loss of the fasteners. Vibration loosening of bolts and fatigue failure of shear joint members are of particular concern to airframe designers.

Separation of the members of a joint supporting tensile load can encourage rapid fatigue failure of the bolts. It can also destroy the integrity of a structure or machine. It can allow corrosion to attack bolts and joint surfaces. Separation means the total absence of clamping force, which means, in effect, that the joint is not a joint at all. Note that most joint or bolt failure modes are encouraged by insufficient bolt tension or insufficient clamping force or both. Self-loosening, leakage, slip, separation, fatigue all imply too little clamp.

N.B. A few problems can be caused by too much tension or clamping force, however. Stress corrosion and hydrogen embrittlement cracking of bolts can occur in both shear and tensile joints and are more likely if bolt stresses are high. Fatigue life can sometimes be shortened by high stress, although more commonly it's caused by insufficient clamping force. But failures caused by too little clamping force are more common in either tensile or shear joints than are failures caused by too much clamp. And, as we've seen, assembly and service conditions are more apt to give us too little clamp than too much.

2.7 BOLTS AND BOLTED JOINTS

2.7.1 Introduction

The most common method of joining one component to another in structural steelwork is bolting. Bolting may be carried out either in the shop or on site and has the advantage that the components can be separated easily should this become necessary for any reason. Most fabricators prefer to use welding for shop connections, but where workshops are equipped with automated punching and drilling machines, shop bolting is generally found to be quicker and cheaper. For site connections, however, bolting is virtually the universal medium of connection.

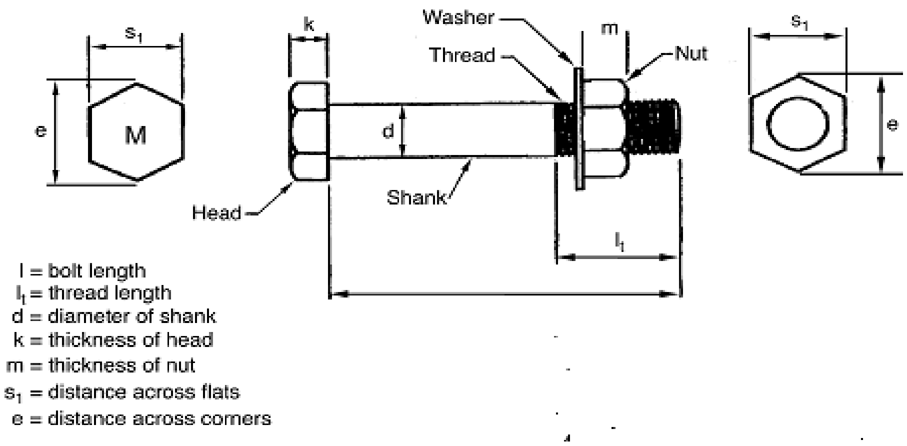


Figure 2.11 Data on bolts **[Bangash.2000].**

The main function of the bolt is to transmit a force from one member to another, Figure 2.11 shows the details of bolts. In all bolted connections a transfer of force is involved and in nearly all cases the transfer is by one or more of the following modes:

(a) shear in the bolt shank,

(b) bearing of the bolt shank against the holes in the two components,

(c) friction between the parts when the bolt is tightened to clamp the parts firmly together, and

(d) tension, when the load is applied in the axial direction of the bolt.

2.7.2 Design considerations of bolts joints

Steel generally exhibits significant ductility. The designer may therefore profit from this ability to deform steel plastically and so to develop “plastic” design approaches in which local stress plastic redistributions in the joint elements are allowed. The use of plastic design approaches in steel and composite construction is explicitly allowed in the Eurocodes. For joints and connections, reference has to be made to clause 2.5(1) of EN 1993-1-8. Obviously, limitations to the use of such plastic approaches exist. They all relate to the possible lack of ductility of the steel material, on the one hand, and of some connection's elements like bolts, welds, reinforced concrete slab in tension or concrete in compression, on the other hand. As far as steel material itself is concerned, the use of normalized steels according to Euro-norms (for instance EN 10025 (CEN, 2004d)) guarantees a sufficient material ductility.

The design of bolted joints, like the design of anything else, involves a detailed consideration of function, shapes, materials, dimensions, working loads, service environment, etc. The joint designer, of course, is faced with all the assembly and in-service uncertainties detailed earlier. In spite of these uncertainties, he must do two things when designing a joint that will be loaded all or in part in tension:

1. The designer must pick bolt and joint sizes, shapes, and materials which will guarantee enough clamping force to prevent bolt self-loosening or fatigue, and to prevent joint slip, separation, or leakage when clamping forces are at a minimum (because of the factors we’ve described) and those hard-to-predict service loads are at a maximum.

2. In addition, the designer wants to select bolts that are able to support a combination of maximum assembly stress plus the maximum increase in stress caused by such service conditions as applied load and differential thermal expansion. If the joint is loaded only in shear, and will depend for its strength only on the shear strength of the bolts and joint members, then those strengths will determine the design. Such joints must not be subjected to varying or cyclical loads, or self-loosening and fatigue problems might be encountered. If service conditions permit it, however, such joints are safe and greatly simplify the design process. There are other things that the designer must worry about when designing tensile joints and some shear joints. He’ll consider the bearing stresses the bolts create on joint surfaces, the amount of change in load the bolts see (which can affect fatigue life), the accessibility of the bolts (which can affect assembly results), and the flexibility or stiffness of bolts and joint members. If a tension joint is designed, the designer will be especially interested in the so-called stiffness ratio of the joint, because this affects the way in which a given service load changes bolt tension and clamping force. In any tension joint and in shear joints where clamping force is important, the designer will want to do everything he can to improve the energy storage capacity of his bolts. The designer will find that long thin bolts and thick, metal joint members can store more energy than short stubby bolts or non- metallic joints.

N.B. Several different types of bolts may be used in structural joints, including ordinary structural bolts (i.e. commercial or precision bolts and black bolts), and high-strength bolts. Turned close tolerance bolts are now rarely used.

2.7.3 Arrangement of joints

2.7.3.1 Joints for force transmission

In many cases, a joint is only required to transmit a force, and there is no moment acting on the group of connectors. While the joint may be capable of also transmitting a moment, it will be referred to as a force joint.

Force joints are generally of two types. For the first, the force acts in the connection plane formed by the interface between the two plates connected, and the connectors between these plates act in shear, as in Figure 2.12a. For the second type, the force acts out of the connection plane and the connectors act in tension, as in Figure 2.12c.Examples of force joints include splices in tension and compression members, truss joints, and shear splices and joints in beams. A simple shear and bearing bolted tension member splice is shown in Figure 9.1a, and a friction-grip bolted splice in Figure 9.1b. These are simpler than the tension bolt joint of Figure 9.1c, and are typical of site joints.

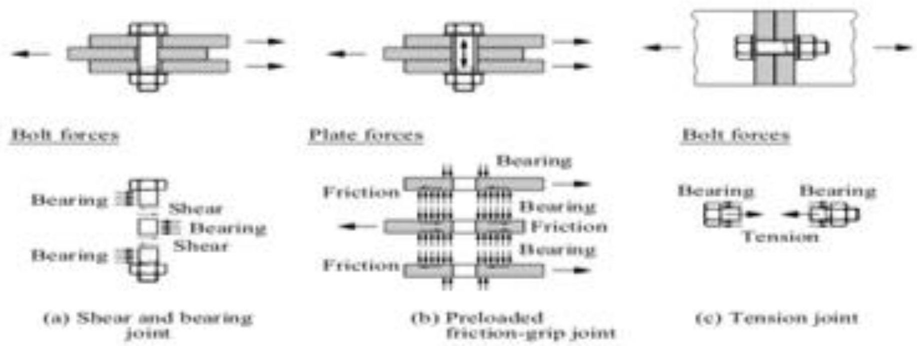


Figure 2.12 Use of bolts in joints.

2.7.3.2 Joints for moment transmission

While it is rare that a real joint transmits only a moment, it is not uncommon that the force transmitted by the joint is sufficiently small for it to be neglected in design. Examples of joints, which may be used when the force to be transmitted is negligible, include the beam moment splice shown in Figure 2.13b which combines site bolting with shop welding, and the welded moment joint of Figure

2.13c. A moment joint is often capable of transmitting moderate forces, as in the case of the beam-to-column joint shown Figure 2.13a.

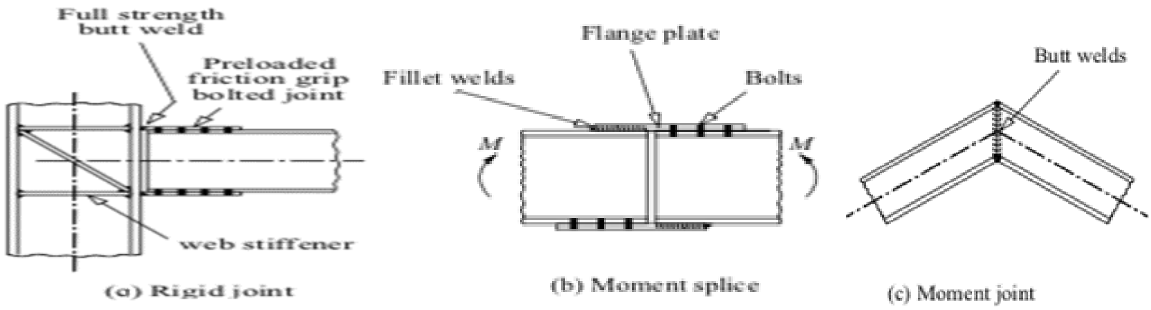


Figure 2.13 Transmitting moment in joints.

2.7.3.3 Joint idealisation [Jaspart et al.2017]

The non-linear behaviour of the isolated flexural spring which characterises the actual joint response does not lend itself towards everyday design practice. However, the $M-\varphi$ characteristic curve may be idealised without significant loss of accuracy.

One of the simplest idealisations possible is the elastic-perfectly-plastic one (Figure 2.14a). This modelling has the advantage of being quite similar to that used traditionally for the modelling of member cross sections subject to bending (Figure 2.14b).

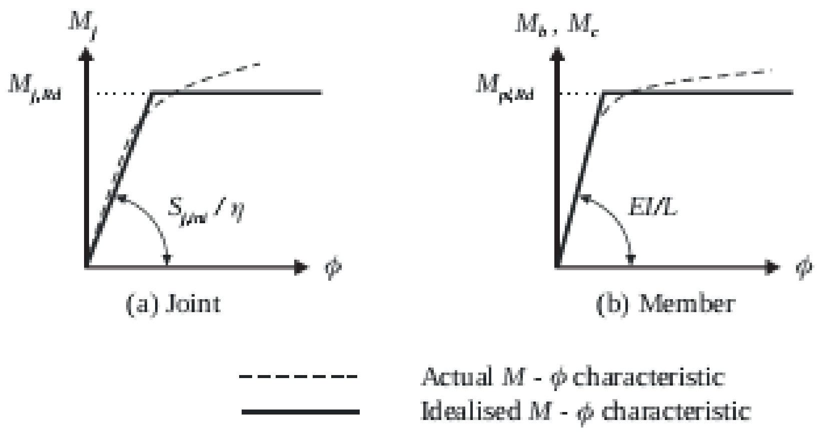


Figure2.14 Bi-linearisation of moment-rotation curves**[Jaspart et al.2017]**

The moment $M_{j,Rd}$ that corresponds to the yield plateau is termed design moment resistance in EN 1992. It may be understood as the *pseudo-plastic moment resistance* of the joint. Strain-hardening effects and possible membrane effects are thus neglected; that explains the difference in Figure 2.14 between the actual $M-\varphi$ characteristic and the *yield plateau* of the idealised one.

- **Elastic idealisation for an elastic analysis**

The main joint characteristic is the constant rotational stiffness.

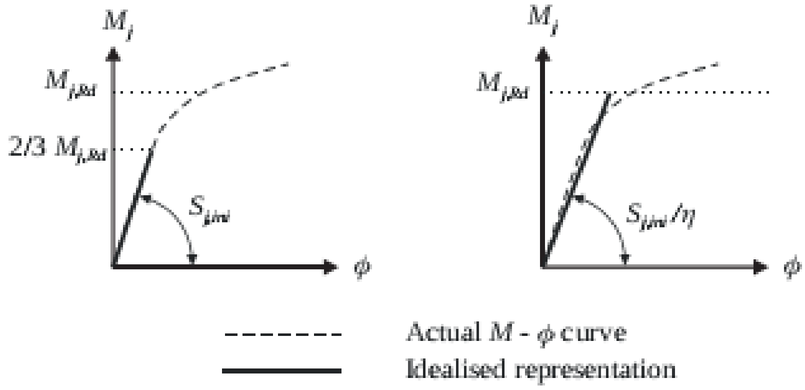


Figure2.15 Linear representation of a M−φ curve**[Jaspart et al.2017]**

Two possibilities are of fere din EN1993-1-8, see Figure2.15: Elastic verification of the joint resistance (Figure 2.15a): the –constant stiffness is taken equal to the initial stiffness $S_{j,ini}$; at the end of the frame analysis, that the design moment Mj,Ed experienced by the joint is less than the maximum elastic joint moment resistance defined as 2/3Mj,Rd;

Plastic verification of the joint resistance (Figure 2.15b): the – constant stiffness is taken equal to a fictitious stiffness, the value of which is intermediate between the initial stiffness and the secant Stiffness relative to Mj,Rd ; it is defined as Sj,ini/η. This idealisation is aimed at "replacing" the actual non-linear response of the joint by an equivalent" constant one; it is valid for Mj,Ed value less than or equal to Mj,Rd.

- **Rigid-plastic idealisation for a rigid-plastic analysis**

Only the designer is required for this type of analysis. In order to allow the possible plastic hinges to form and rotate in the joint locations, it has to be checked that the joint has a sufficient rotation capacity, see Figure2.16.

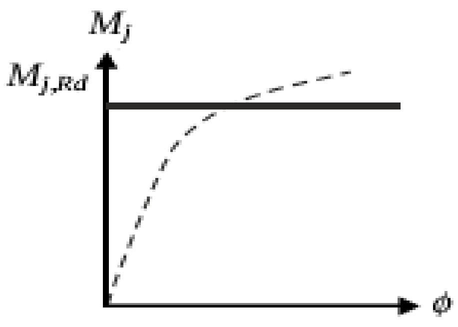


Figure 2.16 Rigid-plastic representation of a M–φ curve **[Jaspart et al.2017]**

- **Non-linear idealisation for an elastic-plastic analysis**

The stiffness and resistance properties are equally important in this case. The possible idealisations range from bi-linear, tri-linear representations or a fully non-linear curve, see Figure2.17. Again, rotation capacity is required in joints where plastic hinges are likely to form and rotate**.**

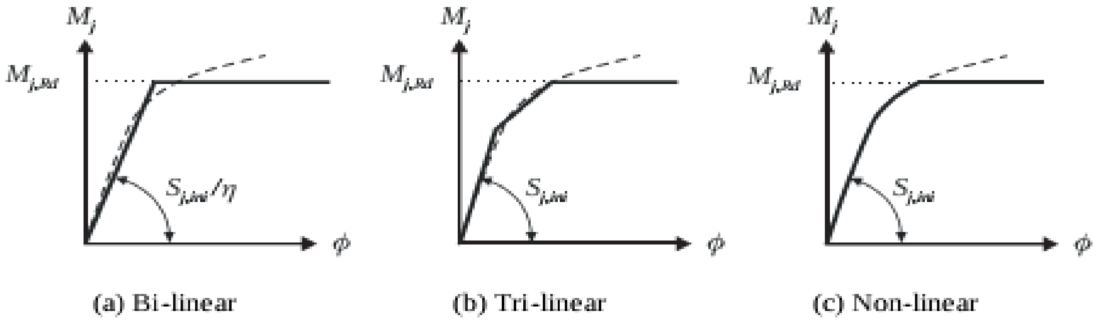


Figure2.17 Non-linear representations of a M–φ curve**[Jaspart et al.2017]**

2.8 END-PLATE [Allen et al.1995]

2.8.1 Design Philosophy

The design model used here is essentially that presented In Annex of Eurocode 3: Part 1.1. It is based on a plastic distribution of bolt forces. The method is the result of Extensive testing in Europe as well as a period of practical use in the Netherlands Although the design philosophy is taken directly from EC3, the strength checks on the bolts, welds and steel Have been modified to suit BS 5950: Part 1.

- **Load paths**

An end plate connection transmits moment by coupling Tension in the bolts with compression at the opposite flange. Unless there is axial force in the beam, the two Forces are equal and opposite illustrated in Figure 2.18. Tests show that, by the ultimate limit state, rotation has taken place with the centre of rotation at, or near, the compression flange that bears against the column. It is Therefore reasonable to consider that compression is concentrated at the level of the centre of the flange. The bolt row furthest from the compression flange will tend to attract the most tension, and traditional Practice has been to assume a triangular distribution of Forces. The method adopted here also gives greater priority to the outer bolts, but differs in that it allows a plastic Distribution of bolt forces. The force permitted in any bolt row is based on its Potential resistance, and not just on its lever arm. Bolts Near a point of stiffness, such as the beam flange or a Stiffener, will therefore attract more load. Rather than arbitrarily allocating force to each bolt row by a linear or 'triangular distribution, the method considers each side of the connection separately, making a precise allocation based on the capacity of each part

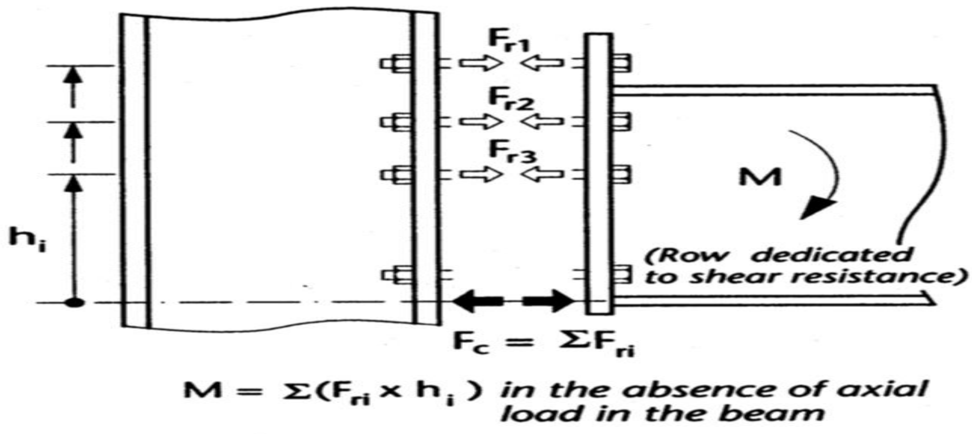


Figure 2.18 Forces in the connection.

Surplus force in one row of bolts can be transferred to an adjacent row which has a reserve of capacity. This principle is closer to the way connections actually perform in Practice. A plastic distribution of bolt forces is only reasonable, However, if the necessary deformation can take place. An upper limit is therefore set on the thickness of the column Flange, or end plate, relative to the bolt strength. Where this limit is exceeded on both sides of the connection, a modification to the bolt tension forces is made to ensure that they do not exceed a triangular distribution for rows below the beam flange. (This triangular limit to the plastic Forces is at present under consideration for inclusion in EC3.) Figure 2.19 compares the two plastic distributions with a more traditional triangular distribution.

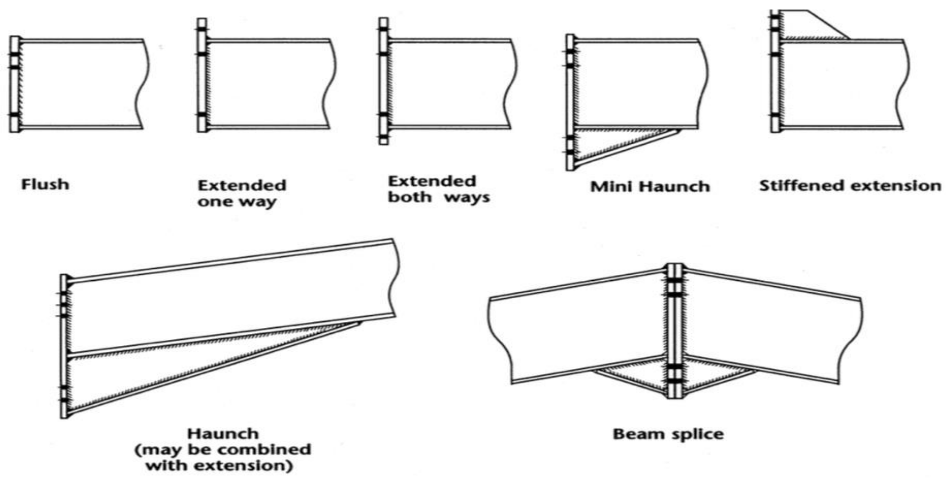
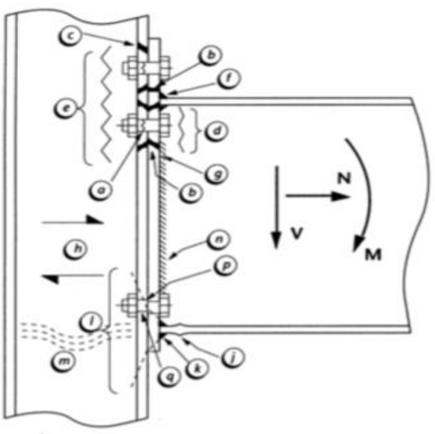


Figure 2.19 Distribution of bolt forces.



ZONE	REF	CHECKLIST ITEM	See Procedure
TENSION	a	Bolt tension	STEP 1A
	b	End plate bending	STEP 1A
	c	Column flange bending	STEP 1A
	d	Beam web tension	STEP 1B
	e	Column web tension	STEP 1B
	f	Flange to end plate weld	STEP 7
	g	Web to end plate weld	STEP 7
HORIZONTAL SHEAR	h	Column web panel shear	STEP 3
COMPRESSION	j	Beam flange compression	STEP 2
	k	Beam flange weld	STEP 7
	l	Column web crushing	STEP 2
	m	Column web buckling	STEP 2
VERTICAL SHEAR	n	Web to end plate weld	STEP 7
	p	Bolt shear	STEP 5
	q	Bolt bearing (plate or flange)	STEP 5

Figure 2.20 Component design checks.

2.8.2 Capacity checks

There are 15 principal checks to be made on the beam, the column, and on the bolts. These are shown, with a checklist, in Figure 2.20. Each of these checks is outlined in detail in the procedures later in this section and a flow chart is included which Leads the reader through the design process.

2.8.2.1 Tension zone

The resistance at each bolt row in the tension zone may be limited by:

- Column flange bending and bolt strength
- End plate bending and bolt strength
- Column web tension
- Beam web tension.

For column flange or end plate bending the method uses The Eurocode 3 approach which converts the complex Pattern of yield lines which occurs round the bolts into a Simple ‘equivalent tee-stub’ as shown in Figure 2.21. The Capacity of the tee-stub is then checked against three possible modes of failure illustrated in Figure 2.22.

One area of difficulty with bolted moment end plates has always been the treatment of the prying force ‘Q’. Depending upon the geometry of the connection, this Force can vary from 0% to upwards of 40% of the tension in the bolt. For this reason, simple design methods make a Blanket allowance for prying by assuming it is present, and has a value between 20% and 30% of the bolt Capacity. This approach is adopted by BS S950: Part 1 with the values for P.

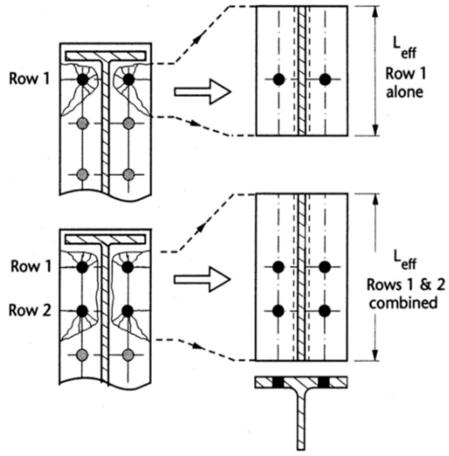


Figure 2.21 Equivalent T-stubs.

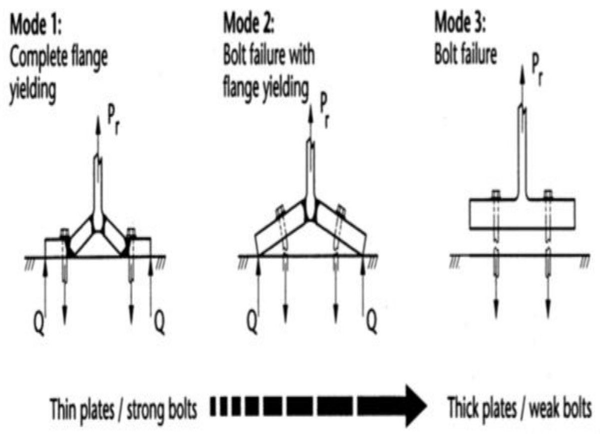


Figure 2.22 Column flange or end plate bending & bolt strength.

- **Distribution of bolt forces**

The resistance in each row, (P_{r1}, P_{r2}, P_{r3}) is calculated One row at a time, starting at the top and working down. In this way, priority is automatically given first to Row 1, Then to Row 2 and so on. At each stage, any bolts below the current row are ignored. The resistance of Row 1 is taken solely as the Capacity for Row 1 acting alone. Subsequent rows are checked both in isolation and also as part of a group in combination with successive rows Above. The resistance of Row 2 is therefore taken as the lesser of:

- The capacity of Row 2 acting alone, and
- The capacity of Rows (2+1) acting as a group Minus the tension already allocated to Row 1.

This process is illustrated in Figure 2.23.

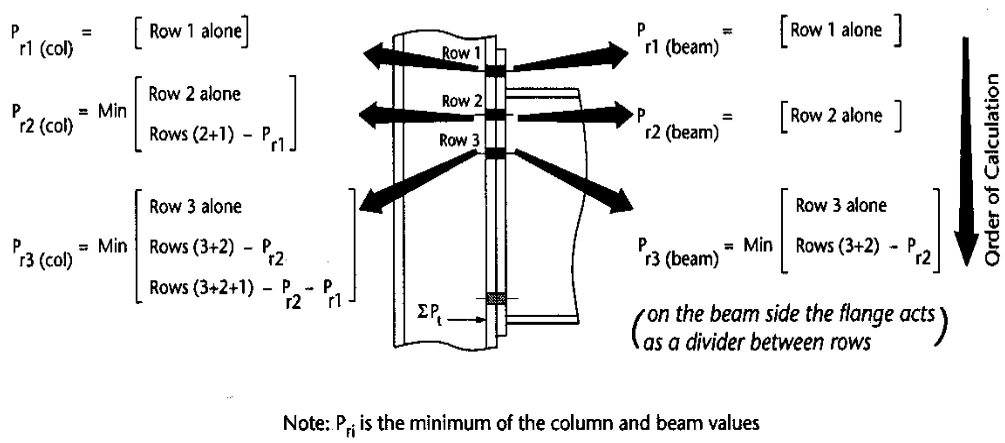


Figure 2.23 steps in calculating the distribution of bolt forces.

A tension stiffener (or the beam flange) acts as a divider between bolt groups, so that no row below a stiffener need be considered in combination with any row above it for that side of the connection. For example, in Figure 2.23, Rows 2 and 1 are not considered together for group action on the beam side of the connection because the beam Flange divides them, but they are considered together for the column side. The limit on full plastic distribution depending on the ratio of minimum flange or plate thickness to bolt diameter must also be considered.

2.8.2.2 Compression zone

Checks in the compression zone are similar to those traditionally adopted for web bearing and buckling. It is Reasonable to expect a properly sawn beam end to provide contact with the end plate, so that compression in the bottom flange is transferred in bearing. Guidance on allowable tolerance between bearing surfaces is given the National Structural Steelwork Specification for Building Construction.

It is common for the column web to be loaded in this Region to a point where it controls the design of the Connection. However, it can be strengthened. The column web must also be checked for buckling, but in this respect, it may be reasonable to consider whether in some cases buckling is prevented by other beam(s) connecting into the web at right angles to the connection under consideration. The compression on the beam side can usually be regarded as being carried entirely in the flange, and the centre of Compression taken at the centre of the flange. However, when large moments combine with axial load, the Compression zone will spread up into the beam web with a corresponding movement of the centre of compression.

2.8.2.3 Shear zone

The column web must also resist the horizontal panel Shear forces. To carry out this check, any connection at the Opposite flange of the column must also be taken into Account, since it is the resultant of the shears which must be borne by the web. In a one-sided connection with no axial force, the web Panel shear F, is equal to the compressive force ‘C’.

For a Two-sided connection with balanced moments, the column Web panel shear will be zero, and in the case of a Connection with moments acting in the same sense, such as in a wind-moment frame, the shear will be additive as showed in Figure 2.24.

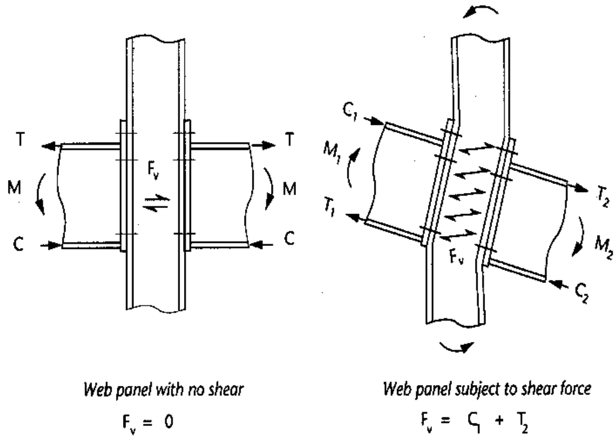


Figure 2.24 Column web panel.

The web of most UC section columns will fail in panel Shear well before it fails in bearing or buckling and Therefore, for one-sided connections, web shear is likely to Govern. Where this is critical, the column web can be Strengthened by using diagonal stiffeners, or by Supplementary web plates.

2.9 TYPES OF SEMI-RIGID CONNECTIONS [Chen.2000]

[1] Single Web-Angle Connections and Single Plate Connections

A single web-angle connection consists of an angle either bolted or welded to both the column and the beam web, as shown in (Fig. 2.25a). On the other hand, a single plate connection uses the plate instead of the angle. This connection type requires less material than a single web-angle connection (Fig. 2.25b). Generally, the single web-angle connection has moment rigidity equal to about one-half of the double web-angle connection. The single plate connection has rigidity equal to or greater than the single web-angle connection because one side of the plate in the single plate connection is fully welded to the column flange.

[2] Double Web-Angle Connections

A double web-angle connection consists of two angles, either bolted or riveted to both the column and the beam web, as shown in (Figure 2.25c). Rivets are used as fasteners in the earliest tests on double web-angle connection conducted by Rathbun (1936). In the 1950s, most specifications for the design of steel structures allowed the use of high-strength bolts in lieu of rivets. To clarify the effect of high-strength bolts on connection behavior when used in conjunction with rivets, Bell, Chesson, and Munse (1958) and Lewitt, Chesson, and Munse (1966) conducted experiments on riveted and bolted beam-to-column connections. Today, high-strength bolts are used popularly as fasteners for this type of connection. Although the connection rigidity of this type of connection is stiffer than those of single web-angle and single plate connections.

[3] Top- and Seat-Angle Connections

A typical top and seat-angle connection is shown in (Figure 2.25d). The AISC ASD Specifications (AISC, 1989) described this type of connection as follows:

(1) the top-angle is used to provide lateral support of the compression flange of the beam; and (2) the seat-angle is to transfer only the vertical reaction of the beam to the column and should not give a significant restraining moment on the end of the beam. However, according to the experimental results, this connection can transfer not only the vertical reaction, but also some end moment of the beam to the column.

[4] Top- and Seat-Angle with Double Web-Angle Connections

This type of connection is combination of a top- and seat-angle connection and a double web-angle connection. A typical top- and seat-angle connection with a double web-angle is shown in (Fig.2.25e). Double web-angles are used to improve the connection restraint characteristics of top-and seat-angle connections, and for shear transfer. This type of connection is considered as Type 3 framing of the AISC ASD Specifications (AISC, 1989), that is to say, a semi-rigid connection.

[5] Extended End-Plate Connections and Flush End-Plate Connections

In general, an end-plate is welded to the beam end along both the flanges and web in the fabricator's shop and bolted to the column in the field. The end-plate connection has been used extensively since the 1960s. The extended end-plate connections are classified into two types: as an end plate either extended on the tension side only or on both the tension and compression sides, as shown in (Figure 2.25f) and (2.25g) , respectively. A typical flush end-plate connection is shown in (Figure .2.25h). Because some end-plate connections are considered as Type FR construction rather than Type PR construction in AISC LRFD Specifications (AISC, 1994), they have often been used as means of transferring beam end moment to the column. The extended end-plate connection on both sides is preferred when the connection is subjected to moment reversal such as during severe earthquake loading. Although the flush end-plate connection is weaker than the extended end-plate connection, this connection type is often used in roof details. The behavior of an end-plate connection depends on whether the column flanges act to prevent flexural deformation of the column flange, thereby influencing the behavior of the plate and fasteners.

[6] Header-Plate Connections

A header-plate connection consists of an end-pate, whose length is less than the depth of the beam, welded to the beam web and bolted to the column, as shown in (Figure 2.25 i). The moment-rotation characteristics of this connection are similar to those of the double web-angle connection.

Accordingly, a header-plate connection is used mainly to transfer the reaction of the beam to the column and is classified as a connection in Type 3 framing of the AISC ASD Specifications (AISC, 1989).

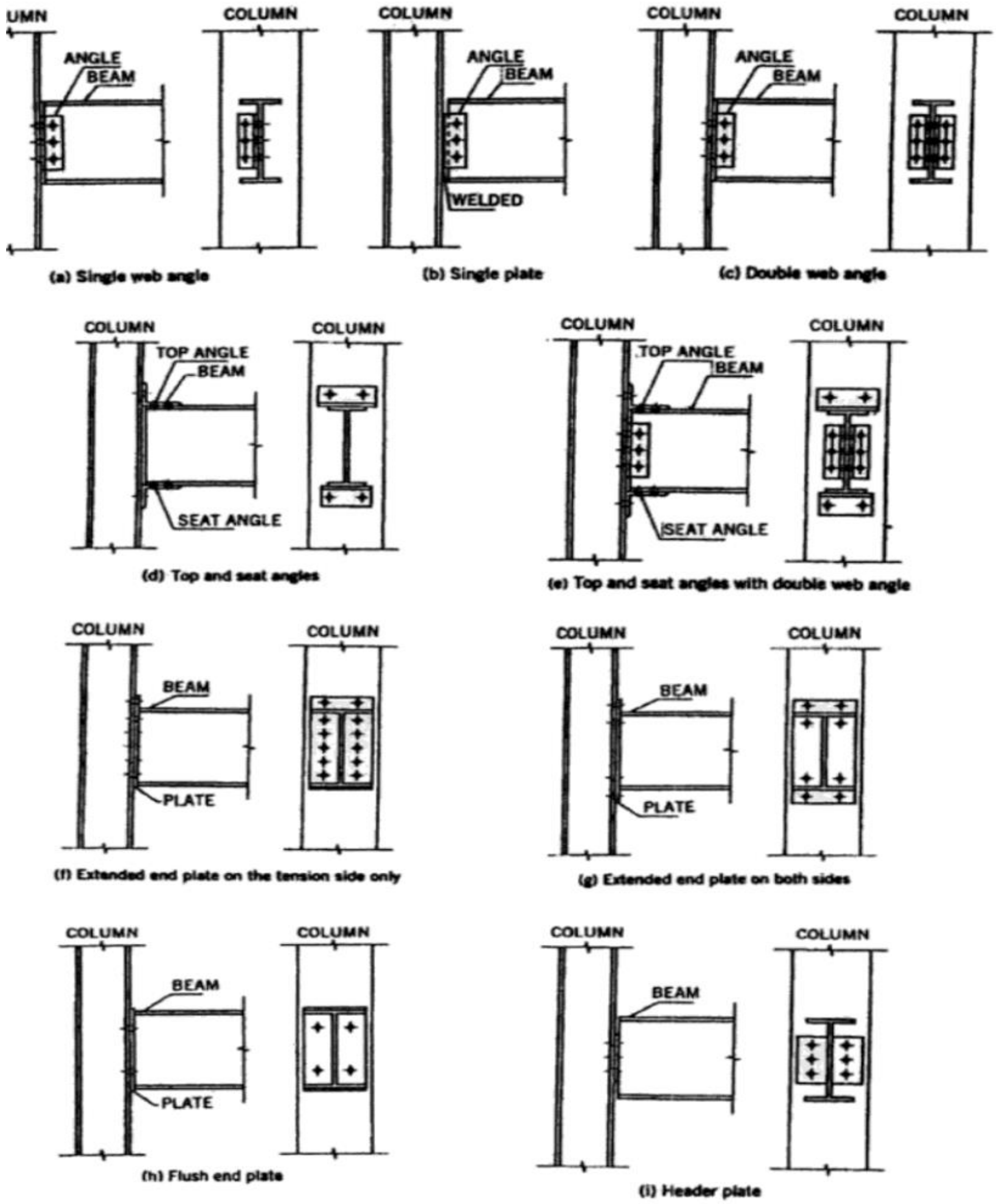


Figure 2.25 Types of semi-rigid connections.**[Chen.2000]**

CHAPTER 3

MODELLING OF END- PLATE CONNECTION

UNDER MONOTONIC LOADING

3.1 INTRODUCTION

In recent years, the engineer's ability to model buildings has increased quickly with the development of advanced analysis programs and the competition among software developers. Many powerful FE software packages have become commercially available (e.g. LUSAS, ABAQUS, and ANSYS). They incorporate facilities that enable a wide range of engineering problems to be efficiently modelled and accurately analysed **[Cafer. 2009].**

The aim of this chapter is to propose a numerical model in order to predict the linear and nonlinear behaviour of beam-to-column semi rigid joints subjected to monotonic load using finite element (FE) process. The first cases analysed by means of ABAQUS are to validate the model with data taken out from literature initially executed with ANSYS. Then parametric analyses are performed taking into account some parameters known to affect the general behaviour of the connection. Some conclusions have been drawn concerning the global behaviour of semi-rigid connections in a beam-column structure under static monotonic loadings.

3.2 INTRODUCTION TO SOME ASPECTS OF ABAQUS

3.2.1 General

ABAQUS is one of the leading finite element packages and has much operational and verification experience to back it up, not with standing 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 Systems.

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

3.2.2 A Modeling Chain

Figure 3.1 shows how to create a Finite Element model in Abaqus/CAE. It is not possible to create elements and nodes directly. Element and nodes are only created by the mesh generator, which works and the geometric objects, which are created drawing a sketch.

The only FE model parameter, which are created directly, are the material properties and the section values. These properties are created within the module Property. The properties are then assigned to the geometric objects (lines, areas and volumes).

After having created a sketch the sketch has to be assigned to a part. If no part exists, a part hast to be created. The properties (materials and section data) are assigned to the sketches' geometric objects.

To create a mesh, an instance is needed, so an instance has to be created. The part with the sketch is assigned to the instance for later meshing. Load-cases are modeled in terms of load steps. So, a new step has to be created as a container for loads and boundary conditions. Loads and boundaries are assigned to the geometric objects of the sketch which were assigned to a part before.

To create the mesh, the mesh's control parameters should be configuration and the element types are to be assigned. Then the instance can be meshed.

After the mesh is created, the complete model can be assigned to a job. To calculate the results the job has to be submit **[Baeck. 2018].**

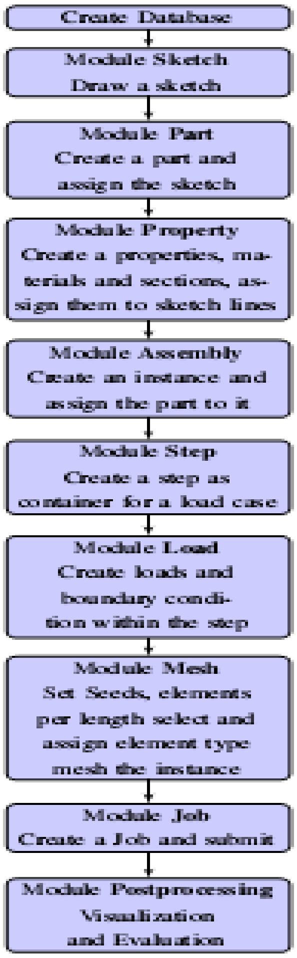


Figure 3.1 Modelling chain diagram.

3.2.3 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 beam-column, specially under cyclic loadings.

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 3.2.

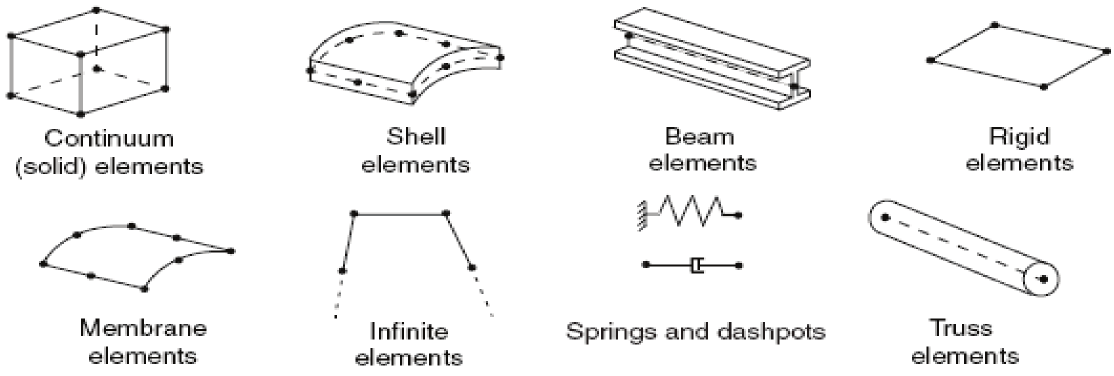


Figure 3.2 Finite element families available in **ABAQUS (2004)**

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.

Meshing: Meshing is performed to discretize the geometry created into small pieces called elements or cells. The rationale behind this can be explained in a very straight for-ward and logical manner. The solution can be expected for an engineering problem to be very complex, and varies in a way that is very unpredictable using functions across the whole domain of the problem. Mesh generation is a very important task of the pre-process. It can be a very time con-summing task to the analyst, and usually an experienced analyst will produce a more credible mesh for a complex problem. The domain has to be meshed properly into elements of specific shapes such as triangles and quadrilaterals. Information, such as element connectivity, must be created during the meshing for use later in the formation of the FEM equations. It is ideal to have an entirely automated mesh generator, but unfortunately this is currently not available in the market.

The density of the mesh depends upon the accuracy requirement of the analysis and the computational resources available. Generally, a finer mesh will yield results that are more accurate, but will increase the computational cost. As such, the mesh is usually not uniform, with a finer mesh being used in the areas where the displacement gradient is larger or where the accuracy is critical to the analysis. The purpose of the domain discretization is to make it easier in assuming the pattern of the displacement field.

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.

For complex problems whose exact solution is of a very high order of polynomial type, or often a non-polynomial type, it is then up to the analyst to use a proper density of the element mesh to obtain FEM results of desired accuracy with a convergence rate.

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.

3.3 GENERAL ON MODELLING USING ABAQUS SOFTWARE

The general multi-purpose finite-element modelling package **ABAQUS** is chosen to be used in this research for many reasons including:

- Nonlinear performance: The strength of the Abaqus code, which was originally developed as a nonlinear solver, is that it will run any nonlinear simulation faster and will converge on truer, more realistic results than other codes.
- Contact modelling: Abaqus is by far the best FEA code at handling all forms of contact, however, automatically sets internal controls for its parameters such that a converged surface contact result occurs during the solver run, even without user influence. The FEA user thus has high confidence that the contact convergence actually works consistently.

- Multiphysics simulations: Abaqus performs nearly all forms of multi-physics FE analysis and the ability to operate between the implicit solver, generally used for stress/strain, and the highly dynamic explicit solver, for a seamless Dynamic-Static co-simulation.
- Fracture and failure: Abaqus offer a general framework for modelling bulk material damage and failure over a wide range of materials (composites, metals, concrete, etc.) and structures. This framework allows simulation of damage initiation and evolution without the need for specifying any initial imperfection in the structure [**FEA services LLC].**

The use of solid finite elements is one of the most suitable methods for modelling connections. Several attempts were undertaken through last four decades to model semi-rigid connections with 3-D finite elements. Earlier models were highly simplified to reduce computational effort. Increasing computational power in the last decade enables to cope with models that are more complicated with ease. However, the improvement in computational power does not necessarily mean accurate simulations of the actual behaviour of the connections. Actually, drawing moment-rotation curves, which represent the result of very complex interaction between connection elements, requires the consideration of the following:

- Geometrical and material nonlinearities of the elementary parts of the connection
- Bolt pretension force and its response under general stress distribution
- Contacts between bolts and plate components: i.e. bolt shank and hole, bolt head or nut contacts
- Compressive interface stresses and friction
- Slip due to bolt to hole clearance
- Variation of contact zones
- Welds
- Imperfections i.e. residual stresses

Recent finite element analyses consider nonlinearities in both geometry and material together with bolt pretension force, contact elements, friction and slip. On the other hand, covering imperfections and variation of contact zones require a level of refinement, which is not yet attained.Besides the above list of details, there are also other modelling approximations that have great influence on the finite element analysis, and these can be stated as follows:

- The used finite element program
- Considered element types

- Meshing of the elements
- Number of the finite elements that is used
- Definition of holes and fills
- Boundary conditions
- Representation of the environment (i.e., temperature, rate of loading etc.)

Although there are many considerations to be taken into account related with the simulation of the semi-rigid connection as listed above, the finite element method provides highly accurate results even when some simplifications to above mentioned considerations are introduced to the model. Meanwhile, simulation with finite elements takes considerable amount of time in spite of the improvement in the computational power. Knowing these and the capacity of the personal computers, the user of a finite element program should consider where to make simplifications carefully. Since even small changes of the above-mentioned properties may cause significant differences in the results. Furthermore, responses of different kinds of connections and modelling considerations for these types are different, as well **[Cafer. 2009].**

3.4 CASE STYDIES

3.4.1 Description of the Finite Element Model

The purpose of this study is to determine if the ABAQUS finite element program can be used to accurately model the nonlinear behaviour of beam to column semi-rigid connection subjected to lateral load and local buckling mode of failure in plastic hinge regions of beams. Three dimensional explicit analyses have been performed in Abaqus V6.12. A great deal of assumptions was made in establishing the model discussed in the following sections.

3.4.2 Detailed data

This model obtained from an edge part of a steel frame, consisted of a rectangular end plate welded to the beam cross section and fixed to the column flange by six bolts distributed at three rows, two rows of them situated above the symmetry axis (at the tension side of the connection)and the third below it (above the compression beam flange), as shown in Figure 3.3.

3.4.3 Validation of the FE model with monotonic test results

The finite element model is made to investigate the results of both experimental and theoretical models under monotonical load that presented by Khalil **[Khalil .2004]** for steel beam-to-column connections, Figure 3.3.

- **Experimental set-up**

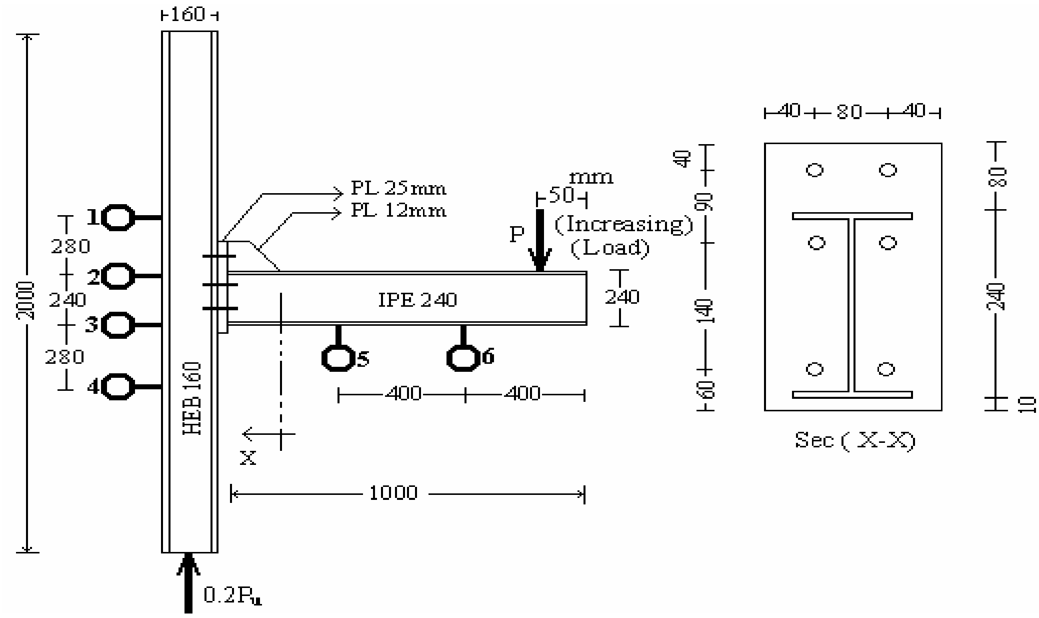


Figure 3.3 Beam-to-column steel joint by Khalil **[Khalil. 2004].**

- **Geometric properties of joint components**

- **Column and beam profiles**

The studied joints have the following data:

- Steel column HEB 160 with a total height of 2000 mm, Table 3.1.

- Steel beam IPE 240 with total length of 1000 mm, Table 3.1.

- Extended end-plate [height = 330 mm, width = 160 mm and thickness = 25 mm].

N.B. - Steel grade for column, beam, end plate, and stiffener is S240.

Table 3.1 Geometric characteristics of column and beam profiles.

PROFILES	H (mm)	b_f (mm)	t_w (mm)	t_f (mm)	r (mm)	d (mm)	Area (cm^2)	Moment of inertia (cm^4)
HEB 160	160	160	8,0	13	15	104	54,3	2492
IPE 240	240	120	6,2	9,8	15	190,4	39,1	3892

- **Bolts**

- High strength bolt [diameter = 22 mm and grade 10.9]. Figure 3.4.

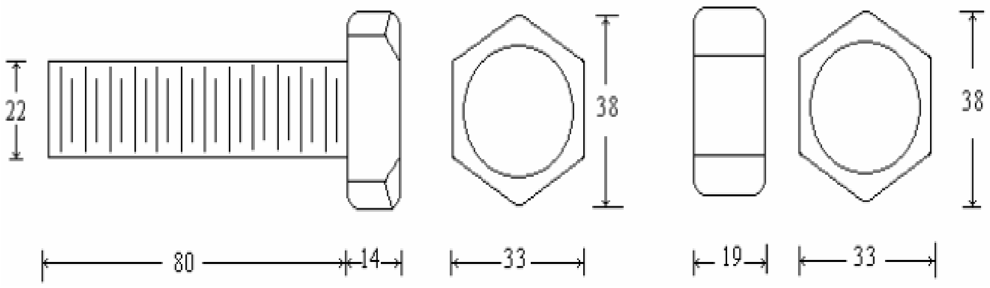


Figure 3.4 Dimensions of Bolts (M22 Grade 10.9) in mm.

The bolt head, shank and nut have been modelled as one solid part as shown in Figure 3.5. The threaded region of the shank is idealized as circular with diameter corresponding to the cross-sectional area A_s= 303 mm^2.

- **Monotonic loading applied system**

In the analysed beam-to-column connection, two loads were applied monotonically, the column is subjected to a uniformly distributed load located on the bottom part (upward) equal to 0.2P_u in which $P_u$is the ultimate load of the column and A_s is the area of the column. While the beam is subjected to a concentrated load acting with 50 mm away from the tip of the beam to generate an increasing bending moment during the test, Figure 3.3. Figure 3.6 Shows the applied load in the model.

As the main goal of the present analysis is to compare, in order to validate the nonlinear numerical model, data from literature were taken as reference that is the experimental of Khalil **[Khalil. 2004]**. In fact, results from six dial gauges, as displayed in Figure 3.3, were used to measure instant both the vertical and horizontal displacements of the connection. The placed dial gauges from one to four were used to give data of horizontal displacements, whereas gauges five and six were placed to measure vertical displacements.

- Pretension of bolts

The method of pre-tensioning full 3D solid bolt in the present model is applying the preload as a force in all bolt instances. An illustration of the pre-tensioning bolts is shown in figure below 3.5.

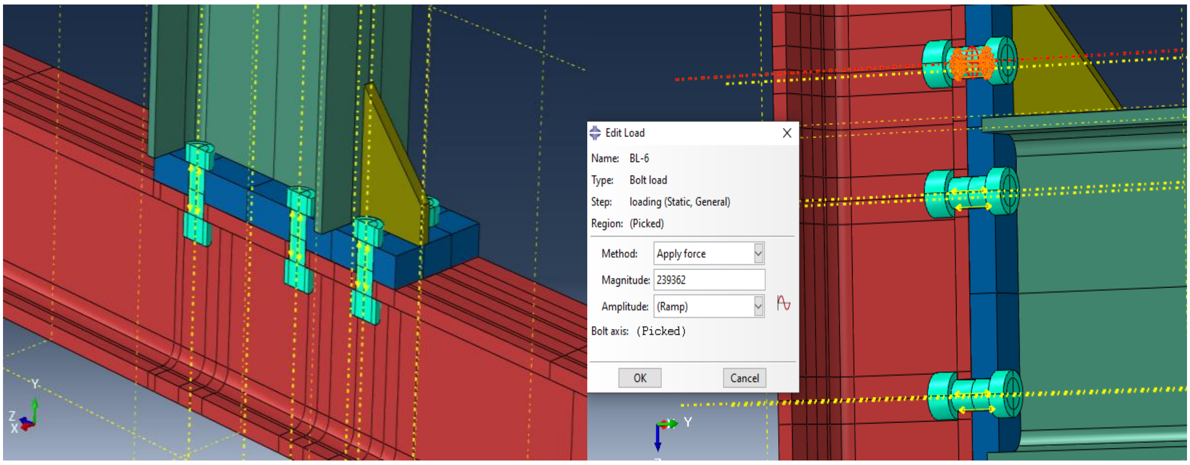


Figure 3.5 Method of Pre-tensioning bolts using ABAQUS.

- Meshing

Meshing is one of the most important issues in modelling since the accuracy of the results largely depends on it. Many meshing techniques able to be used in Abaqus but in some cases just one technic is valid. Structured meshing is the only mesh technic used in all elements of the model, it gives the most control in mesh and transforms the regularly shaped region on to the geometry of the region. The result is a three-dimensional mesh created from a face. The meshing process must balance the need for a fine mesh to give an accurate stress distribution and reasonable analysis time. This is adequate refinement for the constructed 3D model. Linear brick elements with reduced integration (S4R) were used throughout.

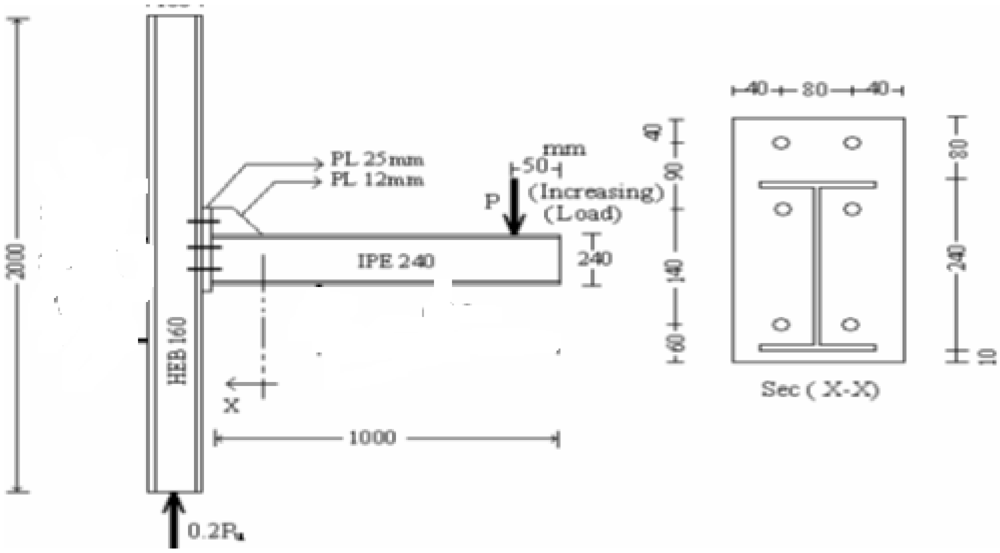


Figure 3.6 Geometric of the model.

The finite element mesh of the beam was refined in the vicinity in the region of the connection where the modes of failures were expected to develop. The total number of elements is (26846) linear hexahedral elements of type C3D8R is used in the modelling as shown in Table (3.2). Figures (3.7) meshing of the connections.

Table 3.2 Details on elements of the Steel Connection.

Parts	Number of nodes	Number of elements
Column	23618	18300 (C3D8R)
Beam	9204	7140(C3D8R)
End-plate	975	608(C3D8R)
bolts	201	128(C3D8R)
stiffener	72	30 (C3D8R)
Total	35075	26846 (C3D8R)

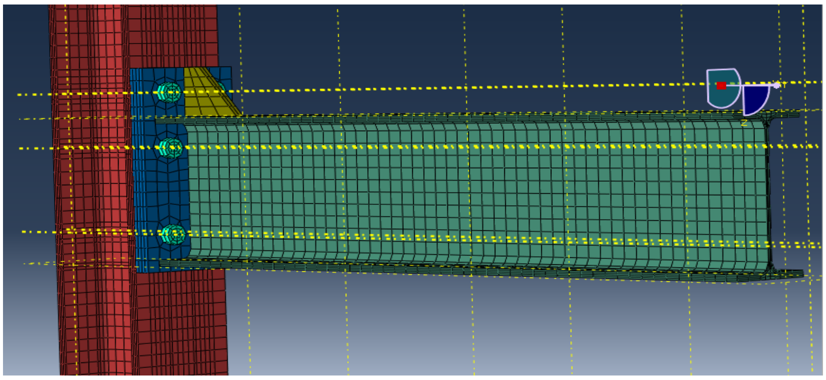


Figure 3.7 General View of the Connection.

- Type element

One finite element type (SOLID C3D8R) 8-node linear brick, reduced integration with hourglass control was used in the modelling of all elements (beam, column, end-plate, bolts and stiffener).

- Material Properties

• Elastic properties

The elastic material properties define the recoverable part of the strain. The elastic response of the material is modelled linearly. Young's Modulus, Poisson's ratio, and the material density are the only parameters required for the elastic part of the material model.

- Density γ =7.85 10^{-9}t/mm^3 = 7.85 10^{-5} N/mm^3.
- Young's modulus E =21 t/mm^2 = 210000 N/mm^2.
- Poisson's ratio ν = 0.3.

The stress strain curves are taken as elastic-strain hardening. This is acceptable since strain hardening is paired with excessive yielding in large areas and a large deflection criterion governs the ultimate strength design. In end-plate connections, however, excessive strain is mostly local and besides considerable shear stresses occur in the region between the top bolts and the beam tension flange, which necessitates considering strain hardening. Stress-strain curves for HS (high strength) bolts, and steel sections are shown in Figures 3.8a and 3.8b respectively **[Mashaly et al 2011].**

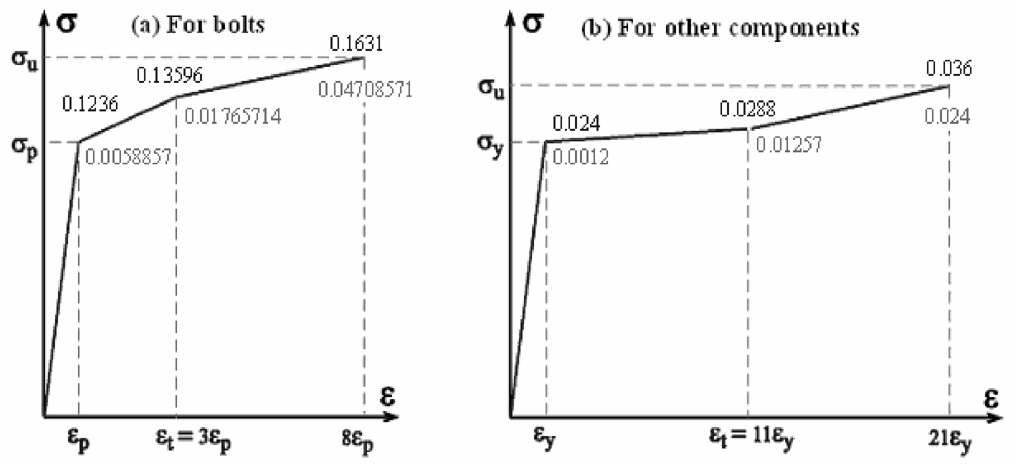


Figure 3.8 Trilinear Stress – Strain Curve: (a) for High Strength Bolts; (b) for Steel Sections **[Mashaly et al 2011].**

- **Nonlinear Isotropic/kinematic Hardening Model**

The nonlinear isotropic/kinematic hardening model was used to model the material response in the plastic range. This material model consists of two components: a nonlinear kinematic hardening component, and a nonlinear isotropic hardening component.

- **Yield Criterion**

The von Mises yield criterion (the classical metal plasticity theory) which is most commonly applicable to initially isotropic engineering materials, is used to predict the onset of the yielding. The behaviour upon further yielding is predicted by the "flow rule "and "hardening law". The associative flow-rule for the von Mises yield criterion, i.e. Prandtl-Reuss flow equations is used along with hardening of steel sections and bolts to model the Bauschinger effect. Kinematic hardening is assumed for modelling of the steel connection assuming that the yield surface only transfers in the direction of yielding and does not grow in size **[Mashaly et al 2011].**

- **Boundary Conditions**

The figure 3.9 shows the way of the application of the boundary condition the upper column end is a pinned support which can resist both vertical and horizontal forces (constrains translation) and allow the structural member to rotate, the lower column end is a roller support along the vertical axis (direction of the column axis) both horizontal and rotational displacement can occur while the end of the beam is considered free not supported.

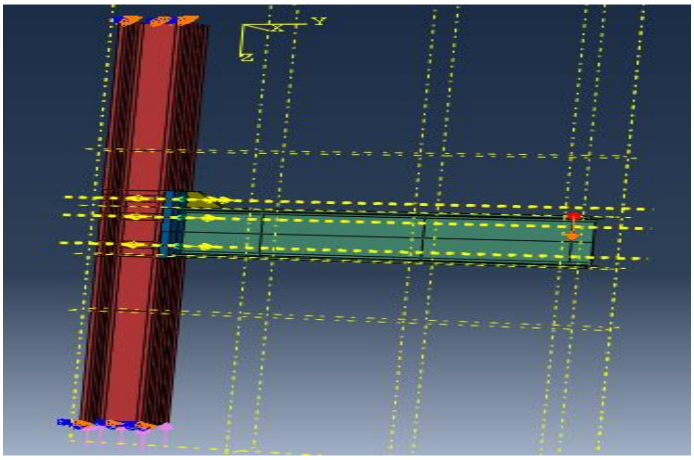


Figure 3.9 Boundary conditions, load and constraints.

- **Contact**

  - Interaction was considered between the following regions:

- Column flange and the back of the end plate.
- Column flange and bolt nuts.
- Column flange hole and bolt shank.
- End plate and bolt head.
- End plate hole and bolt shank.

And a coefficient of friction considered 0.3 defined as the ratio for sliding resistance while the interface is closed.

  - Constraints was applied using tie with surface-to-surface discretization methods and it was considered between the following regions:

- Beam and plate.
- Stiffener, beam and plate.

The boundary at the back of the end-plate is a variable boundary-value problem that can be solved only by an interactive approach. Figure 3.10 explains the mechanism of the interface (contact) between two surfaces when modelled using Finite Elements. In the beginning, the two surfaces know nothing about each other since no mathematical connection (stiffness) exists between the surfaces, Figure 3.10a. If the upper surface displaces downward due to a force *P,* it will move through the lower surface as it does not exist, Figure 3.10b. Attaching a spring of stiffness (K*)* to the nodes of the lower surface will only carry load when the gap closes in compression, Figure 3.10c. Therefore, when the upper surface contacts the lower one, a spring force develops to prevent the upper surface from moving through the lower one. Equilibrium will be achieved when $K\Delta=P$, Fig 3.10d. The amount of "pass through"Δ will therefore depend on the spring stiffness *K*. Real surfaces have zero material overlap meaning that interface stiffness $K = \infty$. However, using a very high stiffness causes numerical problems in the equation solver and convergence difficulties in problems with multiple inter-surface elements. Practically, the required value of *K* is the one which allows an accepted very small amount of overlap between the two surfaces, the interface stiffness *K* is taken between 17.5 and 1750 t/mm **[Mashaly et al 2011].**

Eeight - integration points were used through the solid thickness at the centre of the solid.

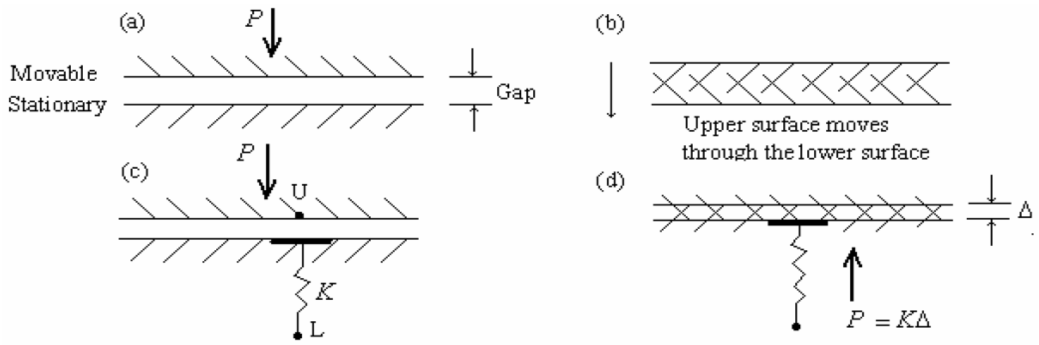


Figure 3.10 Interface element mechanism **[Mashaly et al 2011].**

- ## Results and validation of the model discussion

  - **General information on models**

For a validation purpose with some data available in literature, four finite element models have been implanted in Abaqus. For each model, the total of number of elements is (26846) linear hexahedral elements of type C3D8R is used in the modelling as shown in Table (3.2).

As far as the boundary conditions are concerned, figure 3.9 shows the way of their application of the upper column end is a pinned support, the lower column end is a roller support along the vertical axis (direction of the column axis) while the end of the beam is considered free and not supported. For contact conditions, which are fully described in the upper section showing the interactions between different regions, that is column flange and the back of the end plate; column flange and bolt nuts, column flange hole and bolt shank; end plate and bolt head and end plate hole and bolt shank. Also, it must be recalled that, a friction coefficient which defines as the ratio for sliding resistance while the interface is closed is used for all models treated in this study equal to 0.3. Constraints were applied using tie condition with surface-to-surface discretization methods between beam and plate; stiffener, beam and plate.

- **Actual used data**

As the main goal of the present analysis is to compare, in order to validate the nonlinear numerical model (ABAQUS), data from literature were taken as reference that is the experimental of Khalil **[Khalil .2004]**.

In the analysed beam-to-column connection, two loads were applied monotonically that is the column is subjected to a uniformly distributed load located on the bottom part upward)and equal to $0.2Pu$ While the beam is subjected to a concentrated load acting with 50 mm away from the tip of the beam. In fact, six dial gauges were used to measure instant the vertical and horizontal deflections under progressive load, in positions shown in Figure 4.3. The placed dial gauges from one to four were used to give data of horizontal displacements, whereas gauges five and six were placed to measure vertical displacements.

- **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 Loads vs. displacements curves as they better retrace the loading history. The second part of the discussion will be devoted to the stress analysis by mean of the available failure criteria implanted in ABAQUS: Von Mises yielding criterion. Also, the evolution of forces in high resistance H.R. bolts will discussed in semi-rigid connection of a beam to column structure. 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.

- **Results discussion**

Broadly speaking, and as expected, it can be clearly seen that all load-deflection curves show two distinct branches: linear elastic and curved branches. A close examination for Figures 3.11 indicates that there is a good agreement between compared results.

- **Loads-displacements relationships**

Results of the present numerical study showing the effect of semi-rigid connection of end plate in the beam to column structures considered. Figures 3.11 (a), (b), (c), (d) (e) and (f) give curves Load-displacement at dial positions. In each figure, three different results are displayed, namely the experiment and theoretical method from **[Khalil.2004]** in red and blue respectively, and the nonlinear numerical output of the present analysis in black.

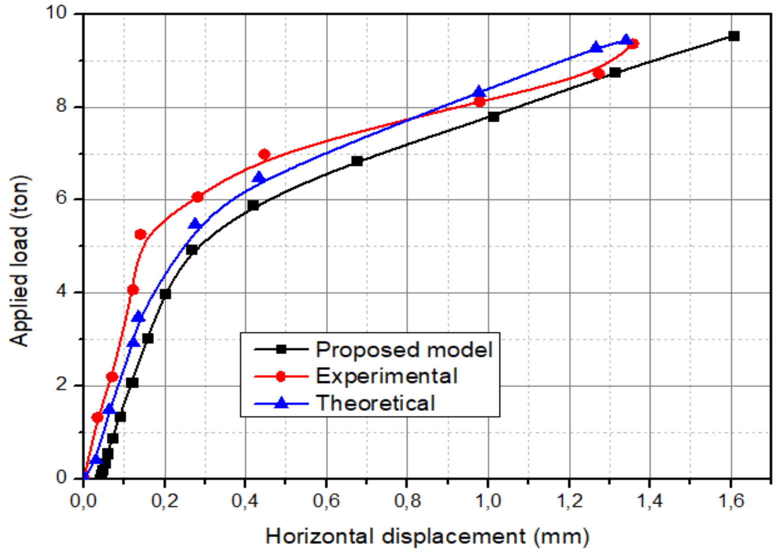
Figure 3.11 shows the relation between the load and displacement at different location (six dial) dial 1to 4 for horizontal displacement where dial 5 and 6 for the vertical one and comparing their changes in finite element model to the experimental and theoretical. The positive sign of the displacement mean that the horizontal displacement is in the right direction while for vertical means downward.

Broadly speaking, and can be clearly seen from Figure 3.11, the predictions of the present finite element model implanted in ABAQUS show a good agreement in terms of Load- deflections results, in the location of dials 1 to 4 and vertical deflections in the locations of dials 5 and dial 6, with those of the theoretical and experimental outcomes **[Khalil .2004].**

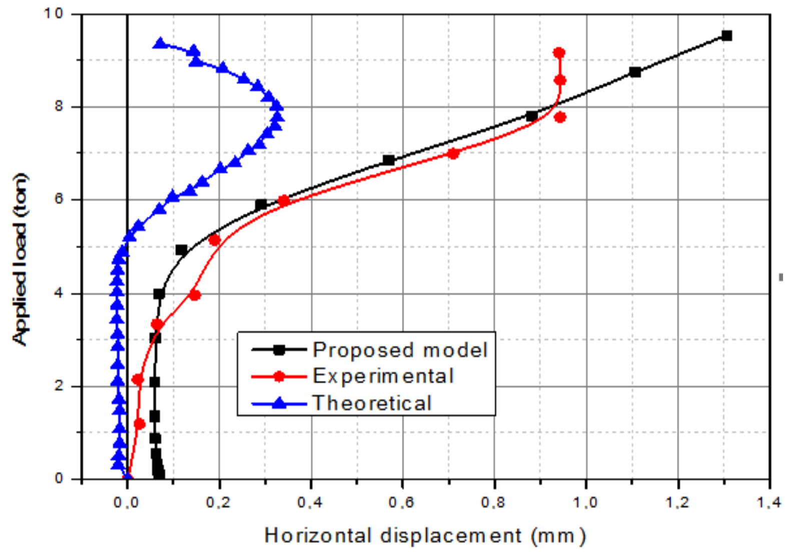
The results and comparisons of the deflection versus load history are shown in Figure 3.11. In all figures, load- displacements are presented through two curve branches: linear and nonlinear relationship, representing the linear elastic and inelastic behaviours of the model. The ultimate applied load values vs. corresponding displacement vary naturally with the position of the corresponding dial gauge in the studied model. Once again, positive sign means that the horizontal displacement is in the right direction (Z - direction or direction of the beam web) while the positive sign for vertical displacement means downward.

It must be noted that the outcomes of the present analysis are closer to the experimental and theoretical results for gauges 5 and 6 dealing with the vertical displacements. However, for horizontal displacement curves some differences can be noticed. The results of the present FE model at the location of dial gage (2) are slightly different from the experimental results and totally different from the theoretical model. This can be may be explained by the fact that experimentally, dial gage (2) is in the region of panel zone kinking and tension bolts which may lead to errors in the very small readings of the dial gage.

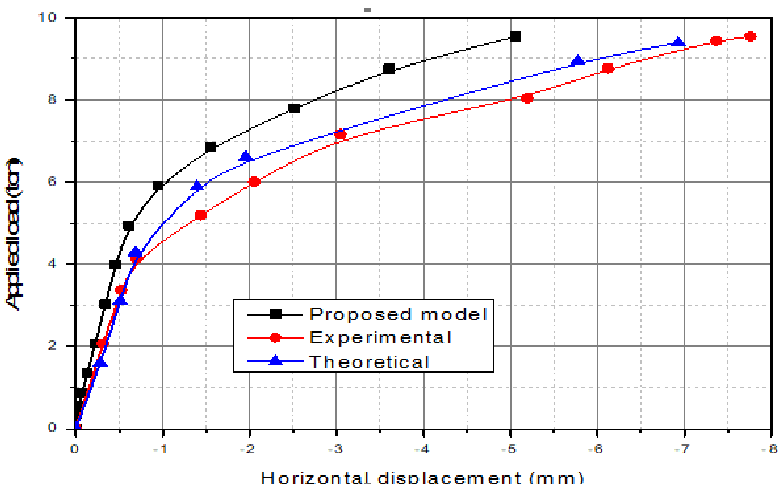
In fact, for dial 1 to 6 the present results underestimate the stiffness of the structure compared to others situations. While, nonlinear numerical outcomes overestimate the stiffness of the structures.



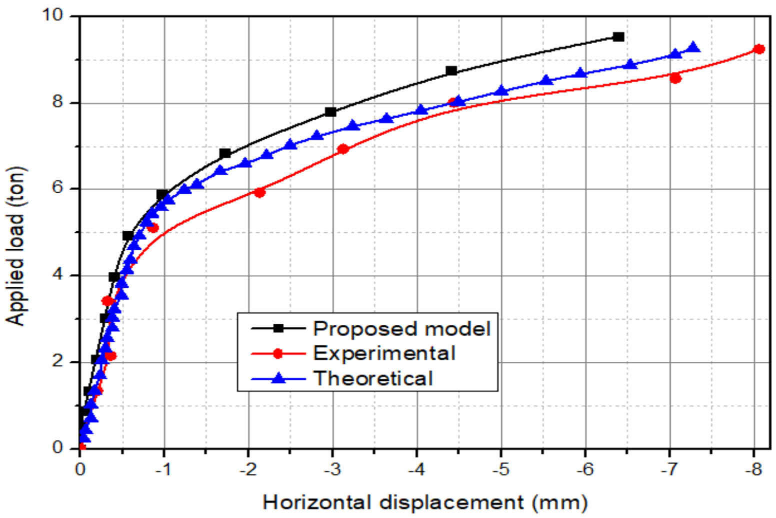
(a)



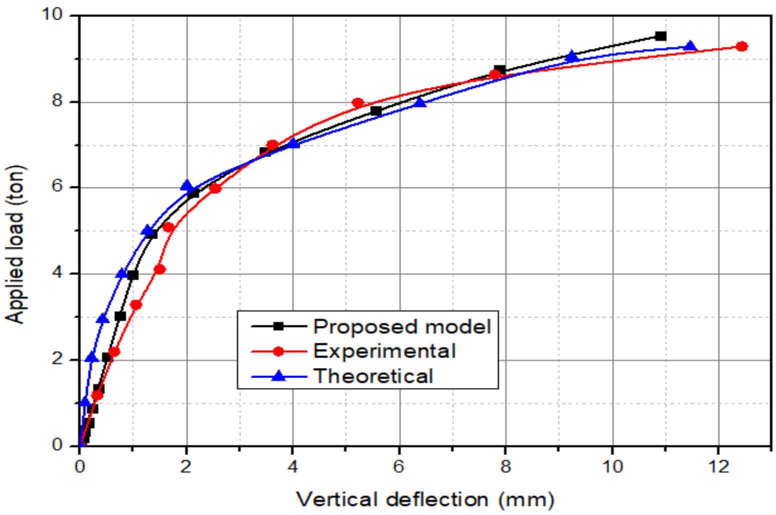
(b)


Applied load (ton)
Horizontal displacement (mm)
Proposed model
Experimental
Theoretical

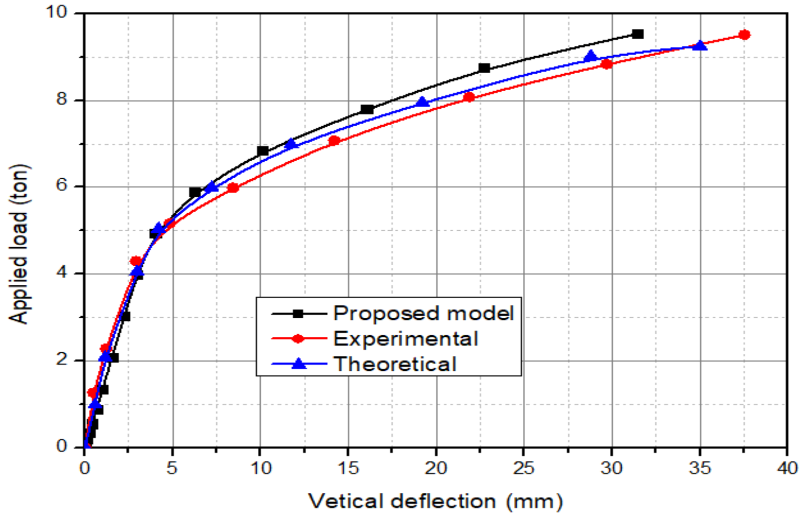
(c)


Applied load (ton)
Horizontal displacement (mm)
Proposed model
Experimental
Theoretical

(d)


Applied load (ton)
Vertical deflection (mm)
Proposed model
Experimental
Theoretical

(e)



(f)

Figure 3.11 Load-displacement curves at different dial positions1, 2, 3, 4, 5 (a), (b), (c), (d), (e) and (f) respectively.

- **Von mises Contours**

ABAQUS outcomes of nonlinear finite element analysis also include the stresses contours, and deformed shape of specimens and will be presented respectively. Figures 3.12 till 3.16. display the FEA results of different elements and positions in the model The von Mises contours are representing the ultimate stresses.

Figure 3.12 shows the stress contours extracted from Von Mises criterion including the configuration of the panel zone under monotonic loading, while Figure 3.13 displays displacement deformed configuration representing the last stage of loading for the reference of the beam to column connection. As can be seen from Figure 3.14 the final configuration of the bolts of the connection including bolts under tension and compression yielding.

Figures 3.12 and 3.13 show the total strain and the plastic stresses in the connection at failure respectively. As expected, the panel zone under goes the majority of strains while bolts are exposed to the maximum plastic stresses in the connection.

Once again, for validation purpose, the obtained results were compared to results of a previous research **[Mashaly 2011 et al]** it must be noted that the obtained results in this study are similar to the ones published undertaken by Ansys **[Khalil .2004]** with almost the same conditions as mentioned earlier.

As can be easily noticed from Figures 3.12, 3.13 and 3.14, most parts of the structure were yielding as the highest stress 979.7 Mpa which is bigger that the yielding stress in the bolts. While for other parts of the structures, most of them were yielding as the imposed stress was clearly largest than their yielding stress that 235 MPa. As far as the panel region, located between the bolts positions, which is highly stress may have local buckling in the shear and compression zones. which leads to a plastic mechanism failure, which is beyond the scope of this research work. While tension region in the upper zone are not concerned by any buckling phenomenon as it is located in tension zone

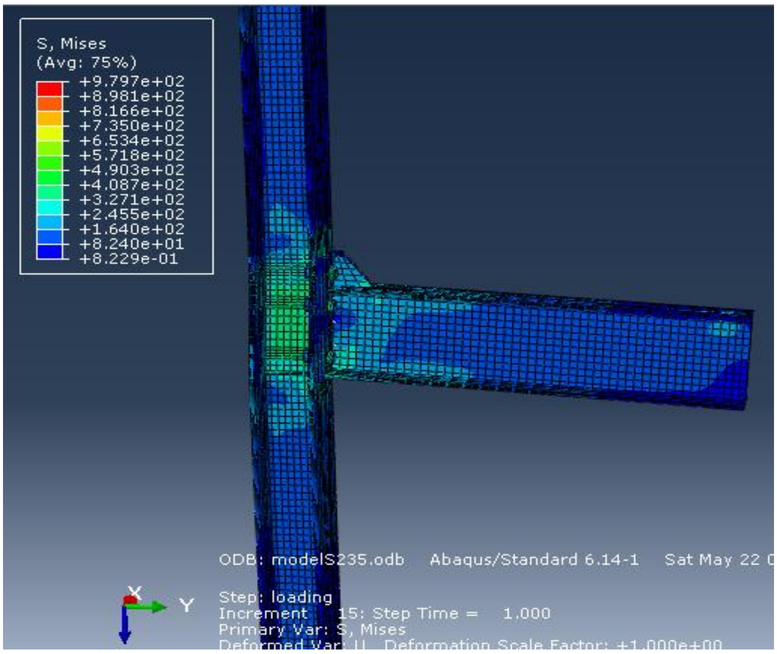


Figure 3.12 Von Mises contours in the connection.

In Figure 3.13, and once again, as the global displacement of the whole structure is concerned, and as expected, the maximum zone is located in the free position and decreases as the position moves toward connection.

As it can be seen from the above figures, the two first rows in tension zone, and as expected, the two bolts in the upper connection undergo high stresses compared to lower bolts in tension zone. However, for compression bolts, higher values are more noticeable in bolts, especially in the shank zone, shown in red colour, for which an exceeding stress larger than the yield stress of bolts with larger zone in the compression bolts.

For deformation aspect, same remarks can be made as for stresses contour, except that the compression bolts are highly deformed compared to the tension's ones. This has a favourable effect as compression does not contribute to the failure of the connection.

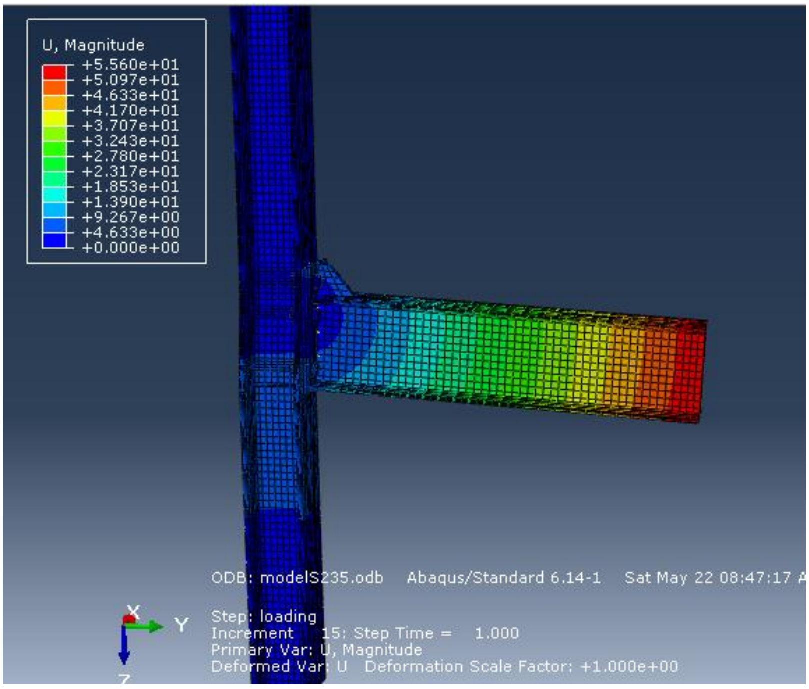


Figure 3.13 Deformed Shape of the Connection Just Before Failure.

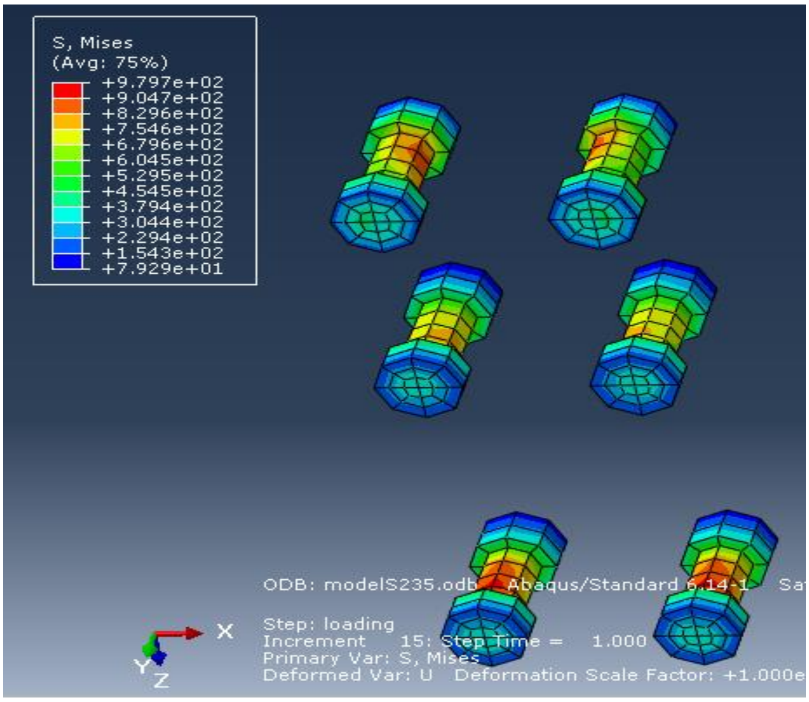


Figure 3.14 Results for stress contour in HR Bolts at ultimate load.

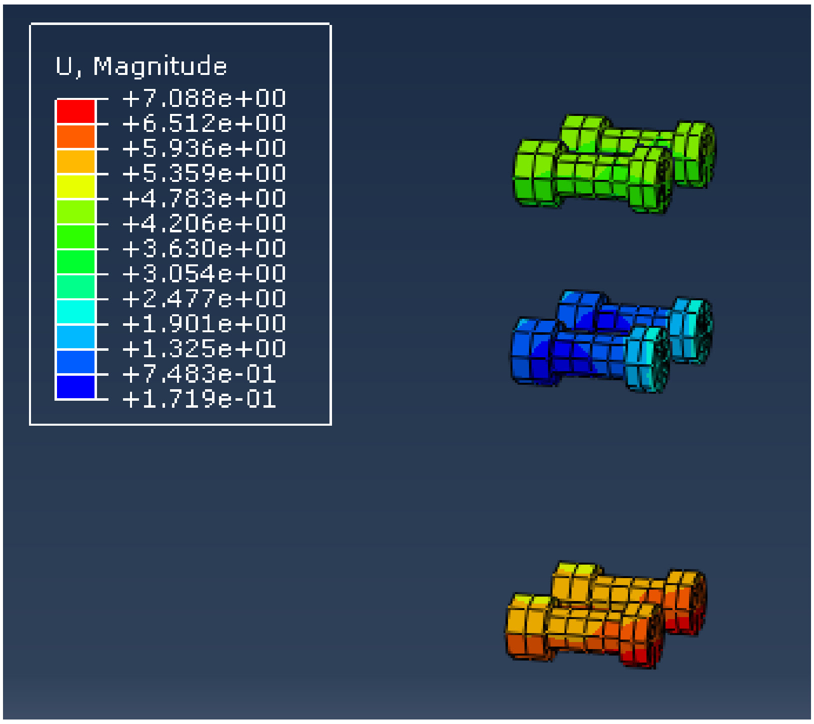


Figure 3.15 Results for deformation contour in HR bolts at ultimate load.

3.5 PARAMETRIC ANALYSIS

3.5.1 Motivation

After the successful task described in the upper section concerning the validation of the model of the nonlinear semi-rigid connection in a beam to column structure, an idea has come to investigate the effect of some key parameters which are thought to be influencing the whole behaviour of a connection in a beam column structure. These parameters studied are: steel grade; bolts grade and placing stiffeners in the panel zone.

In this Section, a parametric analysis is carried out through finite element models implanted in ABAQUS.

N.B It is worth to mention that all conditions in the parametric models are similar to those described in upper section.

3.5.2 Results and discussion

In similar manner, the outcomes of this analysis will be displayed as was the case in the previous analysis. Each single parameter results will be separately presented.

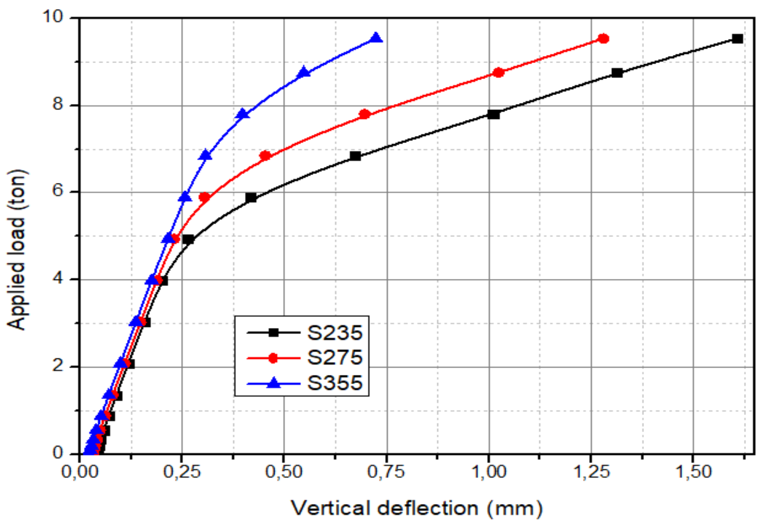
Broadly speaking, it was noticed through this particular analysis, that the parameters investigated has not the same effect on the global behaviour of the structure at different selected positions. Steel grade has the most important effect, the effect of placing stiffeners is noticeable, as expected, in the plastic range without any contribution in the elastic regime. While, changing the bolts grade appears to not having significant effect in both linear and nonlinear behaviours.

- **Steel grade effect**

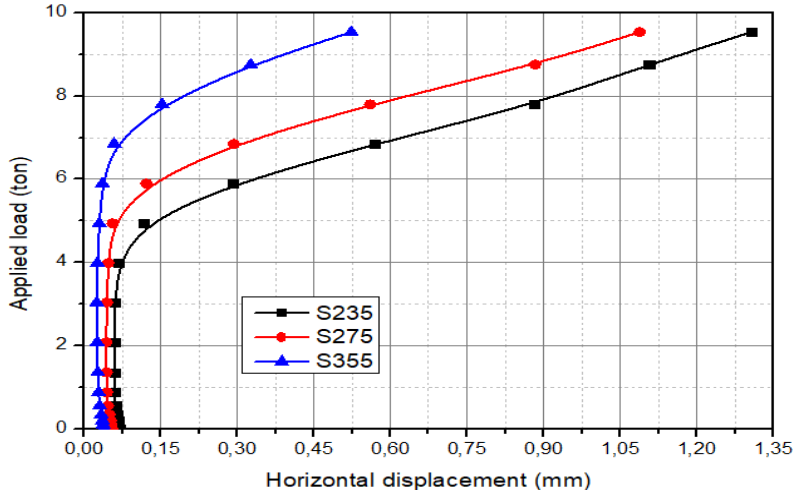
As it can be seen from Figure 3.16 (a) to (f), the effect of steel grade that is S235 as reference grade to S275 and 355, is different from a position to another along with the kind of displacement: vertical or horizontal.

In the elastic behaviour changing the steel grade has no significant differences were remarked, except for position 2, dealing with horizontal displacement were some differences can be noticed. However, in plastic range, it can be seen that the upper grades give more stiffness with reducing displacement under the same loads that is true for all selected positions.

As can be seen from Figure 3.16, the results show clearly the influence of the steel grade parameter when the steel is behaving in static manner, giving as expected, a more stiffness for structures built-up with higher grade with less ductility when highly loaded structures in almost all positions concerned by this study, that is positions 1, 2, 3, 4,5 and 6.

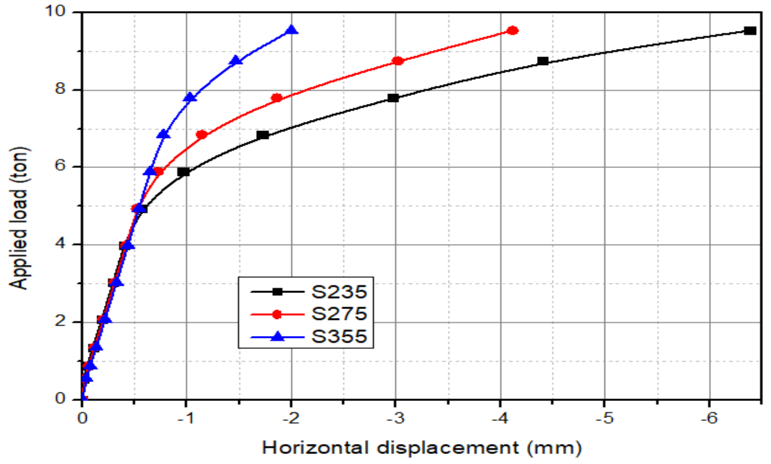


(a)



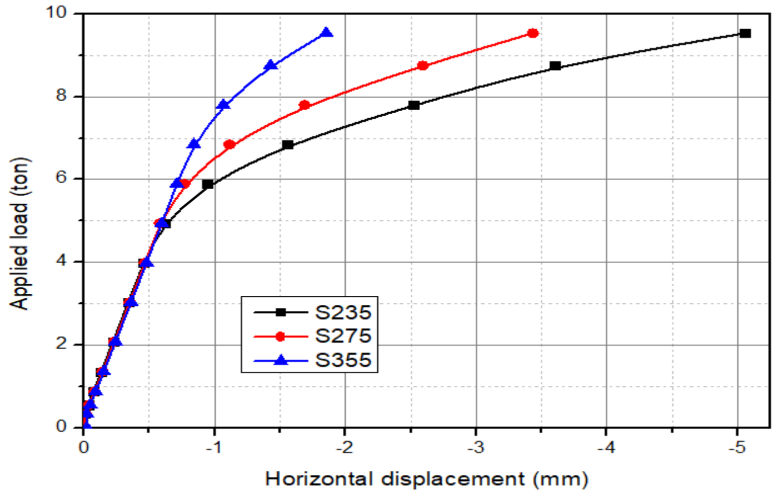
Applied load (ton)
Horizontal displacement (mm)
S235
S275
S355

(b)



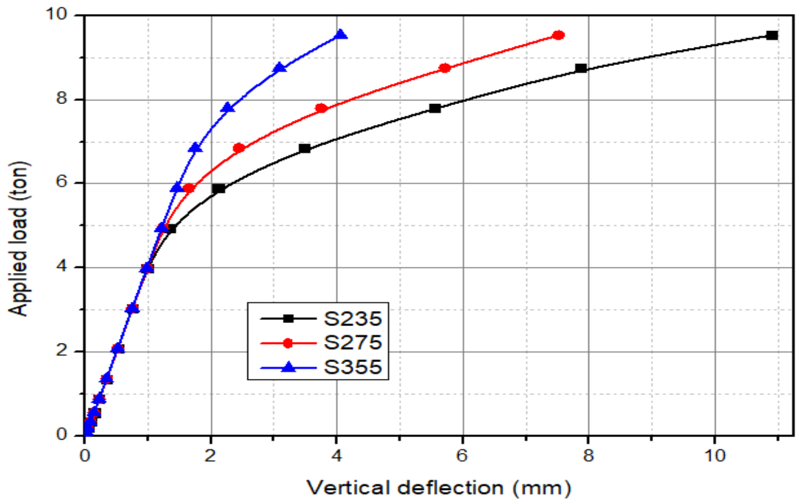
Applied load (ton)
Horizontal displacement (mm)
S235
S275
S355

(c)

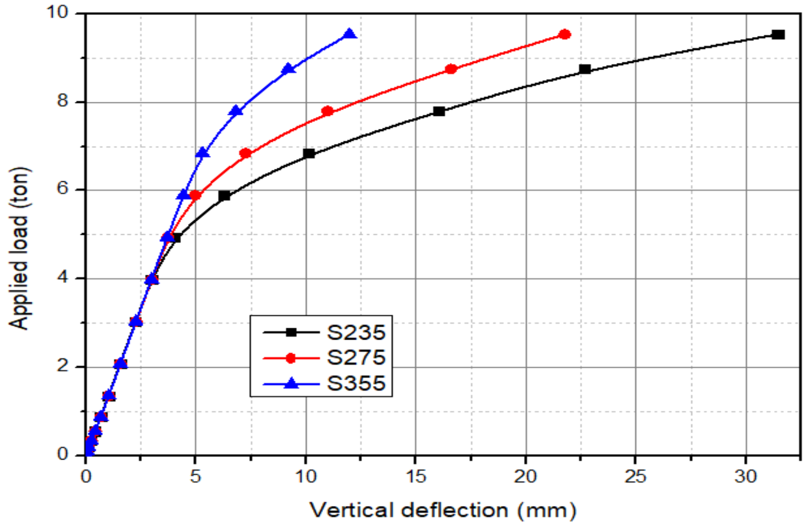


Applied load (ton)
Horizontal displacement (mm)
S235
S275
S355

(d)



(e)



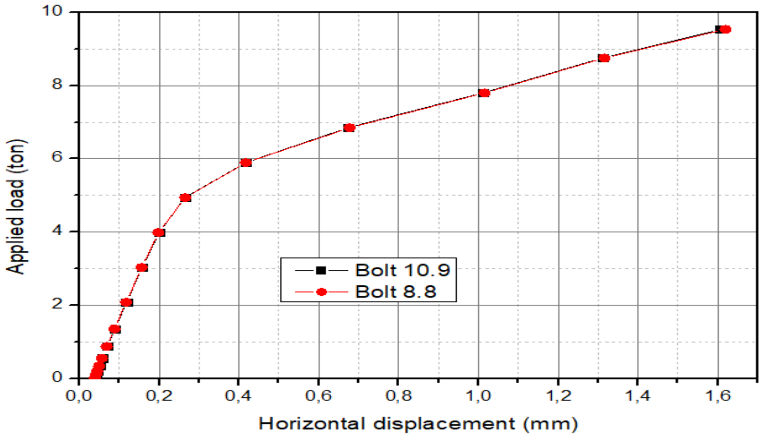
(f)

Figure 3.16 Comparing steel grades in steel connection at different dial positions (a), (b), (c), (d), (e) and (f).

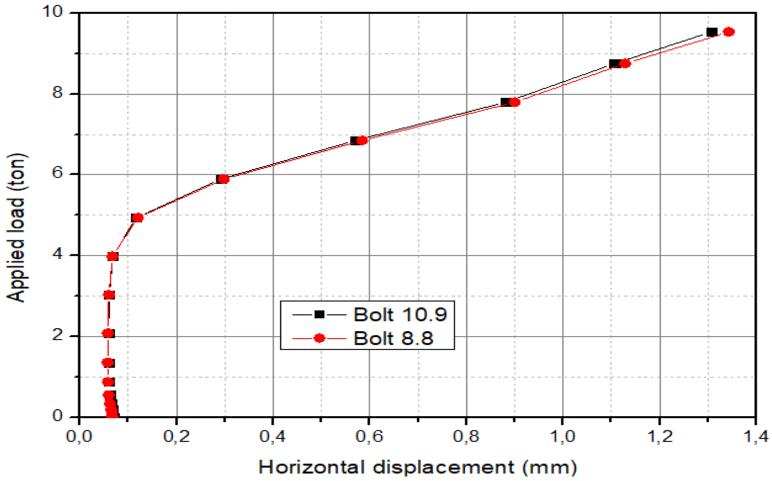
- **bolts grade:**

The second parameter investigated was the bolt grades. According to this research results, changing the bolts grade from 10.9 to 8.8 seems to not have a substantial effect on the global resistance of the connection, as can be clearly shown in relative figure that is Figure 3.17.

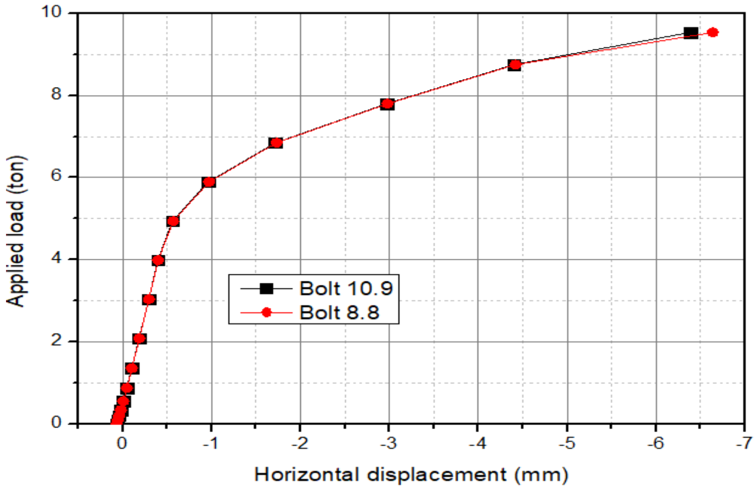
In fact, the bolts show similar behaviour in both elastic and plastic regimes, with practically the same slopes.


Applied load (ton)
Horizontal displacement (mm)
Bolt 10.9
Bolt 8.8

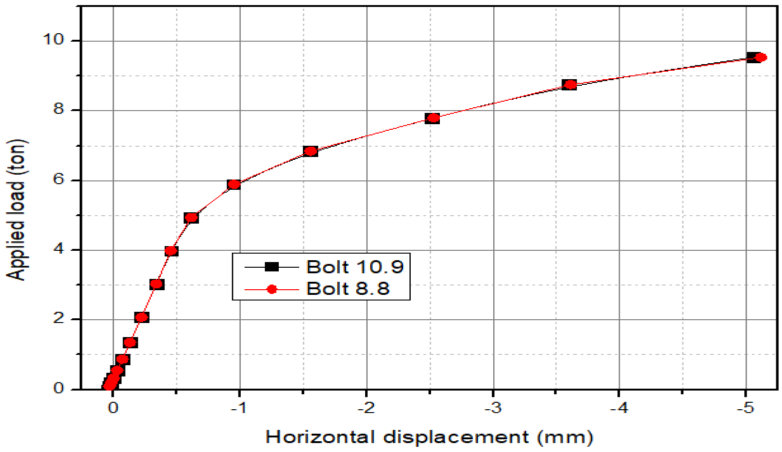
(a)


Applied load (ton)
Horizontal displacement (mm)
Bolt 10.9
Bolt 8.8

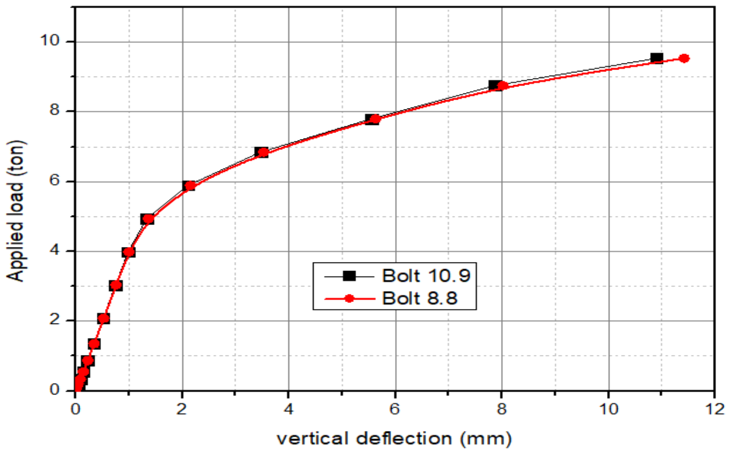
(b)


Applied load (ton)
Horizontal displacement (mm)
Bolt 10.9
Bolt 8.8

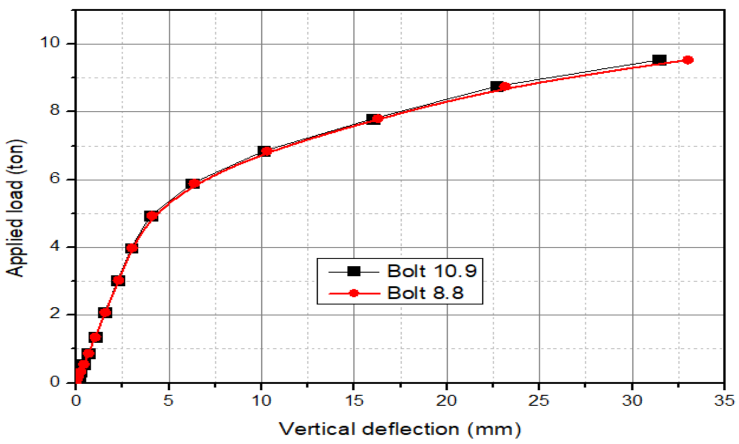
(c)



(d)



(e)



(f)

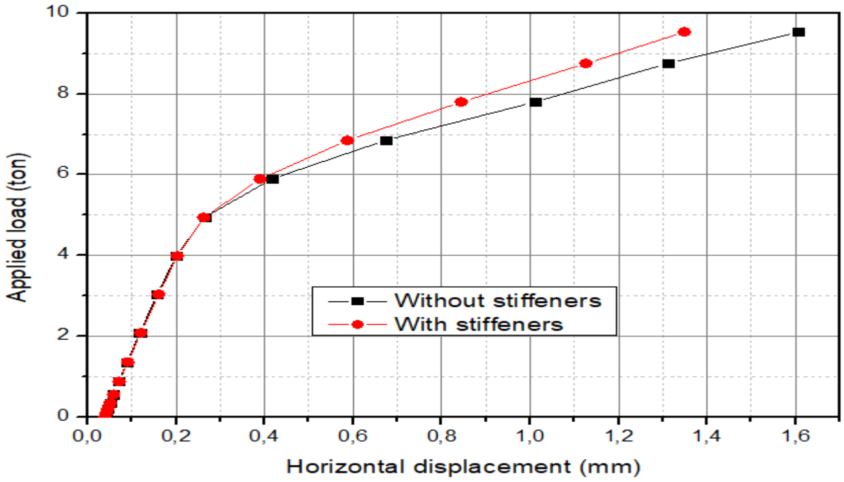
Figure 3.17 Comparing bolt grade in steel connection at different dial positions (a), (b), (c), (d), (e) and (f) respectively.

- **Effect of stiffeners**

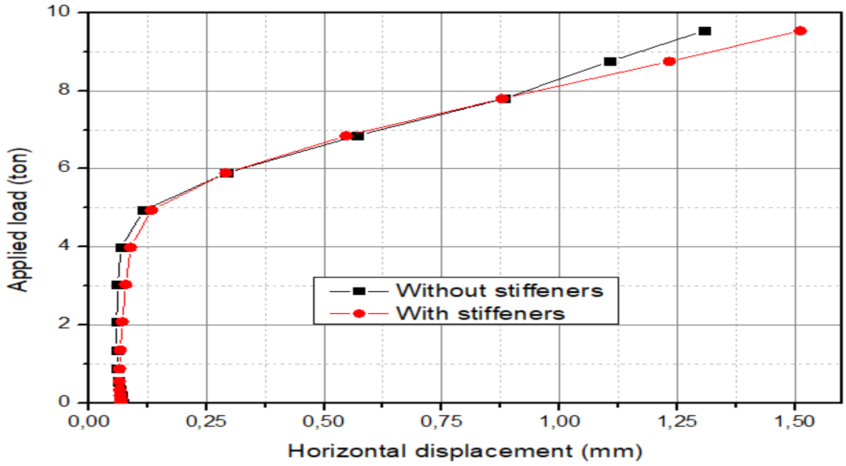
Stiffeners are fundamental details, which can enhance the ductility of the panel plate of the column at the junction by delaying shear buckling of the panel plate webs and allow to prevent inelastic web buckling, which impairs the link performance in the range of the expected ductility demand.

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 EC3.

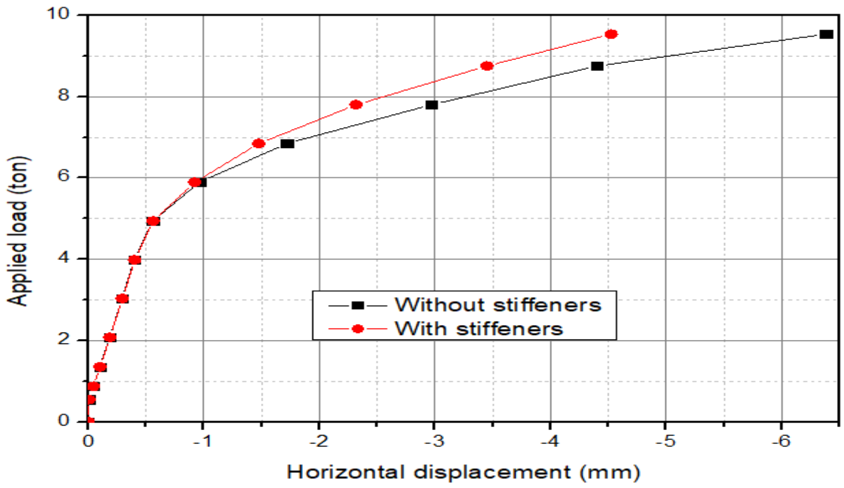
In this study, four stiffeners were added at both sides of the column web for the aim of decrease the local buckling that threatens the column. The effect of stiffeners as showed in figure 3.18 approximately is nil in the elastic zone while in the plastic zone is evident, the deflection decreases in the existence of the stiffeners.



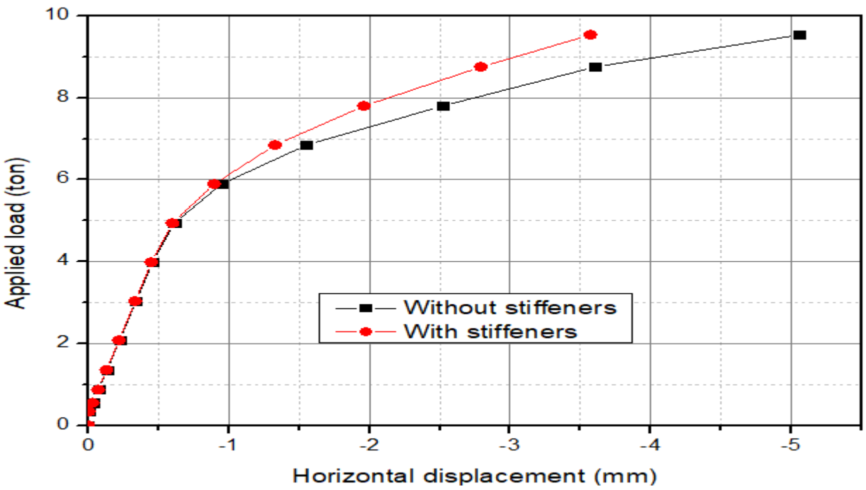
(a)



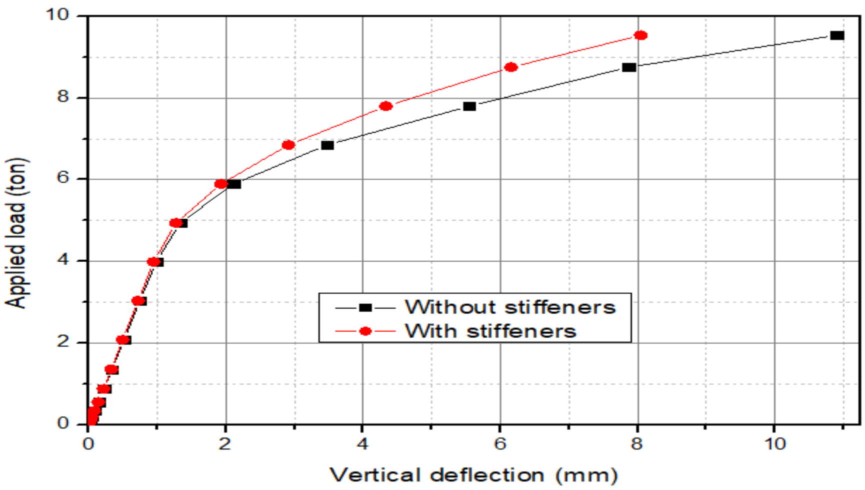
(b)


Applied load (ton)
Horizontal displacement (mm)
Without stiffeners
With stiffeners

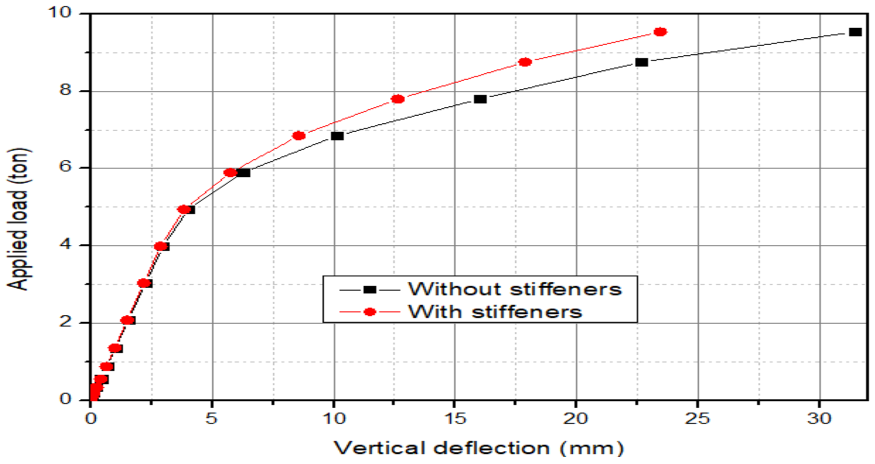
(c)


Applied load (ton)
Horizontal displacement (mm)
Without stiffeners
With stiffeners

(d)


Applied load (ton)
Vertical deflection (mm)
Without stiffeners
With stiffeners

(e)



(f)

Figure 3.18 Effect of stiffeners in steel connection at different dial positions (a), (b), (c), (d), (e) and (f) respectively.

As can deduced from the upper figures, i.e. Figure 3.18; that the effect of placing stiffeners on both sides of the web column does not have the same effect depending on the position of the studied point. However, in the considered cases, this effect is not noticeable while the structure is behaving elastically but some effect can be visible, with favourable effect, especially for positions were the vertical displacement, positions 5 and 6, is plotted vs. the load.

3.6 Conclusion

From this analysis some general conclusions may be drawn. Broadly speaking, the present outcomes of this analysis built-up in ABAQUS software, the validation of models through a comparison with experimental results, and ANSYS results coming from literature in previous research work as fully discussed in the upper sections. Also, it has be proven that steel grade of structural members along with placing stiffeners can improve the behaviour of the structure especially in plastic regime, with an obvious effect in the connection while changing the bolt grade seems to not have any remarkable effect neither in elastic or post-elastic behaviours.

In the following chapter, i.e. chapter 5, a more sophisticated analysis dealing with the same structure but under cyclic loading will be presented.

CHAPTER 4

CYCLIC LOADING ON STEEL CONNECTIONS

4.1 GENERAL

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.

4.2 STEEL BEHAVIOURS

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.

4.2.1 Static behaviour

The stress–strain relationship for steel is shown in Figures 4.1, which present 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.

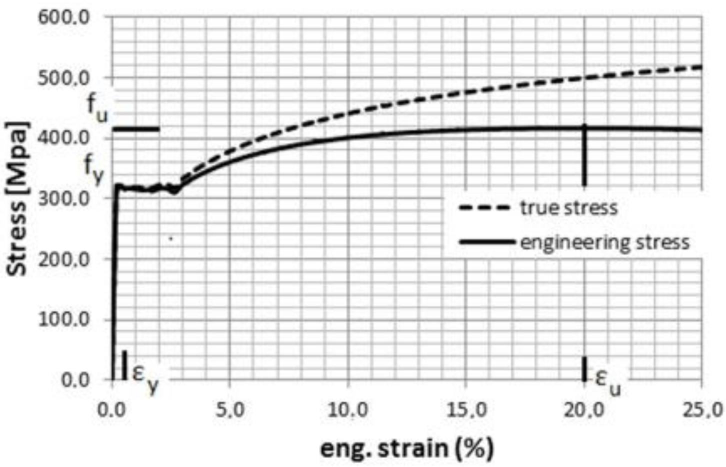


Figure 4.1 Stress–strain curves of steel from tensile test **[Vayas.2019].**

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.

4.2.2 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 .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 4.2(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 4.2(b), (c) and (d).

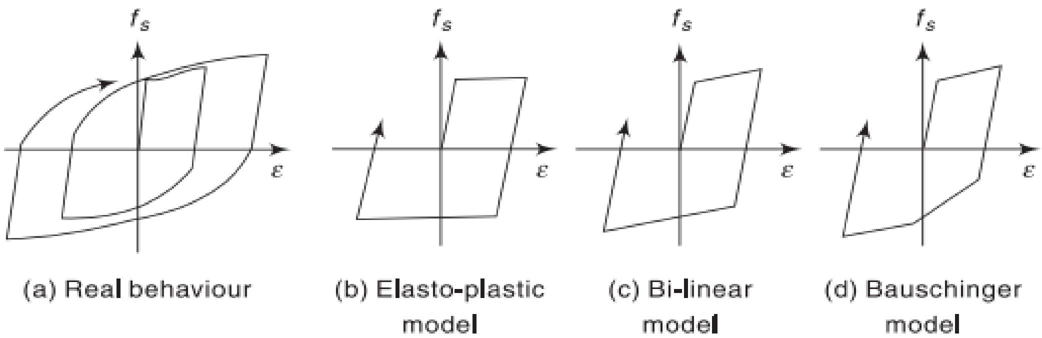


Figure 4.2 Hysteretic behaviour of steel **[Duggal. 2013]**.

4.3 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.

4.4 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 4.3, 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.

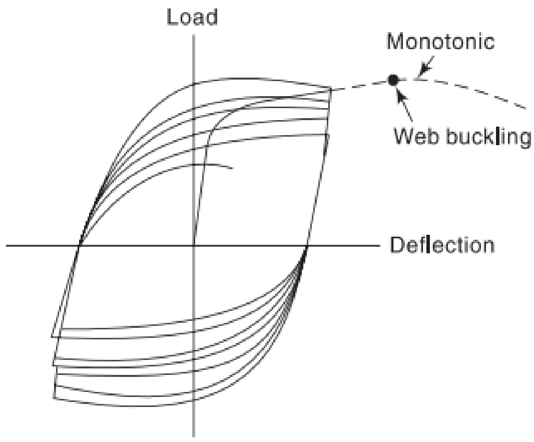


Figure 4.3 Typical hysteresis loops for a steel beam under cyclic bending **[Duggal .2013].**

4.5 LOADING PROTOCOL

4.5.1 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.

4.5.2 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.2009]**. The cyclic loading protocol is 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]**, 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.

4.5.3 Loading protocols

4.5.3.1 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 .

- **ATC-24 Protocol for steel [ATC-24. 1992]**

This protocol showed in figure 4.4, 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.

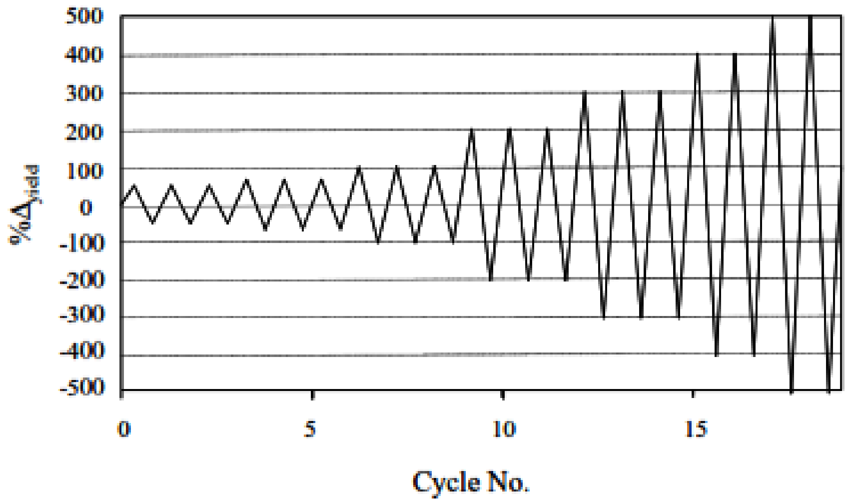


Figure 4.4 Loading protocol for Steel ATC-24 **(ATC-24 1992).**

- ## Steel - SAC Protocol [Clark.1997]

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.

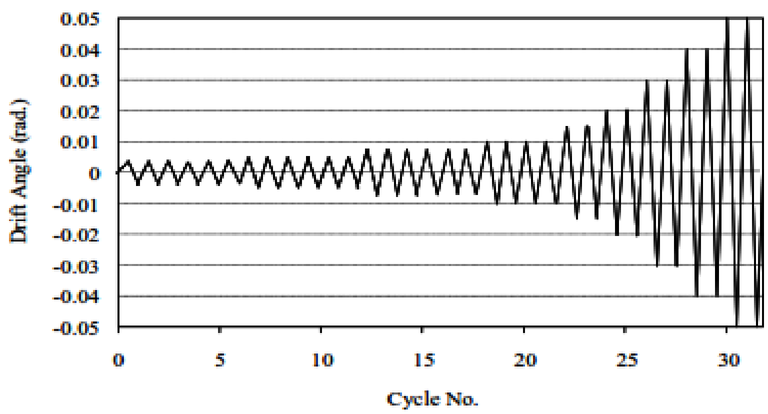


Figure 4.5 Loading protocol for steel SAC **[Clark.1997].**

The SAC protocol (figure 4.5) 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.

- **FEMA 461 [FEMA.2007]**

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 (figure 4.6) 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 $0.3\Delta_m$.

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 4.7 shows Loading protocol specified in AISC 341-16 for testing beam-to-column connections

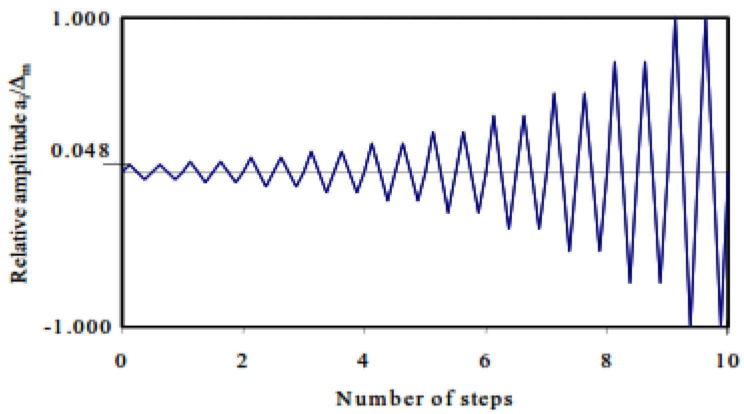


Figure 4.6 Protocol of FEMA 461 **[FEMA.2007].**

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

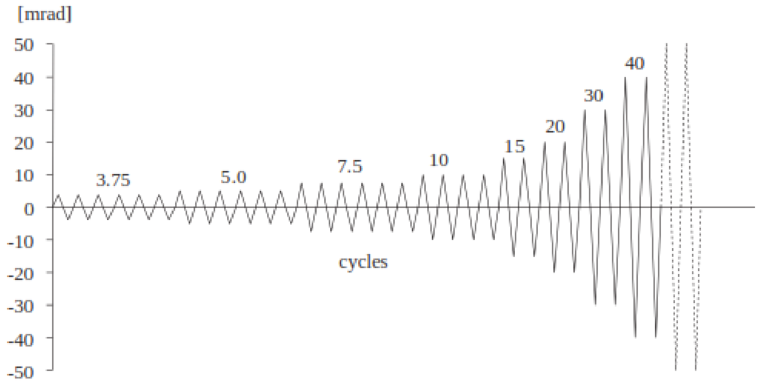


Figure 4.7 Loading protocol specified in AISC 341-16 for testing beam-to-column connections.

- 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 θ = 0.00375 rad

(2) 6 cycles at θ = 0.005 rad

(3) 6 cycles at θ =0.0075 rad

(4) 4 cycles at θ = 0.01 rad

(5) 2 cycles at θ = 0.015 rad

(6) 2 cycles at θ = 0.02 rad

(7) 2 cycles at θ = 0.03 rad

(8) 2 cycles at θ = 0.04 rad

Continue loading at increments of θ = 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 γ_{total} = 0.00375 rad

(2) 6 cycles at γ_{total} = 0.005 rad

(3) 6 cycles at γ_{total} = 0.0075 rad

(4) 6 cycles at γ_{total} = 0.01 rad

(5) 4 cycles at γ_{total} = 0.015 rad

(6) 4 cycles at γ_{total} = 0.02 rad

(7) 2 cycles at γ_{total} = 0.03 rad

(8) 1 cycle at γ_{total} = 0.04 rad

(9) 1 cycle at γ_{total} = 0.05 rad

(10) 1 cycle at γ_{total} = 0.07 rad

(11) 1 cycle at γ_{total} = 0.09 rad

Continue loading at increments of γ_{total} = 0.02 rad, with one cycle of loading at each step.

4.5.3.2 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 et al.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 4.8 and 4.9):

- **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.

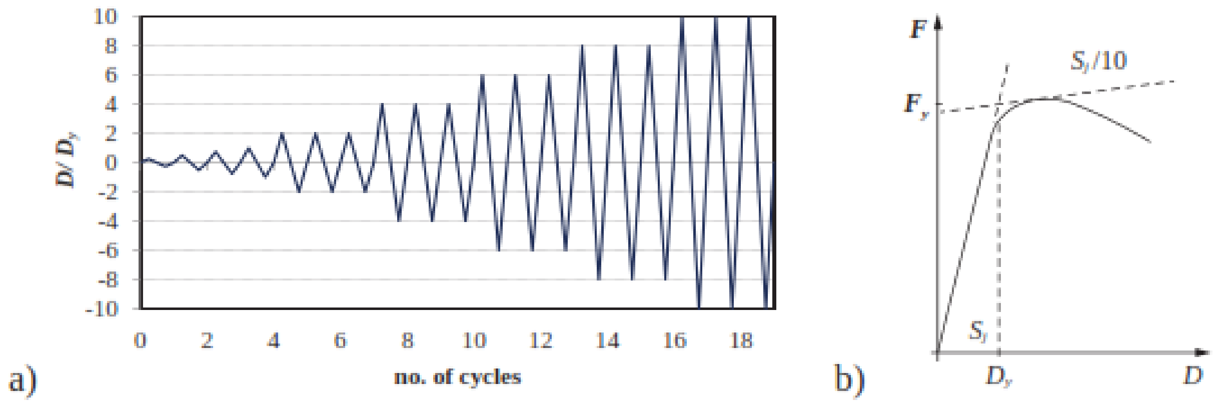


Figure 4.8 Cyclic loading protocol (a) determination of yielding displacement **[Landolfo et al .2017].**

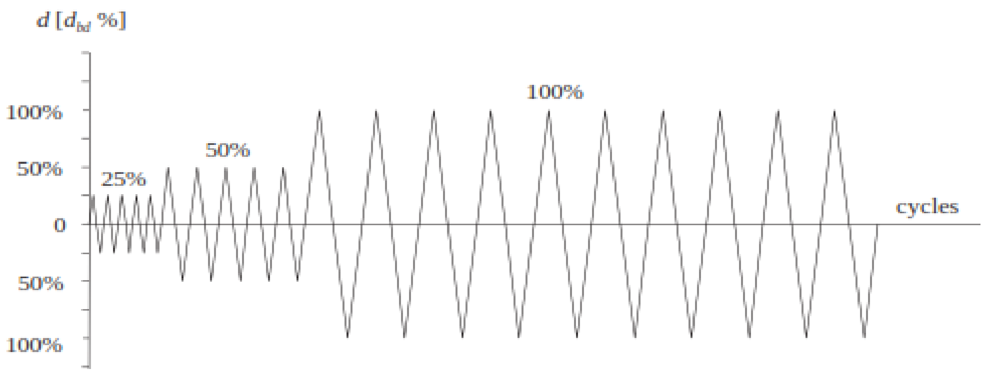


Figure 3.9 Loading protocol specified in EN 15129 **[Landolfo et al .2017[.**

4.5.4 General requirements for a cyclic protocol[AISC.2005]

The test specimen shall be subjected to cyclic loads according to the requirements for beam-to-column moment connections in special and intermediate moment frames, and according to the requirements for link-to-column connections in eccentrically braced frames. Loading sequences other 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.

4.6 NONLINEAR ISOTROPIC/KINEMATIC HARDENING MODEL

The nonlinear isotropic/kinematic hardening model was used to model the material response in the plastic range. This material model consists of two components: a nonlinear kinematic hardening component, and a nonlinear isotropic hardening component. The isotropic hardening component of the model defines the evolution of the size of the elastic range from cycle to cycle. The material parameters which define the isotropic hardening component of the material behavior are as follows:

σ_y = the elastic yield stress

Q = the maximum change in the size of the elastic range

b = the rate at which the size of the elastic range changes as plastic straining develops.

Von Mises yield criterion is frequently used to determine the yield point (the point at which plastic deformation begins) for isotropic metals. This is a distortional energy criterion. It states that yielding occurs when the distortional strain energy density for a multi axial stress state equals the distortional strain energy density at yield in uniaxial tension.

Hysteresis energy plots were generated in ABAQUS by generating stress versus time and strain versus time curves and then combining them to create stress-strain curves for the duration of the loading history.

The stresses and strains used for these plots were total stress and total strain at the centroid of flange elements oriented along the longitudinal axis of the beam .These stress-strain hysteresis curves were then integrated to calculate the area within the hysteresis curves.

The ABAQUS finite element program can be used to accurately model local buckling mode of failure in plastic hinge regions of beams subjected to cyclic loading. The results of this study indicate that the global response of the model in terms of displacement and rotation determined by the ABAQUS program match the laboratory results very well. The program also accurately modeled the size and location of the local buckles.

4.7 YIELD CRITERION

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.2010].**

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].**

The von Mises yield criterion (the classical metal plasticity theory) which is most commonly applicable to initially isotropic engineering materials, is used to predict the onset of the yielding. The behaviour upon further yielding is predicted by the "flow rule "and "hardening law".

The associative flow-rule for the von Mises yield criterion, i.e. Prandtl-Reuss flow equations is used along with hardening of steel sections and bolts to model the Bauschinger effect. Kinematic hardening is assumed for modelling of the steel connection assuming that the yield surface only transfers in the direction of yielding and does not grow in size. 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].**

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].**

It must be mentioned that Von Mises criteria is worldly accepted for steel structures as per EC3, EC8, AISC, UBC etc.

4.8 CYCLIC BEHAVIOUR OF BEAM TO COLUMN JOINTS

4.8.1 General

The key points in the behaviour of Moment Resisting Steel Frames are the beam-to-column joints, located near the dissipative zones. These should possess adequate rotation capacity and resistance in order to resist the earthquake action.

The most widely observed type of failure was cracking in the region of beam-to-column joints of moment resisting frames. As such joints were formed according to different design and construction practices, it is considered worthwhile to examine types of joints other than rigid, where the strength demands are more evenly distributed along the girders.

4.8.2 Joint cyclic behaviour

The structural joints have a major role in the seismic behaviour of framed structures. In fact, a good joint is characterised by high load bearing and with respect to deformation by the absence of clearance, high initial and creep stiffness as well as by ductility and energy dissipation in case of cyclic and seismic loading. An unexpected brittle failure of connections and, in some cases, of members occurred during the last earthquakes of Northridge (1994) and Kobe (1995).

In seismic areas, characterised by repeated load reversal, the resulting joint response should remain as symmetrical as possible, an imposition much harder to ensure in common beam-to-column joints, given the asymmetry of the joint with respect to the centroidal axis.

The behaviour of joints under cyclic loading, when compared to the corresponding static monotonic response, presents the added difficulty of degradation of strength and stiffness in successive loading cycles.

The cyclic behaviour of a joint is always unstable, exhibiting a progressive degradation of its mechanical properties (strength, stiffness and energy dissipation capacity) **[Mazzolani et al 1994].** The behaviour of joints under cyclic loading, when compared to the corresponding static monotonic response, presents the added difficulty of degradation of strength and stiffness in successive loading cycles

4.8.3 End-plate cyclic behaviour

End-plate-type joints are widely used in steel frame structures, connecting either two steel elements (like beam-to-column, beam-to-beam or column-to-column joints) or a steel and a concrete/reinforced concrete element (like column-base joints or joints of a steel beam and a reinforced concrete column). Although these joints have numerous practical advantages, their application results in a more complicated structural behaviour, which must be considered in the design.

The complete understanding of the cyclic behaviour of end-plate joints is essential, especially in the seismic design. The importance of the problem was clearly justified during the recent earthquake events, where significant structural damage of steel frames took place in the connection zones in several cases. Thus, it is important to understand and simply but reliably asses the behaviour of the joints in case of seismic actions, in order to satisfy the required resistance, rigidity, ductility and energy absorption demands.

4.9 STIFFENERS

4.9.1 General

Stiffeners are secondary plates or sections which are attached to beam webs or flanges to stiffen them against out of plane deformations. Almost all main bridge beams will have stiffeners. However, most will only have transverse web stiffeners, i.e. vertical stiffeners attached to the web.

Stiffeners in the form of transverse are necessary to maintain the cross- sectional contour, especially in the presence of high local loading, such as when concentrated transverse forces are acting with regard to fatigue strength when designing structures.

Stiffeners are fundamental details that can enhance the ductility of the panel plate of the column at the junction by delaying shear buckling of the panel plate webs and allow to prevent inelastic web buckling, which impairs the link performance in the range of the expected ductility demand.

4.9.2 Design of stiffeners

It is usually necessary to stiffen the webs of plate girders to prevent loss of strength due to web buckling. 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.

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]**. Panel web stiffeners, as previously stated, should be designed to prevent inelastic web buckling under large rotation 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. The design strength of fillet welds connecting the stiffener to the flanges should be larger than $\gamma_{ov}\ f_y\ A_{st}/4$. 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.

CHAPTER 5

NUMERICAL ANALYSIS OF CYCLIC PERFORMANCE OF BEAM TO COLUMN SEMI-RIGID CONNECTIONS

5.1 INTRODUCTION

The main aim was to investigate the carrying and energy dissipation capacity capacities with the influence of web stiffeners in column section of a beam to column connection.. In this chapter, numerical nonlinear cyclic investigation was conducted through finite element model implanted in ABAQUS software for a semi-rigid connection in a beam to column structure. The steel connections used in MRFs have high ductility, good seismic behaviour, easy retrofit, lightweights. The steel moment resisting frames MRFs are generally preferred in major earthquake-prone areas due to their high ductility and strength as compared to other construction practices. However, following the Northridge in 1994 and Kobe earthquake, a significant number of steel moment resisting structural systems were damaged. Also, an unexpected brittle failure of connections and, in some cases, of members occurred during the last earthquakes of Northridge (1994) and Kobe (1995). The most crucial component of MRFs is the beam-column connections, especially in seismic events. The conventional practices to design the connections, consider it as rigid with the infinite stiffness in order to fulfil the safety considerations of high stiffness and adequate over-strength regardless of the cost of construction. These conventional practices generally design these frames to remain almost in the elastic range.

5.2 MODELLING THE CYCLIC BEHAVIOUR OF BEAM TO COLUMN CONNECTIONS

5.2.1 General

Several studies have been conducted to improve the seismic performance of steel structures and specially their beam-to-column connections. A typical natural event that, for simplicity, is usually approximated by cyclic loading is an earthquake. Usually, seismic events provoke relatively high amplitudes of rotation in the joint area, so that steel repeatedly reaches the plastic range and the joint fails after a relatively small number of cycles. This typical behaviour is usually called oligo-cyclic fatigue, in close analogy with the behaviour of steel under repeated cyclic loading stressed into the plastic range.

Because of the topological complexity of connections and a large number of possible yield mechanisms, the cyclic behaviour can vary significantly even within the same connection type. Semi-rigid connections have been examined as alternatives to welded connections for seismic regions due to their adequate ductility to dissipate large amounts of energy with a minimal loss of strength or stiffness.

5.2.2 Modelling the cyclic behaviour

The aim of using cyclic loading is to determine the suitability of end-plate displacement for use in seismic force resisting frames. There are four different approaches to model the cyclic behaviour of beam-column connections:

1) Phenomenological modelling;

2) Mechanical modelling;

3) Refined three-dimensional finite element modelling and

4) Neural network (NN) based modelling approach. In this section, the existing modelling approaches are briefly reviewed and a neural network-based modelling approach combined with mechanical model is introduced in more detail.

5) Finite element modelling

As it was reported in many research works, the three-dimensional finite element analysis can reasonably predict the cyclic behaviour of the test specimen until tearing of the beam flange occurs. However, the three-dimensional finite element analysis has difficulties in reproducing the post-limit behaviour. As can be seen from Figure 5.4, a remarkable regularity and stability of the hysteresis loops up to failure, with no deterioration of stiffness and strength properties characterize the cyclic behaviour of bolted joints. Significant distortion of the joint panel zone has been observed though no noteworthy plastic deformation occurred in the column.

In this research work, we are concerned with the 3D finite element modelling. Three-dimensional finite element model is the most accurate approach to predict the cyclic response of beam-column connections. Recently, many general-purpose nonlinear finite element analysis packages and advanced finite element mesh generation tools are routinely available, such as ABAQUS, ANSYS, etc. For detailed modelling of components of the connection, modelling techniques such as metal frictional contact, assembly torque, geometric and material nonlinearity are easily employed in complex three-dimensional finite element models. With such a detailed finite element model, realistic responses under cyclic loadings can be simulated by classical metal plasticity model with mixed hardening definitions such as kinematic and non-linear isotropic hardening model.

A number of studies on three-dimensional finite element analysis of beam-column connections have been reported in the literature. However, there are several drawbacks in the three- dimensional finite element models of connections subjected to cyclic loadings. While the approach can provide the most accurate prediction of the capacity of connections, the computational time and cost are enormous and there are still unresolved issues with respect to modelling of post-yield behaviour such as local buckling, fracture and tearing of components

5.3 LOADING PROTOCOL

5.3.1 SAC loading protocol

After the unexpected failure of numerous fully-welded beam-to-column connections during the 1994 Northridge California earthquake, a significant amount of the research was made through the SAC Joint Venture (SAC is a steel project funded by the Federal Emergency Management Agency FEMA-USA to solve the problem of the brittle behaviour of steel frames when subjected to seismic loads and commonly known as the *SAC Steel Project*) **[Mohammed.2009].**

The model was loaded cyclically according to standard load history recommended by SAC. In the SAC basic loading history, the control parameter is inter-story drift angle.

As illustrated in figure 5.1, the connections which are analysed by the SAC basic loading history, need to endure 29 damage cycles in order to reach the maximum target rotation of 0.04 radians, including one complete cycle in this deformation range. At this rotation level, the target cumulative rotation of the loading history must reach 0.59 rad. This loading protocol is developed based on the target values of the study done by [Krawinkler et al 2000]. Therefore, this loading protocol has a good consistency with the target values of the study. This loading protocol not only imposes the deformation demand to the connection in the maximum target deformation range of 0.04 rad, but also incorporates the demanded number of the damage cycles in this rotation (29 damage cycles).

Table 5.1 SAC loading protocol SAC 2000.

SAC loading protocol				
Loading step	Number of cycles in the step	Cumulative number of cycles	Amplitude of inter-story drift angle (rad.)	Cumulative rotation (rad.)
1	6	6	0.00375	0.045
2	6	12	0.005	0.105
3	6	18	0.0075	0.195
4	4	22	0.01	0.275
5	2	24	0.015	0.335
6	2	26	0.02	0.415
7	2	28	0.03	0.535
8	2	30	0.04	0.695

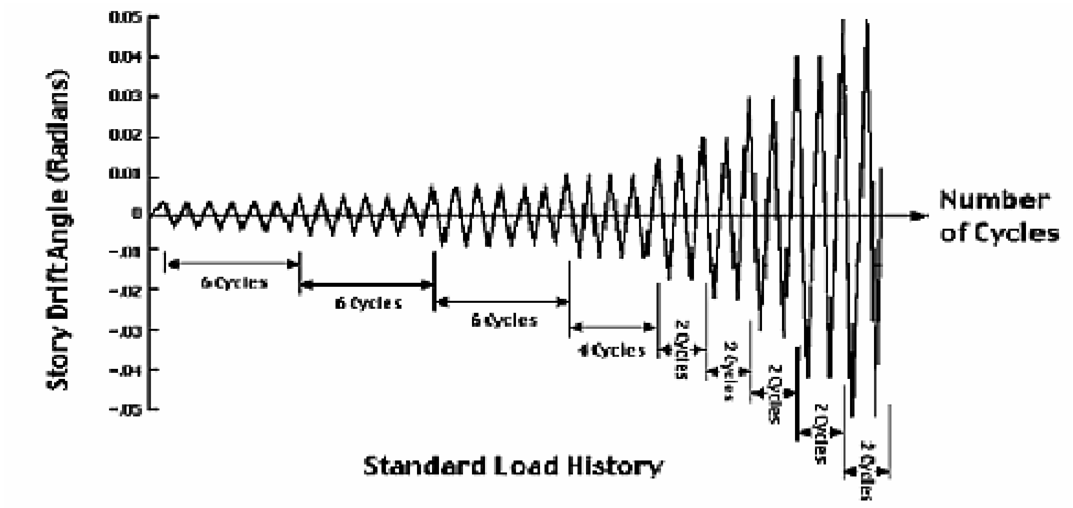


Figure 5.1 Load history SAC 2000.

5.3.2 Actual history input data

As explained above, the model was subjected to cyclic protocol (SAC) in fact to get the displacement of beam column connection under the cyclic force, the values of displacement were picked from the relation $\Theta = (\Delta / L)$

Where:

Θ the amplitude (rad)

Δ the displacement (mm)

L the length of the beam (mm) / L=1000mm.

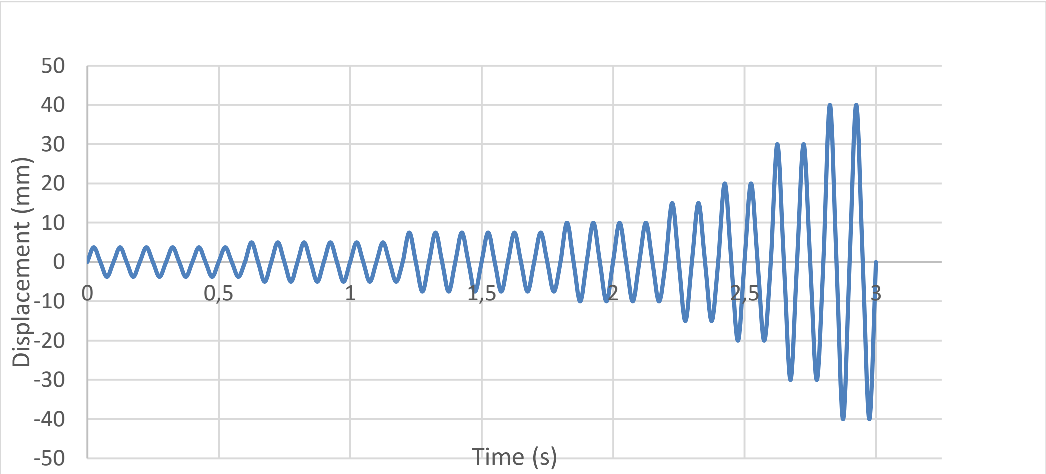


Figure 5.2 Actual Load displacement history.

Table 5.2 SAC Actual loading protocol used in this study.

Loading step	Number of cycles	Displacement (mm)
1	6	3.75
2	6	5
3	6	7.5
4	4	10
5	2	15
6	2	20
7	2	30
8	2	40

N.B. Figure 5.2 and table 5.2 give the actual data of cyclic loading used in this study.

5.4 FINITE ELEMENT MODEL

5.4.1 General description of the model

In this study in order to investigate the numerical behaviour of connections, ABAQUS ver. 6.10 software was used. Numerical modelling of the all connections is carried out under the assumptions described in the above section including dimensions and geometry of the beam, column and connection components. All the components including beam, column, shear tab, bolts and weld materials are modelled using three-dimensional solid elements to reproduce experimental results. All components of the model are modelled using (SOLID C3D8R) 8-node linear brick, reduced integration with hourglass control. Therefore, to consider the frictional forces, Coulomb's coefficient is assumed to be 0.30, which yields the best results. The mechanical properties of all component materials are taken from the experimental specimens listed in Table 3.1. The load is applied in two steps. Bolt pre-tension is applied as the first load case by "Bolt load" option on the bolt shanks to yield equivalent pre-tension force.

The whole design procedure has been carried out with reference to S235 steel grade as fully described in chapter 3. Regarding beam and column elements, a bilinear model characterized by a hysteretic behaviour with no stiffness degradation, no ductility-based strength decay, no hysteretic energy-based decay and no slip has been considered.

5.4.2 Meshing

To achieve computational efficiency and convergence, a fine mesh was used in zones where large inelastic incursions were expected and a coarser mesh was used in the other zones. In summary, the whole characteristics of the model used in this study, Table 5.3 provides the essential of data. Also, the models were analysed with the following consideration: the diameter of the holes was assumed equal to the diameter + 2mm of the bolts.

One finite element type (SOLID C3D8R) 8-node linear brick, reduced integration with hourglass control was used in the modelling of all elements (beam, column, end-plate, bolts and stiffeners).

- Total number of nodes: 71585
- Total number of elements: 54908
- 54908 linear hexahedral elements of type C3D8R

Table 5.3 General characteristics of the studied model.

Parts	Number of nodes	Number of elements
Column	34191	26696(C3D8R)
Beam	30613	23520(C3D8R)
End-plate	4455	3236(C3D8R)
bolts	201	128(C3D8R)
stiffener	280	172 (C3D8R)
Total	71585	54908(C3D8R)

Figure 5.3 shows the FE model and mesh pattern of the connection. In order to improve the accuracy of numerical modelling, Finger shims was modelled in the contact place of the bracket to column flange similar to their actual specimens. The geometrical discontinuities are simulated by surface-to-surface contact interaction. This interaction, was used for contacts between the end-plate and column flange, finger shims and brackets, finger shims and column flange, bolts and bolt holes. The surface to surface contact interaction, allows surface separation after collision.

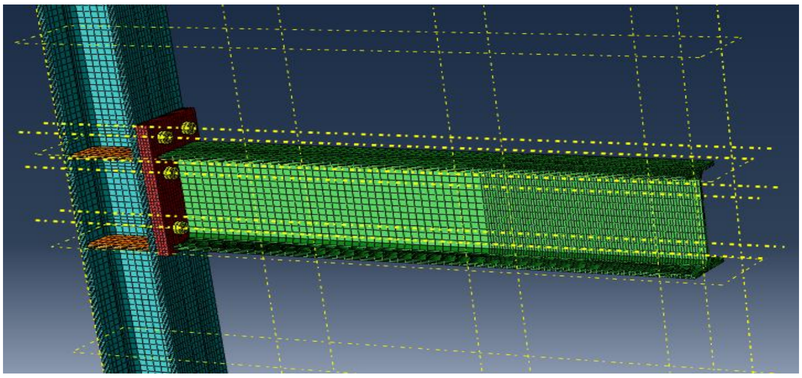


Figure 5.3 General layout of the studied beam to column connection.

5.4.3 Boundary conditions and loading

In the studied model, boundary conditions were shown in Figure 3.6. The numerical model was subjected to the loading history specified in SAC 2000. Incremental cyclic loading history was applied up to 40 mm story drift angle in accordance with SAC 2000. The interaction between elements of connection was deemed through difrent types of contacts. A “Frictional” contact with a 0.3 friction coefficient was used to simulate the contact between end plates beams and column.

5.4.4 Results presentation of the FE model

On the basis of the upper design data of steel beam-column moment joint, 3-D nonlinear finite element model was established using general finite element program ABAQUS to analyse the mechanical properties of the connection. The effects of both geometrical and material nonlinearities are taken into account.

- **General remarks**

Bearing in mind the huge amount of different results coming from ABAQUS, it is obvious that only part of results will be presented and discussed, this is due to the lack of time given to this research work to be fined and submitted.

It must be noted, that generally speaking, the prediction the behaviour of steel and composite joints are quite complex as it combines several phenomena such as material non-linearity (plasticity, strain-hardening), non-linear contact and slip, geometrical non-linearity (local instability), residual stress conditions and complicated geometrical configurations and is further complicated by the successive static loading and unloading under cyclic loading. The first consists of the management of the hysteretic cycles, where a clear is distinction between positive and negative moment must be made because of possible asymmetry of joint response under hogging or sagging bending.

The behaviour of steel or composite joints under cyclic loading is characterized by hysteretic loops with progressive degradation of strength and stiffness that eventually lead to failure of the joint. In the conventional analysis and design of steel framework, the behaviour of connections is idealized as perfectly rigid or ideally pinned. However, numerous experimental investigations have clearly shown that actual connections behave nonlinearly due to gradual yielding of connection components such as plates and angles, bolts, etc. A beam-to-column connection is generally subjected to axial force, shear force, and bending moment.

As it is known, modern seismic codes **[CEN.2005],** promote the dissipation of seismic input energy by properly designed so-called dissipative zones that are some zones of structural members engaged in plastic range, properly detailed in order to assure wide and stable hysteresis loops.

- **General discussion**

The present study addressed characterizing hysteresis response of end-plate beam-to-column connection. The beam-column connections implemented in this study are supposed to be semi-rigid ones having an isotropic hysteresis behaviour. In this section cyclic behaviour of steel beam-column connections is discussed focusing on their seismic performances. Accurate modelling of the cyclic behaviour of connections is very important in evaluation of seismic performances and design of steel moment-frame buildings MRFs.

- **Results presentation**

In the following, results coming up from FEA model will be ^presented for each component of the structure in such manner that each component will have a particular interest. That is for each component, at the first stage, the stress contours of principal stresses followed by shear stresses will be first presented and discussed. Then, the end plate results with the influence of holes in the global behaviour and in similar manner the presentation of the obtained results in column part of the structures. A particular interest on the panel zone, as being the weakest part of the connection, with more figures depicting all aspects of its behaviour with the influence of placing transverse stiffeners. Then at last, but not the least, the behaviour of bolts will be presented and discussed.

- **Hysteretic curves**

In the following, the FEA outcomes will be presented in forms of the hysteretic curves computed at each of the six locations considered in this study, see Figure 5.4. In this study, the hysteretic energy dissipation capabilities of the joints for various levels of ductility were determined, and the mechanisms of failure were identified, see Figure 5.4 To identify the measure of energy dissipation for a given connection, cyclic numerical analysis is conducted to obtain the load-deflection hysteresis loops. The hysteresis behaviour in the lateral load-displacement relationships are plotted in Figure 5.4.

According to the definition of hysteresis, one strain value is corresponding to multiple stresses. The system capacity to resist a seismic event, mainly due to hysteretic energy dissipation capacity. The energy dissipation of the specimens was computed from the area hysteretic loops at each cycle. The area under the outer loop of the hysteresis loops, at failure, is an indicator of the energy dissipation capability of a connection

As can be seen in Figure 5.4, hysteresis curves are similar shape for positions 5 and 6 as they compute the vertical displacement while positions 1, 2, 3 and 4 show quite different hysteretic curves shapes for other computed positions.

Indeed, and in summary Figure 5.4 shows the hysteresis curves obtained from the numerical investigation.

Hysteresis: The property of a steel structure or element by which it dissipates strain energy through plastic straining.

Hysteresis loops of frames under repeated horizontal load, having the same monotonic response and displacement capacity δ_u.

Figure 5.4: (a) Representing the hysteresis curves of reference of the position 1.

(b) Representing the hysteresis curves of reference of the position 2.

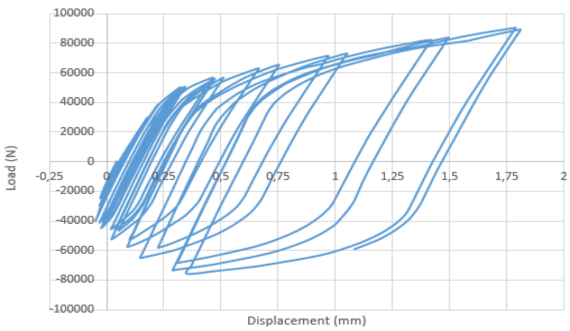
(c) Representing the hysteresis curves of reference of the position 3.

(d) Representing the hysteresis curves of reference of the position 4.

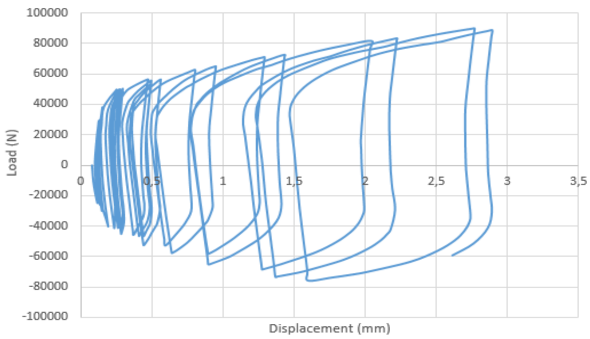
(e) Representing the hysteresis curves of reference of the position 5.

(f) Representing the hysteresis curves of reference of the position 6.

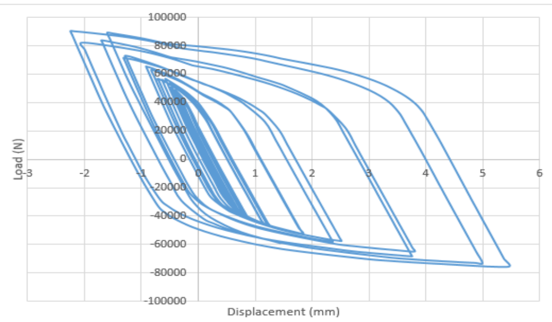
These series of hysteretic curves generated from each position provided the basic data for determining its overall behaviour of the structure. The prediction of this global behaviour can only be achieved through studying the load-deformation history of each element separately and then summing their effects.



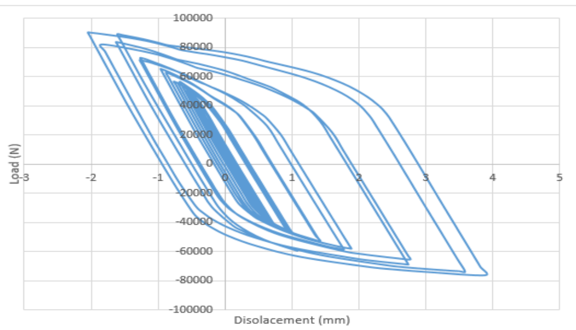
(a)



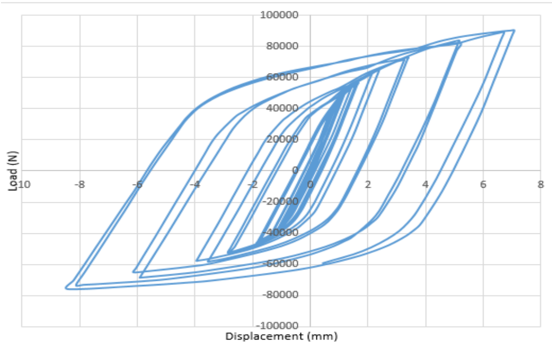
(b)



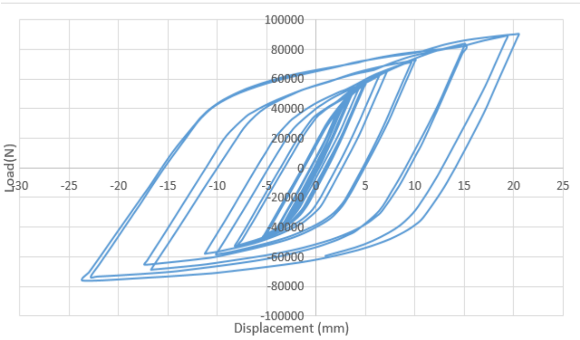
(c).



(d)



(e).



(f).

Figure 5.4 Results of Load-displacement Hysteresis loops at different dial positions1, 2, 3, 4, 5 (a), (b), (c), (d), (e) and (f) respectively

- **Results of Von-Mises contours**

  - **General remarks**

The von Mises yield criterion with kinematic hardening rule was adopted to model the steel material in the finite element analysis. The bilinear stress-strain relationship, was used to model the steel plate. Beam-column connections are regions that suffer from severe yielding, local buckling and tearing, etc as evidenced in damages from the past earthquakes. Classical plasticity theory cannot deal with such complex hysteretic behaviours under earthquake loadings. The key observations were local flange and web buckling, ductile flange tearing, significant local buckling in compression flange and brittle fracture of the flange.

Noting that modelling of the complex behaviour of connections such as buckling and tearing, etc is still challenging, an alternative method to model the complex nonlinear behaviour of connections is in needs. As it can be noted from Figure 5.5, the deterioration of the flexural strength of the connection was related to occurrence and spreading of local buckling in the beam flanges and web. A well-defined plastic hinge in the beam has formed column size the panel zone deformation has not been remarkable, and the plastic deformation took place mainly in the beam see Figure 5.5.

As the primary role of the stiffeners is to prevent local buckling prior to overall buckling and to increase overall buckling strength. In seismic applications, an additional, yet equally important role of stiffeners is to increase ductility of the cross section under cyclic loading. The cyclic behaviour of the top and seat with web angle connections was characterized by bolt slippage and yielding and spreading of plastic deformation in the top and bottom angles, cyclically subjected to tension. No significant rotation of the column and distortion of the panel zone have been observed throughout the experimental tests carried out on these specimens.

- Results presentation

In the following, results coming up from FEA model will be ^presented for each component of the structure in such manner that each component will have a particular interest. That is for each component, at the first stage, the stress contours of principal stresses followed by shear stresses will be first presented and discussed. Then, the end plate results with the influence of holes in the global behaviour and in similar manner the presentation of the obtained results in column part of the structures. A particular interest on the panel zone, as being the weakest part of the connection, with more figures depicting all aspects of its behaviour with the influence of placing transverse stiffeners. Then at last, but not the least, the behaviour of bolts will be presented and discussed.

- **The whole structure**

Generally speaking, Figures 5.5, 5.6 show the global configurations of the structures in the last stage of loading in terms of stress and displacement contours. Von Mises stress distribution is shown in the last step of loading for a stiffened connection in the following figures. Von Mises stress distribution is shown in the last step of loading for a stiffened connection.

As can be seen in Figure 5.5 and 5.6 a sample is based on beam plastic capacity and plastic hinge formation fails by yielding and local buckling of the compression flange of the beam.

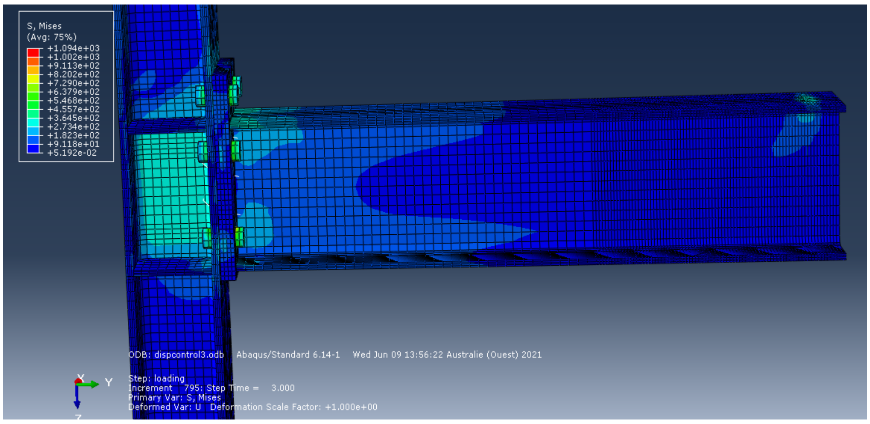


Figure 5.5 Global view of Von-Mises principal stresses contour at the last stage of cycle loading.

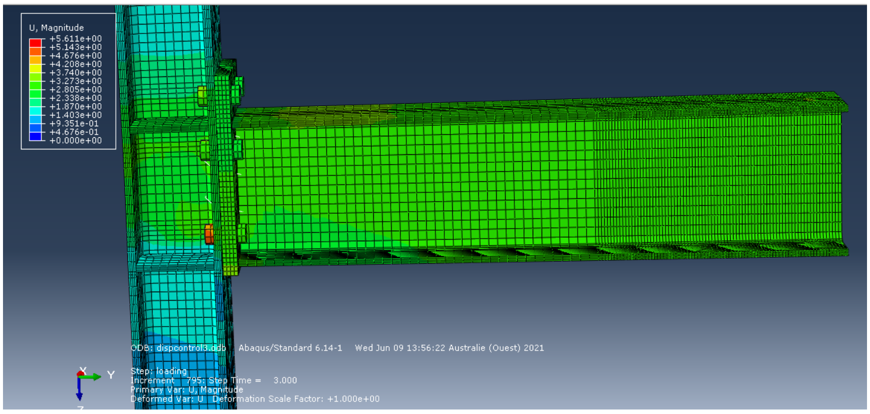


Figure 5.6 Global view of Displacement contour at last stage of cycle loading.

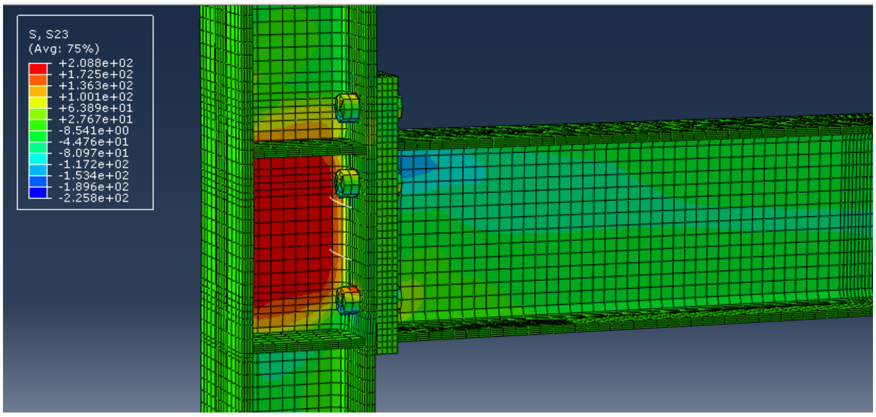


Figure 5.7 Global view of Von-Mises shear stresses contour at the last stage of cycle loading.

As can be seen, the end-plate connection response is associated with some pinching effect because of nonlinear contact between end-plate and column flange. The response is associated with low stiffness degradation effect too. However, accurate modelling of the connection behaviour is required for frame analysis, and mentioned effects should be considered in models whose experimental behaviours are not available.The Von Mises yield criterion is adopted to determine whether the steel reaches the yield point in the multi-axial stress state. A bilinear kinematic hardening model was applied to all component of the connection including the high-strength bolt constitutive model, which is usually applied for high.

As it can be seen from the above figures, no apparent collapse is noticeable in the components of the connection, even some of the constituting part are fully yielded particularly the panel zone. Each part of the beam to column results will be discussed separately.

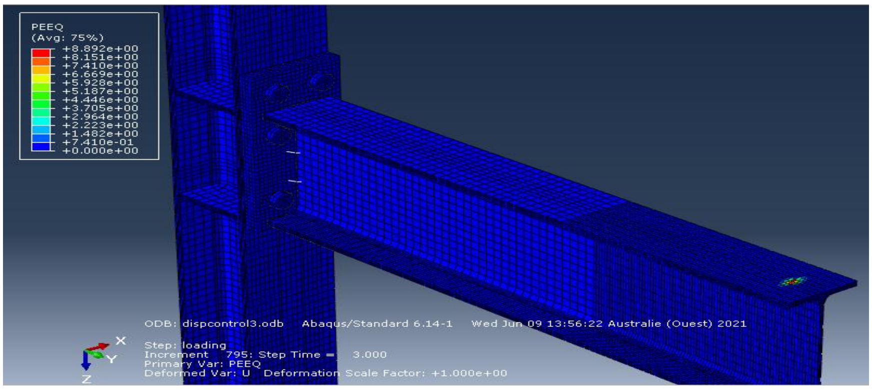


Figure 5.8 PEEQ results for the whole structure

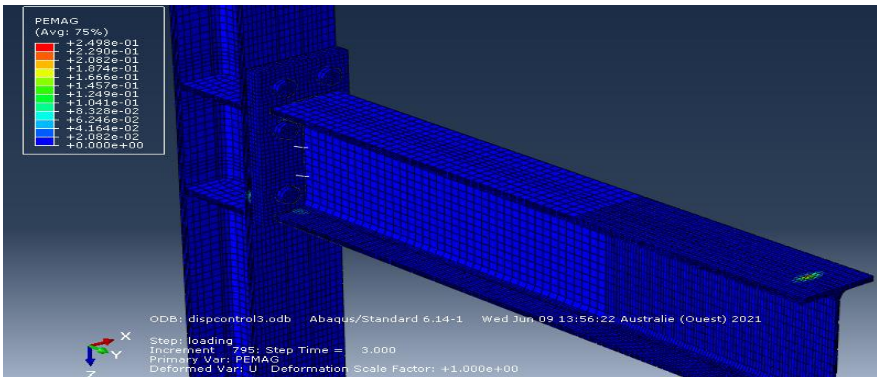


Figure 5.9 PEEMAG results for the whole of the structure .

In figure 5.8 the PEEQ refers to the equivalent plastic strain which describes the degree of work hardening in a material when removing the load. While PEMAG refers to the plastic strain magnitude of the accumulated plastic strain as depicted in the relative figure 5.9. These quantities will discussed later on.

- **Beam**

This part of the discussion presents the numerical study on the seismic performance of end-plate moment connection between I-beam to HEB column in terms of the principal and shear stresses contours along with the deformations. No significant rotation of the column and distortion of the panel zone have been observed throughout the loading history.

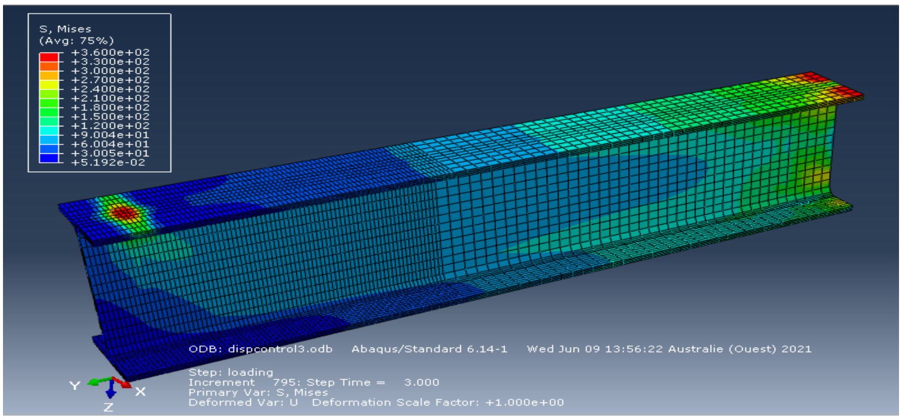


Figure 5.10 Von-Mises principal stresses contours in the beam.

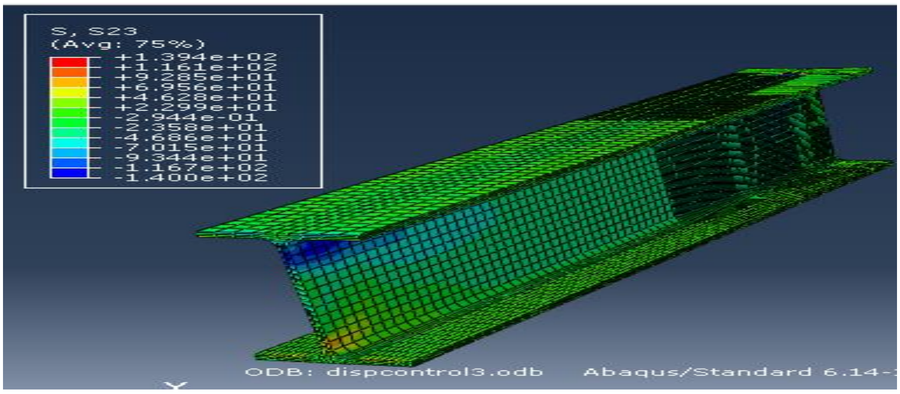


Figure 5.11 Von Mises shear-stress contour in beam.

As far as the deformed shape of the beam is concerned, the essential is plotted in figure 5.12.

As can be seen in Figure 5.10 that the upper flange of the cross-sectional is yielding in the location of the joint level which can be explained, and as expected, by the fact of a development of plastic hinge which complies with the design of the whole based on the principle of Weak Beam vs. Strong columns which will lead to a local buckling of the upper flange. A concentration stress yielding zone is also noticeable in the location of the applied load. The remaining areas of the beam are less stressed with increasing values as the section is nearer the connection. Unlike the web, flanges of beam in the area adjacent to the endplate have not yielded, which shows that most of the yielding is due to the moment transfer in this area. The ultimate loading stage shows, as depicted in Figure 5.11, that the areas in the vicinity of the connection are suffering from high level of shear stresses, exceeding the yield shear stress in many positions, as consequence of interaction between the beam and the remaining parts of the connection. This will also contribute to failure of the weakest part at this location, i.e. the beam. Parts of beam in the other side seem to not have a big influence of shear stresses.

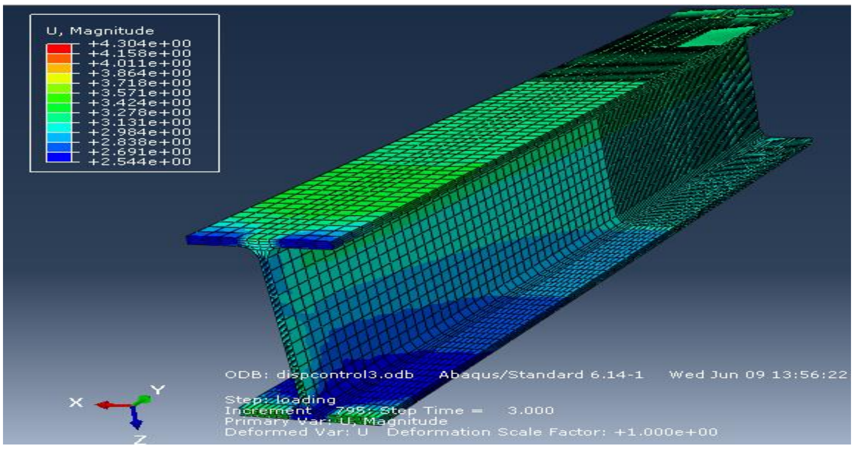


Figure 5.12 Displacement contour in beam.

Following the same pattern showed in Figure 5.12, figure 5.11 shows the contribution of the shear forces to increase the deflection of beam.

- **End plate**

The cyclic behaviour of endplate connection was investigated numerically using the finite element method of the analysis in this paper. It has been showed in several experimental works that in the case of extended-end-plate connections, flexural deformations of the end plate and axial deformations of the bolts contribute to the energy dissipation under cyclic loading. If the end plate is stiffened, it can ensure yielding of the beam and lead to very good energy dissipation capacity.

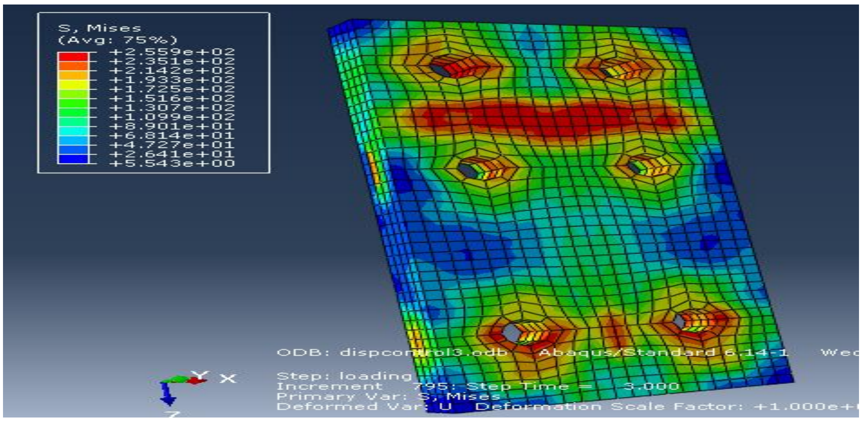


Figure 5.13 Von-Mises principal stresses contour distribution in plate.

Von Mises stress distribution around the connection region and the bolt is shown in Figures 5.13 and 5.14 for the last step of loading.

Figures 5.13 and 5.14 show the distribution of principal and shear in the surface of the end plate respectively. As can be seen from Figure 5.13 plasticity zones are spread over the end-plate surfaces, especially in the middle zone along with a net yielding of the bolt's holes. This is due to the high value of bending moment action in this region. As consequence of the relatively small shear force value, the consequent shear stresses are less important compared to the principle ones as can be easily noted from Figure 5.14. Clearly, increasing the thickness of the end plate will decrease this stress concentration.

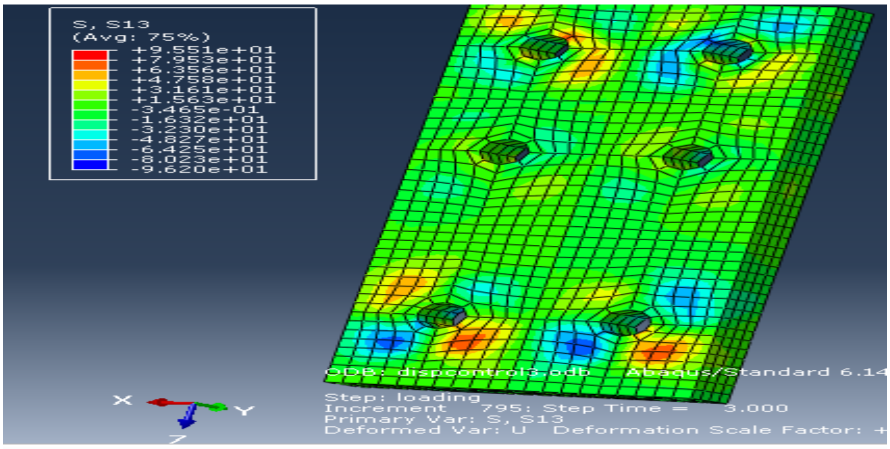


Figure 5.14 Von-Mises shear stresses contour distribution in plate.

The displayed of the PEEQ and PEMAG values in figures 5.15 and 5.16 respectively are indicating the level of ductility of the end-plate steel, theoretically ductile and isotope until the ultimate deformation.

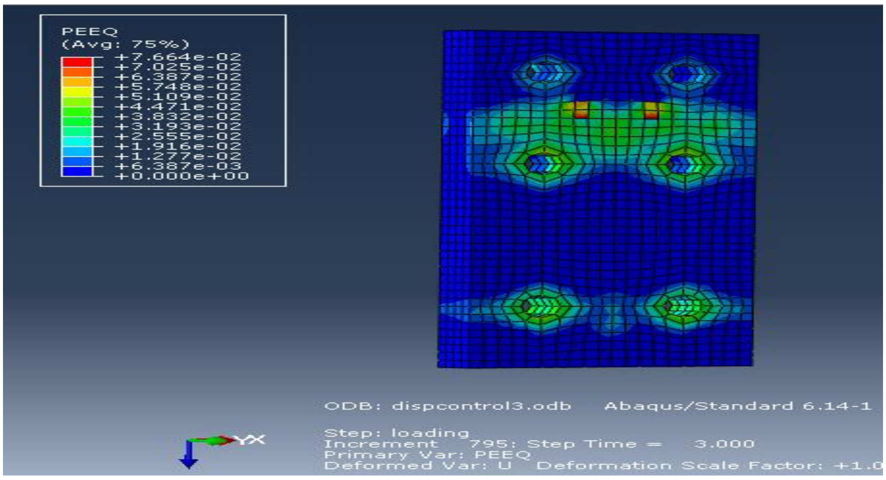


Figure 5.15 PEEQ results in plate

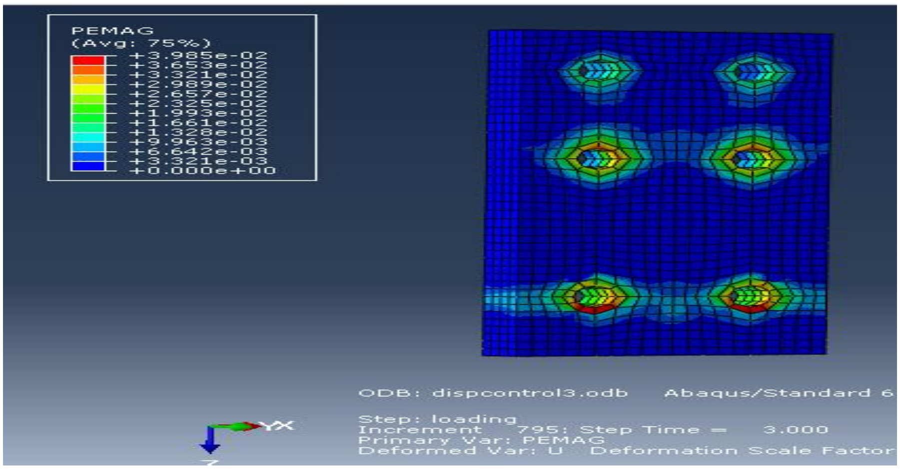


Figure 5.16 PEEMAG results in plate.

- **Column**

As the column represents the most important of the structure according the design criteria. The numerical model consists of a cantilever beam that was connected to a column by an endplate connection. The outcomes of numerical analysis are shown in Figures 5.17 to 5.21.

As can be clearly seen from Figure 5.17, the column is suffering from high stresses field leading to a total yielding in the central zone between the stiffeners. Also, a stress concertation shear yielded zone can be detected in the junction between the end plate and the column flange. Moreover, the stress in column flange in area adjacent to the stiffener is reduced; this shows that the plastic hinge location is moved toward the column tip.

Panel zone can be the weakest element in the beam to column joint when the frame is submitted to lateral loads. The FEA results show the possibility that the panel zone would yield in shear before the beams and columns reach their flexural capacities and demonstrate the importance of the panel zone on the behaviour of the beam to column joints, with important ductility supplies and hysteretic energy dissipations. show that panel zone failure governed the ultimate capacity. Three different load-carrying mechanisms including shear bucking of the joint panel, the buckling of column, and both shear bucking of the joint panel and buckling of column are identified.

When the joints are subjected to reversal of loading, the joint panel zone may deform in shear and the panel zone shear resistance would affect the joint's moment capacity. show that panel zone failure governed the ultimate capacity.

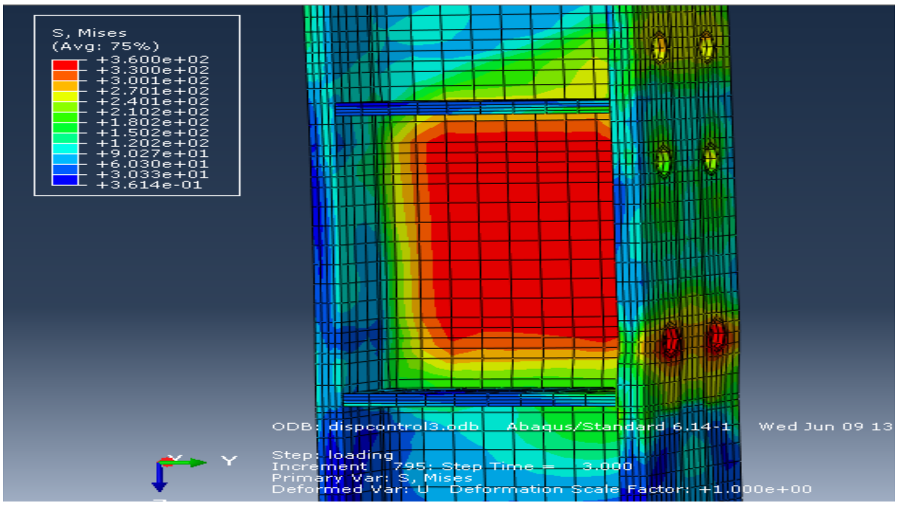


Figure 5.17 Von-Mises principal stresses contour in column

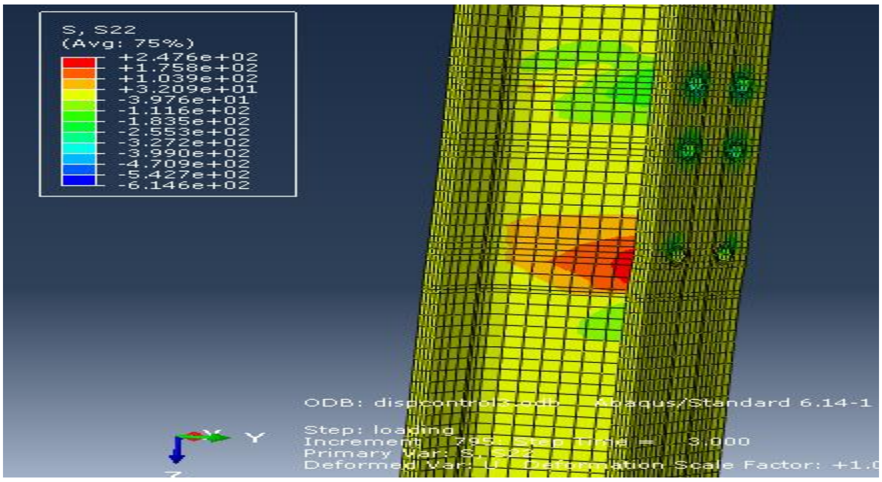


Figure 5.18 Shear stresses contours in the column flange

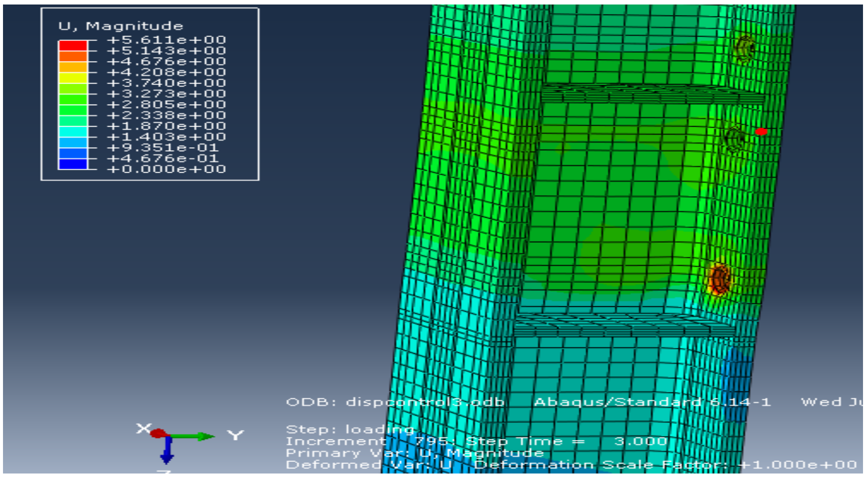


Figure 5.19 Displacement contour in column.

As can be seen from Figure 5.19, a local buckling of the column flange due to the fact that the finite element model reaches the buckling moment, numerical instability. In the other words, the model becomes unstable because of large plastic deformations in the panel zone and beam flanges but the test specimen is failed due to the lateral-torsional and local buckling of members, specially beam. The panel zone of connection has shown a major plastic behaviour, which is desirable circumstance.

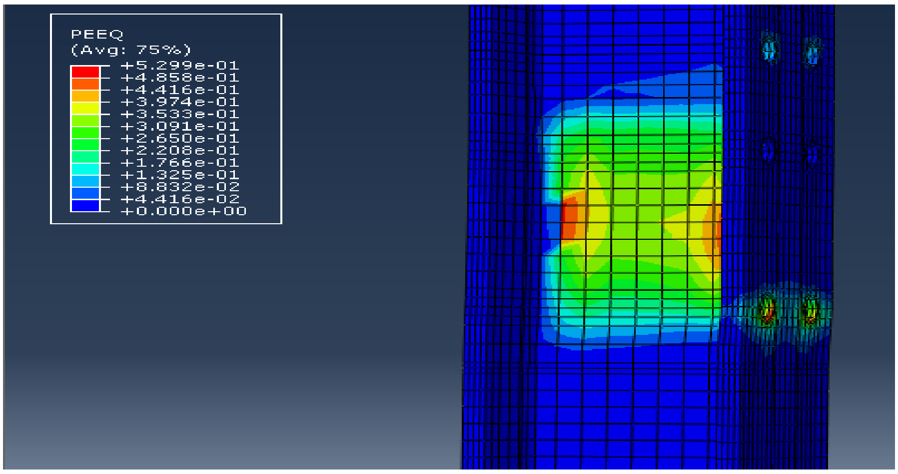


Figure 5.20 PEEQ results in Column

PEEQ and PEEMAG are reporting in figure 5.20 and 5.21 respectively. As it can be seen, a concentration of PEEQ value is located in the panel zone and the lower part of the contact surface due to high level of stresses as a result of cyclic loading.

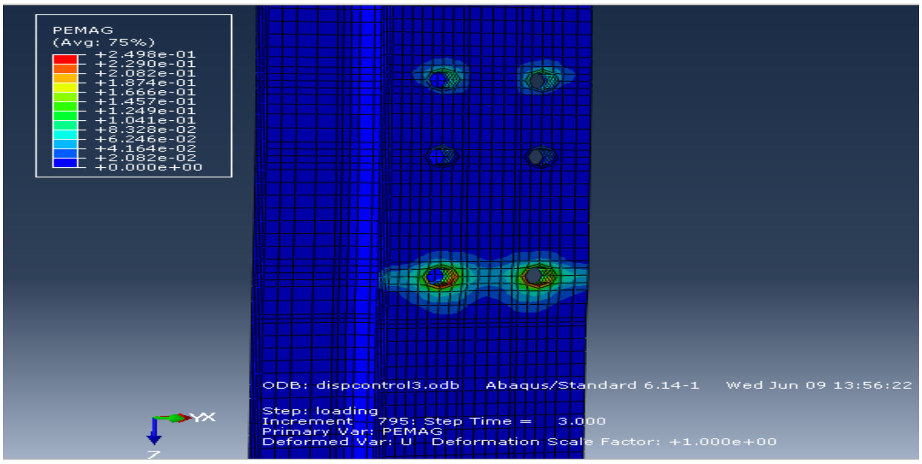


Figure 5.21 PEEMAG results in column.

- **Bolts**

The general behaviour of bolts is depicted in the following figures. In this particular analysis no failure Bolts have no rupture.

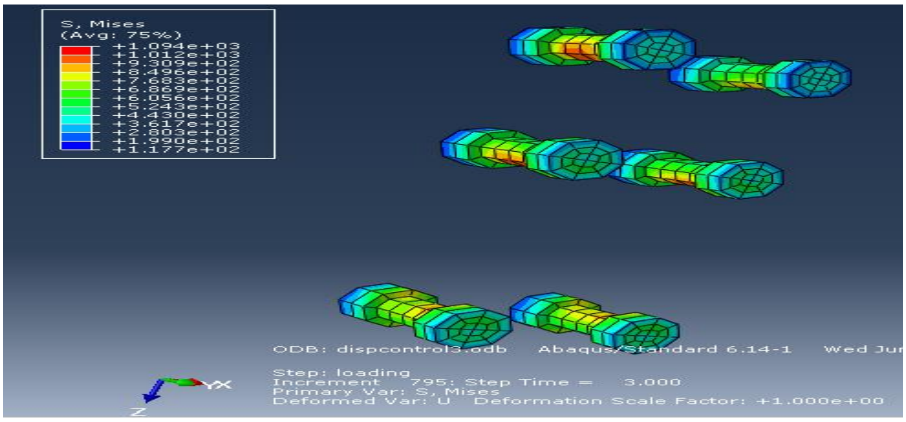


Figure 5.22 Von-Mises principal stresses in bolts

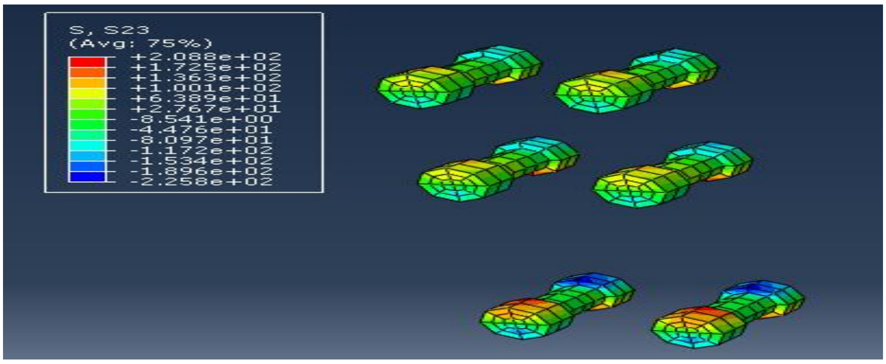


Figure 5.23 Von-Mises under shear stresses in bolts

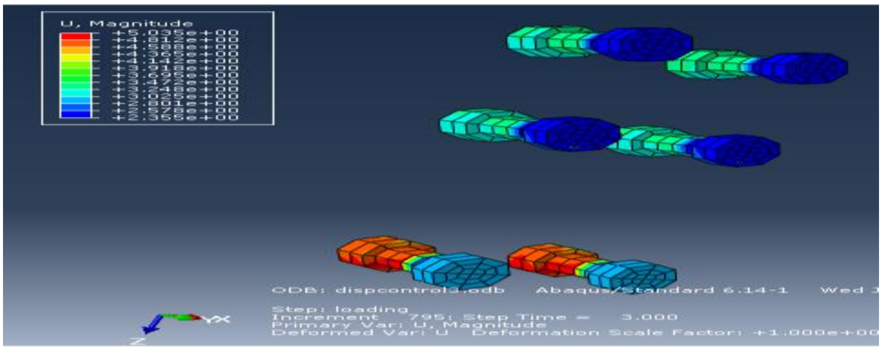


Figure 5.24 Displacement contour in bolts.

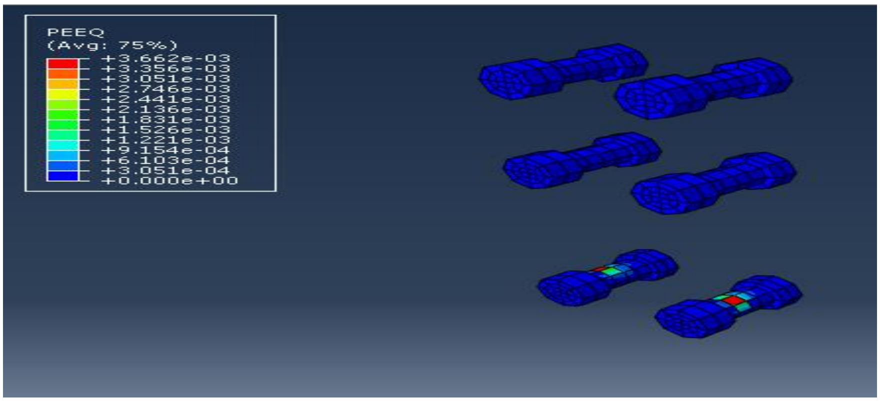


Figure 5.25 PEEG results in bolts

Because the endplate is bolted to the column flange, it shows a complex bending behaviour. As long as the plate is under compression is concerned, it stays in contact with the column flange and force is transferred between these parts. Bolts also have the same condition with their surroundings. Bolts are more brittle and could bear a lower strain before failureTypical connection failure is categorized by bolt failure, angle/plate rupture, weld failure, plate bearing failure, and the limit state defined by excessive deformation (rotation) of the connection.

The cyclic response of connections in terms of shape of the hysteresis loops, stiffness and strength degradation and resulting dissipation capacity are directly related to the components involved in plastic range, mainly the weakest component. It has been accepted that the panel zone should not be destroyed before the collapse of beam-column connection, and even if shear buckling occurs, the load carrying capacity does not decrease immediately that significantly influence on the response of the cyclic behaviour of joint panel zones in beam-to-column.

- **Effect of stiffeners**

The primary role of the stiffeners is to prevent local buckling prior to overall buckling and to increase overall buckling strength. In seismic applications, an additional, yet equally important role of stiffeners is to increase ductility of the cross section under cyclic loading. This stiffener includes two steel plates, which are welded on column flanges. The overall behaviour of connection has been improved and stiffener has some effect on delaying the yielding in connection.

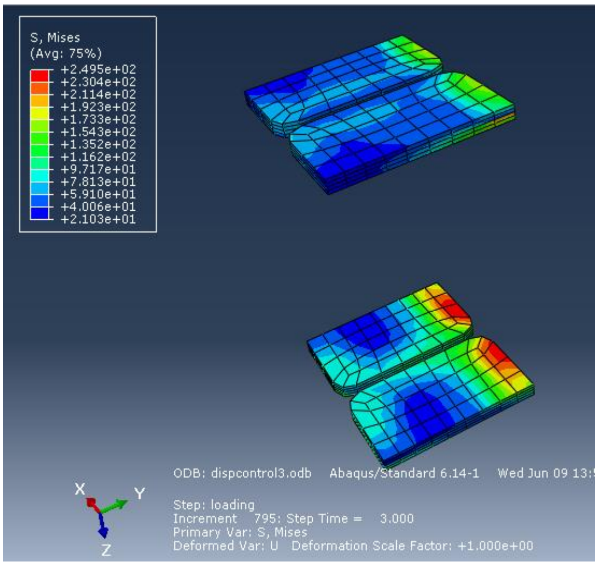


Figure 5.26 Von-Mises contour in Stiffeners.

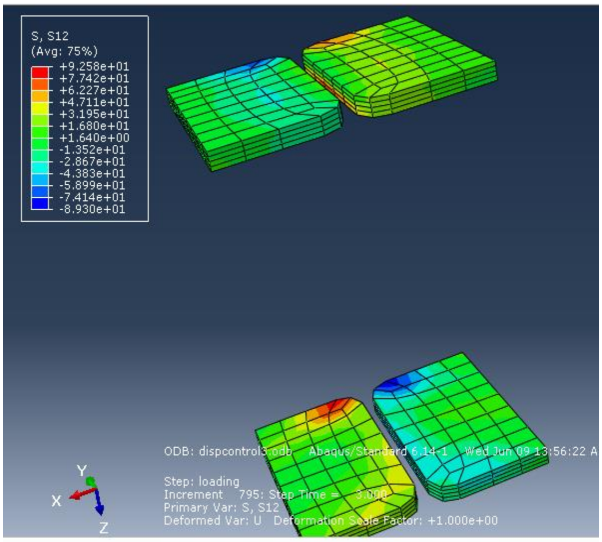


Figure 5.27 Shear-Stress in stiffeners .

5.4.5 Comparison between monotonic and cyclic loading results

It is of most important to find a relation between the behaviours of a beam-to-column joint when loaded either monotonically or cyclically. Figure 5.28 shows both monotonic and cyclic curves for the modelled joint. The maximum cyclic rotation which represents in reality in a MRFs structure the drift-storey of the joint is much less than the monotonic in (c),(d),(e) and (f) one. The cyclic behaviour is considered as nonductile. On the other hand, the second the drift-storey is bigger in (a) and (b) which exhibits very good ductile behaviour (large amount of energy dissipated) during loading.

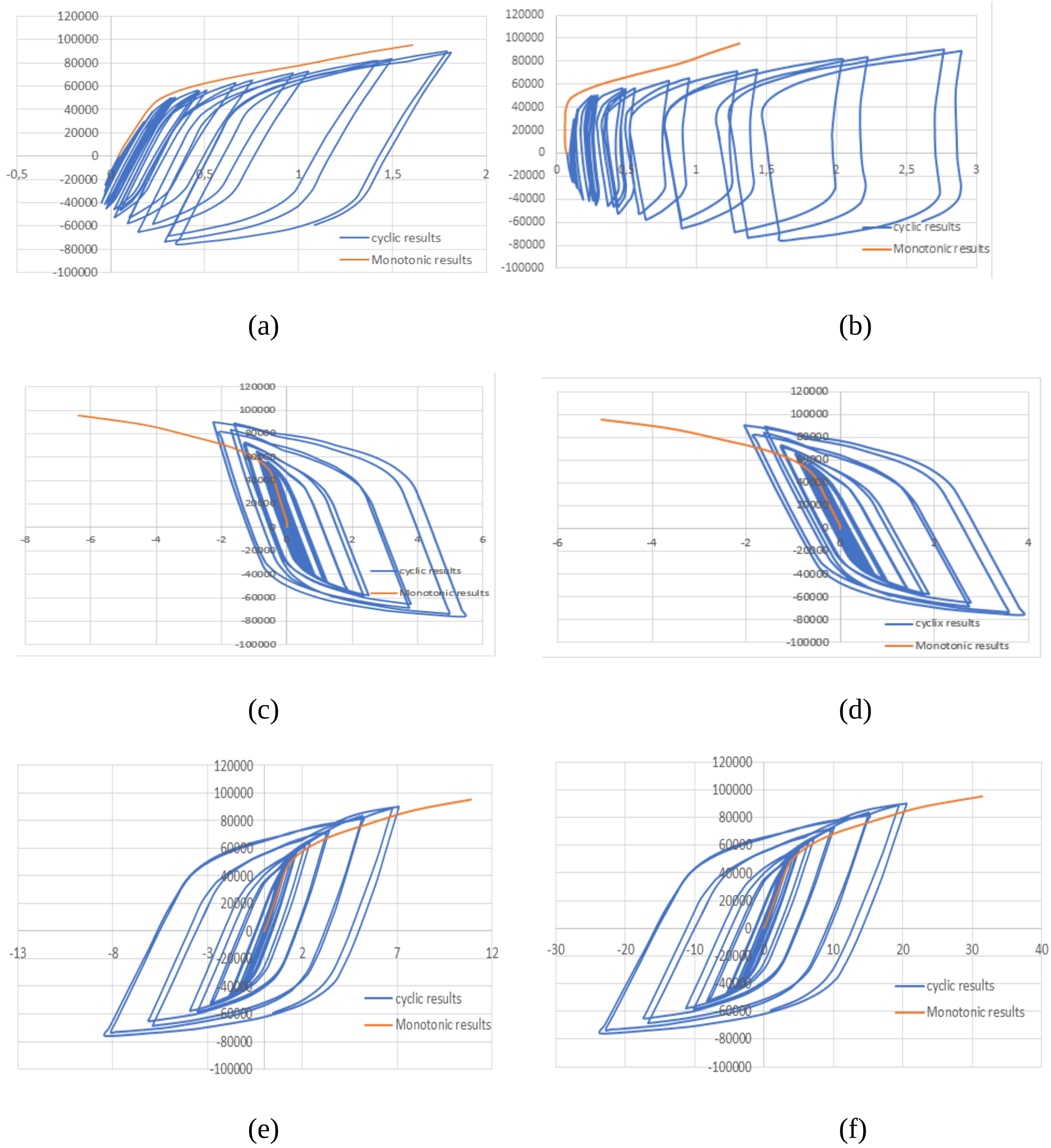


Figure 5.28 Comparison of monotonic and hysteretic curves for positions 1,2, 3,4,5 and 6 respectively.

In summary, the results showed that the finite element models that were built could effectively simulate the load bearing behaviour of bolted connections under monotonic and cyclic loadings. The hysteresis curves of the joint model are all lower than that of the monotonic loading curves. This is because under cyclic loading and due to the Bauschinger effect, the plastic deformation of the joints continuously accumulates, and the local buckling phenomenon becomes more obvious, resulting in the degradation of the strength and rigidity of the structure. However, in some positions. the hysteresis curves were roughly the same as the under monotonic load-displacement curve.

5.5 GENERAL CONCLUSIONS

The analysis of the finite element outcomes needs more attention and specially more time to investigated details Once again, because of time lack, only few general conclusions may be drawn. The cyclic behaviour of endplate connection was investigated numerically using the finite element method of the analysis in this chapter. The followings can be concluded from the upper described analysis:

- The panel zone of connection has shown a major plastic behaviour, which is desirable circumstance
- Bolts have no rupture
- On top and bottom of endplate in the extended region, there is little stress in both ends, which shows a little force is transferred in this region.
- Unlike the web, flanges of beam in the area adjacent to the endplate have not yielded, which shows that most of the yielding is due to the moment transfer in this area.
- The prying action has begun and it may cause the bolts to rupture. This phenomenon should be studied more thoroughly .

CHAPTER 6

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FURTHER FUTURE WORK

6.1 GENERAL

On a broad level, the present work undertaken in this Master's dissertation asserted that the initial objectives of this study with a substantial effort has been made to understand the theorical and the code prescriptions of beam to column connections, have been globally attained. Typically, the behaviour of semi-rigid connections relates to the performance on sub-assemblage frame of beam-to-column connection.

Due to the failure of the "pre-Northridge" connection, end-plate connections are becoming much more attractive for seismic design. The reliable reason of using extended end plate in steel frames specially in seismic zones that it has a big energy of dissipation while increasing load and has a ductility strength and stiffness needed in joints subjected to seismic and cyclic loads. The extended end-plate connections provide excellent performance in resisting seismic loads in high-risk areas.

6.2 CONCLUSIONS

From the work described in this dissertation, some main conclusions may be drawn to summarise the findings of the undertaken analyses of beam to column semi-rigid connections through 3D models implanted in ABAQUS software. The FE model can provide a variety of results at any location within the model with a viewing of the full fields of stresses and strains are possible in the FE model. This provides a great advantage in monitoring each components of the connection. The analysis of the finite element outcomes needs more attention and especially more time to investigated details Once again, because of time lack, only few general conclusions may be drawn.

1- The detailed literature review showed that even though the finite element modelling can predict well the behaviour of a semi-rigid connection, the reliable test results are essential to calibrate the model before it can be used for a different connection. In order to study the mechanical behaviour of bolted beam-column connections, the accuracy and applicability and the predictability of the finite element model were firstly validated by comparison to the published experiments in literature on end-plate connections using ABAQUS. The proposed FE model outcomes show good agreement with the test results. One at different stages of monotonic loading.

2- After determining that the models could effectively predict the response characteristics (e.g., bolt forces, plate separation, and plastic rotation) of the moment endplate connections, a parametric analysis under monotonic loading was performed. The FE results showed that steel grade parameter has a significant effect on the behaviour of beam-column especially in plastic regime along with the addition of web stiffeners in column delay the local buckling, while changing bolts grade seems to have no significant effects. Due to the failure of the “pre-Northridge” end plate connections are becoming much more attractive for seismic design.

3- The results showed that the finite element models that were built could effectively simulate the load bearing behaviour of bolted connections under monotonic and cyclic loadings. The hysteresis curves of the joint model are all lower than that of the monotonic loading curves. This is because under cyclic loading and due to the Bauschinger effect, the plastic deformation of the joints continuously accumulates, and the local buckling phenomenon becomes more obvious, resulting in the degradation of the strength and rigidity of the structure. In some positions, the hysteresis curves were roughly the same as the under monotonic load-displacement curve.

4- Panel zone can be the weakest element in the beam to column joint when the frame is subjected to lateral loads. The FEA results show that the panel zone would yield in shear before the beams and columns reach their flexural capacities and demonstrate the importance of the panel zone on the behaviour of the beam to column joints, with important ductility supplies and hysteretic energy dissipations shows that panel zone failure governed the ultimate capacity.

5- It has been showed in several through the FE results that in the case of extended-end-plate connections, flexural deformations of the end plate and axial deformations of the bolts contribute to the energy dissipation under cyclic loading.

6- The panel zone of connection has shown a major plastic behaviour, which is desirable circumstance

7- . Bolts are more brittle and could bear a lower strain before failure. However, bolts have shown no rupture

8- On top and bottom of endplate in the extended region, there is little stress in both ends, which indicates that a little force is transferred in this region.

9- Unlike the web, flanges of beam in the area adjacent to the endplate have not yielded, which shows that most of the yielding is due to the moment transfer in this area.

10 - The prying action has begun and it may cause the fracture of the bolts.

6.3 SUGGESTIONS FOR FUTURE RESEARCH

Several topics should still be considered in future research.

1- Parametric investigations of the influence under cyclic loadings of some of parameters such as the plate dimensions and thickness, beam shape forms and length, the bolts diameter, the orientation of stiffeners in the panel zone and steel grades. Three- dimensional FE analysis may be helpful for investigating these specific topics.

2- The same connection model can be used to examine the response characteristics of numerous end-plate configurations.

3- Use of different loading protocol to access by mean of comparison the behaviour of the semi-rigid connection.

4- Further research is required into other configurations of end-plated connections, such as these utilising eight bolts per flange, hunched beams, and other forms of column flange stiffening.

5- The assessment of the effect of the level of ductility under cyclic loading of the beam to column semi-rigid connection seems to be an interesting topic

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