FCTUC DEPARTAMENTO DE ENGENHARIA CIVIL
FACULDADE DE CIÊNCIAS E TECNOLOGIA
UNIVERSIDADE DE COIMBRA

Advanced modelling of joints between hollow and open sections

Thesis submitted for the degree of Master in Civil Engineering in the specialization of Structural Mechanics

Author

Pedro Filipe Gonçalves Martins

Supervisor

Rui António Duarte Simões

František Wald

Coimbra, October, 2015

"Sobe o primeiro degrau com fé. Não é necessário que vejas toda a escada,
apenas que dês o primeiro passo."

Martin Luther King

Aos meus pais.

AGRADECIMENTOS

Ao meu orientador Professor Doutor Rui António Duarte Simões, por ter aceite ser meu orientador, por me ter ajudado a estabelecer contacto com a Universidade de Praga, pela sua disponibilidade, paciência, conselhos e boa disposição com que sempre me recebeu e ajudou.

Ao meu coorientador Professor Doutor František Wald por me ter recebido de forma acolhedora na Universidade de Praga, pela informação experimental disponibilizada, pela sua simpatia e auxílio prestado.

À minha mãe por ter sido o meu anjo da guarda ao longo deste percurso, pelos conselhos e pela força que sempre me deu para superar as dificuldades. Ao meu pai, pelo seu olhar que representa o quanto me quer bem e pelas conversas que me ajudaram a crescer como Homem.
Agradeço-vos pela educação que me deram e por toda a confiança que sempre depositaram em mim. Dedico-vos este curso!

Á minha irmã Sandra e ao meu cunhado António Pedro, por terem estado sempre presentes e terem sido os amigos que eu muitas vezes precisei. Nunca vou esquecer o que fizeram por mim!
Aos meus sobrinhos Afonso e Mafalda, pelo enorme sorriso que têm sempre para me oferecer e por me fazerem sorrir!

À Mariana, pelo seu amor, cumplicidade e amizade. Teres estado sempre ao meu lado neste percurso é algo que nunca esquecerei!

Aos meus amigos e colegas de curso, Rafael, Filipe, Diogo Aguiar, Mário, Mauro, João Neves, Herman, Roberto e João Costa. Foram a minha família em Coimbra, sem vocês teria sido tudo mais difícil. Vou guardar as nossas vivências, mas acima de tudo quero guardar a nossa amizade!

Ao Departamento de Engenharia Civil, à Universidade de Coimbra e a Coimbra.
Devo muito a esta Instituição e a esta cidade por aquilo que aprendi!

Muito Obrigado!

ACKNOWLEDGEMENTS

I'm grateful to my supervisor Professor Rui António Duarte Simões, for agreeing to be my supervisor, for helping me to liaise with the University of Prague, for his availability, patience, advices and goodwill with which he always received me and helped me.

I'm grateful to my co supervisor Professor František Wald, for the welcoming receive at University of Prague, for the provided experimental data, for his kindness and help given.

I'm grateful to my mother, for being my better angel lifelong, for the advices and strength she always gave me to get over the difficulties. To my father, for his kind gaze which shows how much he cares about me and for the conversations that helped me to grow as a Man.
Thank you both for the education you gave me and for all the trust you put on me. I want to dedicate this course to you!

I'm grateful to my sister Sandra and to my brother-in-law António Pedro, for being always present and for being the friend whose I needed a lot of times. I'll never forget what you've done for me!
To my nephews Afonso and Mafalda, for the huge grin they always gave me and for making me smile!

I'm grateful to Mariana, for his love, complicity and friendship. Being always by my side in this journey is something I'll never forget!

To my course colleagues and friends, Rafael, Filipe, Diogo Aguiar, Mário, Mauro, João Neves, Herman, Roberto e João Costa. You were my family in Coimbra, without you I'm sure this course would have been harder. I'll keep the good memories, but above all, I want to keep our fellowship!

I'm grateful to the Department of Civil Engineering, to University of Coimbra and to Coimbra. I owe a lot to this Institution and to this city for what I learned!

Thank you very much!

ABSTRACT

This dissertation intends to increase the knowledge on the structural behaviour of welded joints between hollow and open sections under static loading.

On the most common design rules for steel joints, such as EC 3-1-8, there are two kind of design philosophies. One for joints with open sections and another for joints with hollow sections. As far as the open sections are concerned, it's possible to apply the component method which allows the designer to consider the joint as a set of individual components and acquire a large knowledge on the strength, stiffness and deformation capacity of each component. Thus, performing its later assembly, it's possible to obtain the behaviour of the joint as a whole. However, for hollow sections it's only possible to access the knowledge on the resistant capacity of the joint based on empirical formulas.

Considering this limitations, arises the interest to obtain more information on the performance of joints with hollow sections respecting to its resistance, stiffness and deformation capacity. In this study, an analytical, experimental and numerical analysis on the behavior of welded T joints between hollow and open sections was made, with the open section acting as the chord and the hollow section acting as the brace. These joints were studied under compression or bending.

Soon after, the referred joints were compared with T-joints between two open sections. For these joints it's possible to achieve their behaviour according to the component method, so, the aim of this comparison was to verify if it's appropriate to apply the component method for similar T-joints with hollow sections.

Thereby, bringing together all the information achieved, it's intended to define the basis for a simpler and intuitive method for the design of steel joints independently of the structural elements used. In this particular case, the study is carried out in such a way that the hollow sections could be used in civil engineering with more reliability, accuracy and frequency.

INDEX

INDEX OF FIGURES

INDEX OF TABLES

SYMBOLS

Lowercases

i	integer used as an index to characterize the element of a joint. i = 0 symbolizes the chord and i = 1,2 or 3 the braces. Concerning the joints with two braces, i= 1 symbolizes the compressed brace and i = 2 the tensioned brace.
j	integer subscript used to designate the overlapped brace in overlap joints
b_0	external width of the chord
b_i	external width of brace i (i = 1 or 2)
d_i	external diameter of brace i (i = 1 or 2)
h_0	external height of the chord
h_i	external depth of brace i (i = 1 or 2)
h_z	distance between the centres of gravity of the effective parts of the RHS brace
t_0	flange thickness of an I or H section chord
t_i	wall thickness of CHS or RHS brace i (i = 1 ou 2)
t_w	web thickness of an I or H chord
t_f	flange thickness of an I or H chord
θ_i	angle between the brace i and the chord (i = 1 or 2)
g	gap between the braces
r	inside corner radius between the web and flanges of an I or H section
σ_{yo}	yield stress of the chord
σ_{yi}	yield stress of the brace i (i = 1 or 2)
σ_u	ultimate stress
ε	strain
ν	Poisson's ratio
b_{ei}	effective width of an RHS brace
d_{ei}	effective width of a CHS brace
b_w	effective width for the web of an I or H section
d_w	depth of the web of an I or H section
α	factor used in the equation of A_s
$b_{e,ov}$	effective width of an overlapping RHS brace at the connection to the overlapped brace

$d_{e,ov}$ effective width of an overlapping CHS brace at the connection to the overlapped brace

$l_{b,eff}$ effective perimeter for local yielding of the overlapping brace

γ_{M0} partial safety factor for the resistance of cross sections of any class

γ_{M1} partial safety factor for the resistance of the elements to buckling

μ Rotation stiffness of a joint. μ = 1 for initial rotation stiffness.

Uppercases

A_s shear area of a chord member

A_0 cross-sectional area of the chord member

A_v shear resistant area of an element

E Young modulus or modulus of elasticity

O_v overlap ratio, expressed as a percentage

$O_{v,lim}$ overlap limit for brace shear check

N_0 applied axial force in chord

N_i applied axial force in brace i (i = 1 or 2)

$N_{pl,0}$ axial yield capacity of the chord

N_u axial ultimate capacity

M_0 applied moment in chord

$M_{pl,0}$ plastic moment capacity of the chord

M_u ultimate moment capacity

$M_{ip,1,Rd}$ design value of the in-plane moment in brace i (i = 1 or 2)

V_{Ed} design value of the shear force in a chord member at the gap location

$V_{pl,Rd}$ design value of the shear force in a chord member

V_s^* design resistance of the joints, expressed in terms of the axial force in member i (i = 1,2)

L_0 length of the chord in test set up

L_1 length of the brace in test set up

The symbols not described in this section are duly described in its respective part of the dissertation.

ABREVIATIONS

AISC	American Institute of Steel Construction
CEN	European Committee for Standardization
CIDECT	International Committee for the Development and Study of Tubular Structures
CHS	Circular Hollow Section
CISC	Canadian Institute of Steel Construction
EC	Eurocode
EN	European Norm
H	Section with H shape
HEA	Section with H shape
I	Section with I shape
IPE	Section with I shape
IIW	International Institute of Welding
ISO/FDIS	International Standard Organization/ Final Draft International Standard
LNEC	National Laboratory of Civil Engineering
RHS	Rectangular Hollow Section
SHS	Square Hollow Section

1. INTRODUCTION

1.1. Steel structures with hollow sections

Over the last decades, the use of steel structures has been a common option among the construction market. Steel is capable of offering new possibilities, tied to its capacity of prefabrication and assembly works in complex structures. The simplicity and speed of production process are also relevant allies to the strong adoption of these type of structures, which, associated to the physical and mechanical properties of steel, allows to minimize the self-weight of structures.

Moreover, it is more and more required to make choices which grant sustainability of structures, where steel plays an important role due to its natural characteristics and recycling potential. There’s also the need to give structures an aesthetical meaning without losing their structural quality, a fact that puts civil engineers aware to the necessity of increasing their skills, mainly about new technologies and critical sense.

The structural conception in aesthetic and functional meaning is the key for a reliable construction, not only because of architectural and technological competitiveness, but also because of the pursuit for economic and innovative solutions. Thereby, hollow steel sections have been highlighted relating to other type of sections in steel construction.

However, the use of hollow sections is not yet a frequent option, mostly due to its complex design and detailing with other structural elements, leading to a lack of the agility processes which raise project and construction costs. Thus, the particular behavior of joints with hollow sections and the lack of design methods on the main design norms, lead to a certain discrimination by joint designers in using these kind of sections.

Although, due to its properties, the demand for hollow sections is more and more a requirement. So, the need to investigate more about the technology and typology of joints with these sections has increased over the last decades.

The hollow sections can be applied in most of structural types depending on the purpose for which they are intended. Usually, this type of sections are chosen when there is a wish for a visible structure where the structural elements have the role of making the structure aesthetically appealing or, in other cases, due to its geometrical characteristics which provide an higher resistance to the structure according to the type of forces applied.

According to Wardenier *et al* (2010), the use of hollow sections provides some benefits over the open sections, such as:

- The ability to withstand high compression and tension forces.
- The center of gravity and the shear center are coincident to each other, thus guaranteeing symmetry in any direction that passes through this center. These characteristic provides a torsion stiffness considerably higher comparing to equivalent open sections.
- Comparing to open sections, hollow sections enable a self-weight structures optimization, because they are not elements subjected to buckling by nature, a fact that results in material savings.
- The possibility of filling the inside with concrete, conferring high resistance, structural stability and good fire resistance.
- The possibility to incorporate technical installations inside them.
- The drag coefficients are smaller when these sections are exposed to wind forces or water forces (Figure 1.1), ensuring better protection conditions to corrosion.
- The small surface area comparing to equivalent open sections generate a small surface to be protected by painting (Figure 1.2).

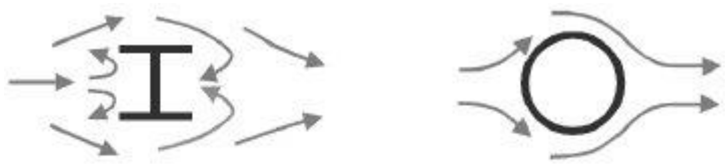


Figure 1.1 – Wind or water action on open and hollow sections
(Wardenier *et al*, 2010)

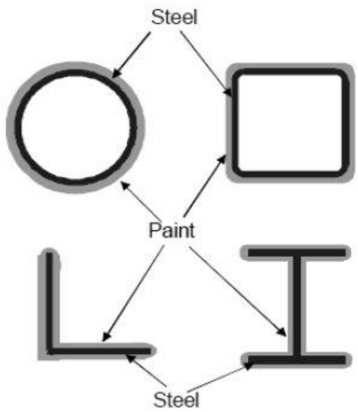


Figure 1.2 – Surface to be painted in hollow and open sections (Wardenier *et al*, 2010)

Therefore, according to Freitas (2013), hollow sections have some drawbacks comparing to open sections, such as:

- The manufacturing costs are slightly higher, approximately 10% more.
- Owing to its geometry, there are some challenges on the design of joints with hollow sections with the need to ally aesthetic to a reliable structural behavior, because it’s not possible to access the inner part of the sections.
- When hollow sections act as beams subjected to bending, there’s a large quantity of material that does not contribute meaningfully to the resistance of the element, representing a considerable material waste. However, this drawback can quickly become a benefit in cases where buckling appears.

1.2. Some remarkable structures

In Figure 1.3, is presented an example of structures where the use of hollow sections is frequent, mainly because of its resistance against dynamic forces caused by fluids which stimulate buckling of the elements where they are acting upon. This advantage exists due to the circular shape of hollow sections, where the drag coefficient is smaller enough to allow a good protection against corrosion and to make the structure lighter.

Figure 1.3 - Offshore structure
(Wardenier *et al*, 2010)

The evolution of the iron melting process in a large scale, prompted by the work of Abraham Darby III, a well-known English metallurgist, had an important role in the beginning of the Industrial Revolution that ended up with the construction of the first iron bridge in 1779, called "Iron Bridge". This bridge is presented in Figure 1.4.

Figure 1.4 - Iron Bridge (English Heritage, 2015)

In Figure 1.5, are shown some other examples of remarkable steel structures with hollow sections which are described in the paragraphs bellow.

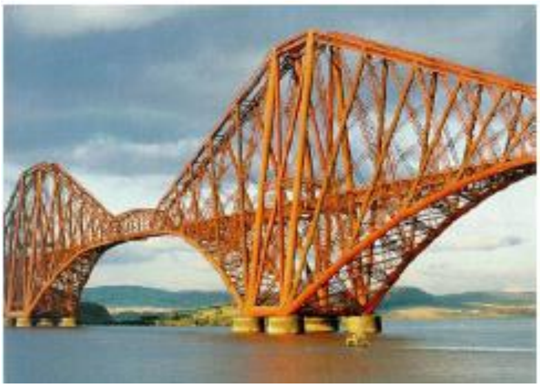
A century later of the “Iron Bridge” construction, was designed one of the most known and old steel bridges, the “Firth of Forth Bridge” in Scotland, concluded in 1890. The hollow elements with bigger dimensions were made using laminate riveted plates because at that time there was no other production methods for such big sizes. Furthermore, in the end of the 20^{th} century, began the development of different production methods related to bolted and welded elements for this kind of sections (Wardenier *et al*, 2010).

Another steel structure that also deserves highlighting is the Hall of Departures of Stuttgart Airport, opened to public in the year of 1936. The shown solutions with hollow sections are very attractive to people who is walking nearby, looking similar to branches of trees.

In Figure 1.5, can be seen the “Estádio da Luz”, construction concluded 2003 in Lisbon. This stadium has 3 000 tons of steel structure and 42 000 m^2 of metallic cover. According to the company which designed the stadium, at the time of its construction was used an innovative welding system, using pioneering technologies in Portugal. Also concerning “Estádio da Luz”, there’s a picture presenting different types of joints, of which between hollow sections or between hollow and open sections (Martifer Group, 2015).

In the city of Rio de Janeiro, more exactly in the metro station “Cidade Nova”, was built a railway bridge in 2010. Before opening to the traffic, this bridge turned out to be a concern due to the impact the structure would create on the landscape, so, the best solution fell into the use of an hanging railway bridge suspended by large metal arches, which withstand large spans imposed by the width of the avenue and the train yard. The painting of the arches also emphasizes the visual aspect of the structure (CAU/RJ, 2015).

In 2012 was built in London, the biggest public art construction in England, the “Orbit Tower”. The tower with 114,5 meters of height offers outstanding views over the “Olympic Park” and over the city of London. It was designed with 9 kilometres of hollow profiles connected with 900 welded steel joints or bolted end plates, depending on the complexity of the joint (Arup, 2015).



Firth of forth bridge, Scotland
(Wardenier *et al*, 2010)

Hall of departure, Airport of Stuttgart
(Wardenier *et al*, 2010)

Orbit Tower, London (Arup, 2015)

Railway bridge for metro station “Cidade Nova”, Rio de Janeiro(CAU/RJ, 2015)

“Estádio da Luz”, Lisbon
(Martifer Group, 2015)

Different type of joints in “Estádio da Luz”, Lisbon (Martifer Group, 2015)

Figure 1.5 - Examples of remarkable structures with hollow sections

1.3. Scope

In the present scenario, the use of hollow sections in steel structures is not a regular option, mainly due to the complex detailing of their connection with other structural elements, a fact that reduces the agile process and increase the design and construction costs. The particular behavior of these joints combined with the absence of design methods which could give a full understanding on this subject, results in a certain discrimination by designers in the use of hollow sections.

As already lay out, for hollow sections is only possible to know the resistance of the joint as a whole and not by the assembly of the behavior of each component. So, in order to get a bigger knowledge of joints with hollow sections, this work is oriented to an advanced modelling of joints between hollow and open sections subjected to static loading. The choice for this type of joints, was proposed by the "Department of Steel and Timber Structures of Technical University in Prague", with the purpose to develop a simpler and intuitive method for joints with hollow sections.

In conformity with EC3-1-8, the design of these type of joints, as any other joint which involves hollow sections, is based on empirical formulas which give an estimation of the resistant capacity of the joint as a whole, for specific loads and specific arrangements. These factors restrict the field of application and consequently the freedom to change the joint's arrangement (Jaspart *et al*, 2005).

Thus, for the development of this dissertation, a numerical modelling calibration is done, using a finite element analysis software called ABAQUS, of a T-joint with an H chord and SHS brace investigated by Chen & Wu (2015). Looking forward to make a reliable validation and verification process of the results obtained, this paper proved to be a good source to proceed with the study.

Once the numerical modelling is correctly validated, the tests provided by the "Department of Steel and Timber Structures of Technical University in Prague" set up as T joints between RHS braces and HEA or IPE chords were modelled. These joints differ from each other on the section geometry of the structural elements in such a way that it's possible to analyse the failure modes and the failure loads of each one. A total of eight joints were tested, four of them subjected to compression and the other four subjected to bending. The numerical results of these joints were compared with their experimental and analytical results. The differences between them are emphasized in the respective chapter.

To analyse the inconsistency between the behaviour of T joints with hollow (brace) and open (chord) sections with T-joints with open sections (chord and brace), a numerical modelling of four T-joints with two open sections was made. The purpose of this modelling is to compare the Moment-Rotation curves of the T-joints with open section chords, varying the braces (hollow and open sections) of them, in order to check the possibility to apply the component method for joints with hollow sections and consequently empower the opportunity to get more knowledge on the resistance, stiffness and deformation capacity of joints with this type of sections.

1.4. Chapter's organization

Considering the exposed information in the previous subchapters, where is pointed out the important role hollow sections have in this dissertation and also considering the proposed objectives, it seems relevant to carry out a brief recapitulation relating to the structure/organization of the next chapters.

In **Chapter 2** the state of art is presented, where a general literature review about hollow sections is accomplished. Hereby, the main standard documents and some important researches about the subject of the dissertation are briefly exposed.

In **Chapter 3**, the study of joints with hollow (brace) and open (chord) sections is conducted, according to Eurocode 3 and CIDECT, in a way to know the existing type of joints with this sections and to understand the existing analytical models. A comparison between these two standards is also carried out.

In **Chapter 4** is performed the calibration of the numerical modelling to be used in the course of this study. This calibration was supported by the research of Chen & Wu (2015) on T-joints between H chords and SHS braces. A validation and verification procedure of the obtained results is made, in such a way to get a well calibrated model to proceed with the aim of the dissertation.

In **Chapter 5** is done a numerical modelling of the experimental tests of the T joints between hollow and open sections provided by the "Department of Steel and Timber Structures of Technical University in Prague". A total of eight different joints were modelled, four subjected to compression over the braces and the other four subjected to bending. With the numerical models completed, the verification and comparison of the obtained results with the respective analytical and experimental results was done to take some conclusions.

In **Chapter 6** is accomplished a study of the component method for joints between open sections according to Eurocode 3. Thereby, it is also performed a numerical modelling of the referred joints with open sections, and subsequently, a comparison of their behavior with the joints with hollow sections of Chapter 5 is made.

In **Chapter 7** a conclusion about all the research of this dissertation is done and some suggestions are given for future developments on this subject.

2. STATE OF ART

2.1. Standard documents

Since the 50's, some manufacturing problems especially in welding processes were broken, opening a path for the success of steel structures. The remaining problem kept on the knowledge of the resistance capacity (Wardenier *et al*, 2010).

In 1951, W. Jamm presented the first set of recommendations for the design of joints with hollow sections in steel trusses, being this recommendations the first attempt to surpass the lack of studies about the functional meaning of joints with hollow sections (Wardenier *et al*, 2010).

The first manual of hollow joints, comes out in the beginning of the 70's, called "Hollow Structural Sections – Design manual for connections", further appearing in this decade some other guides and design manuals, for instance, the "Limit States Design Steel Manual" edited by CISC – Canadian Institute of Steel Construction (Freitas, 2013).

Still in the 70's, the Commission of the European Communities launched some investigations with the aim to elaborate a set of harmonised technical rules for structural building design and other construction works, which turn to be known as Structural Eurocodes. This project is, with no doubt, a work in a large scale that called forth a huge mobilization of human and material resources. Initially, this work was conducted by the European Commission but, shortly after the publication of Directory of Construction Products, the responsibility of Eurocodes was transferred by the European Commission to the European Committee of Standardization (CEN) through a Mandate. Thus, the purpose was to publish the Structural Eurocodes as European Norms (EN). The set of publications of Structural Eurocodes, finished in 2007, has 58 European Norms, standing out for this dissertation the norms regarding steel structures with special attention for EN 1993-1-3, hereafter called EC 3-1-8, where the design of steel joints is approached (LNEC, 2015)

In the 80's, more exactly in1982, Jacob Wardenier developed some investigations mostly based on analytical, simplified and experimental models on joints with hollow sections, which ended up like recommendation design guides for joints with these sections, called "Hollow section joints" (Wardenier, 1982).

The great improvement over joints with hollow sections also occurs in the 80's through several works, with special focus for investigations managed by CIDECT association – Comité International pour le Développment et Étude de la Construction Tubulaire. The CIDECT association, was founded in 1962 as an international association, joining the main manufacturers of hollow sections whose goal was to combine all the investigations made by them to achieve a powerful research source on the application of hollow sections throughout the world, establishing coordinated and consistent norms for steel structure applications (Wardenier *et al*, 2010).

Also in this decade, more precisely in 1984, the IIW – International Institute of Welding, became the first of only three associated bodies approved as International Standardizing Organizations by ISO to develop International Standards. The IIW has made various contributions to international standardization. Its aims are to continue in this role, to maintain its competence as an ISO approved body and to work in close cooperation with ISO on the development of international welding standards and the resolution of welding standardization problems. Following the scope of this dissertation, the IIW design guide presented as "Design, analysis and fabrication of welded structures – ISO/FDIS 14346: Static design procedure for welded hollow section joints – Recommendations" written by Zhao (2013) consists on a good information source (IIW, 2015).

In 1992, CISC launched a design guide for hollow steel joints called "Hollow Structural Sections – Connection and Trusses", written by Jeff Packer and Ted Henderson. This guide is an update from the same design guide edited in the 70's also by CISC. This association of Canada is the pioneer of steel industry in this country. It gives to other organizations the right tools, resources and contacts to help members and associates to develop the steel construction business (CISC, 2015).

In 2010 comes up a book written by J. Wardenier, J.A. Packer, X.-L. Zhao and Van der Vegte, called "Hollow sections in structural applications". This book turned out to be a relevant source, develop under an international consensus on the topic. It brings up recommendations for hollow section joints according to IIW (2009) and CIDECT (2008 and 2009), being both consistent with each other. This recommendations are considered as the maintenance basis, harmonization and future development of the Eurocode 3 (EC 3-1-8), AISC (ANSI/AISC 360) and CISC recommendations (Wardenier *et al*, 2010).

2.2. Research on joints between hollow and open sections

Due to the big investment and various investigations on joints with hollow sections, it was made a compilation of those that would be of more benefit and reliability to the fulfilment of this dissertation. These investigations can be seen in Table 5.1.

Table 2.1 – Summary of relevant investigations for this dissertation

Date	Authors	Title	Type
1977	***Wardenier et al***	Behaviour of axially loaded K- and N type joints with bracings of structural hollow sections and an I-profile as chord.	**E**
1982	***J. Wardenier***	Hollow section joints.	**A;E**
1983	***Lachal e Petit***	Improvement of design and calculation methods for steel hollow sections to H type section welded joints.	**A;E**
1985	***Koning e Wardenier***	The static strength of welded joints between structural hollow sections or between structural hollow sections and H-sections. Part 3: Joints between structural hollow section bracings and a H-section chord.	**A;E**
1985	***T.W. Giddings***	The development of recommendations for the design of welded joints between steel structural hollow sections and H-sections.	**E**
2003	***Weynand et al***	Application of the component method to joints between hollow and open sections.	**A;E;N**
2005	***Jaspart et al***	Development of a full consistent design approach for bolted and welded joints in building frames and trusses between steel members made of hollow and/or open sections – Application of the component method: Vol. 1 – Practical Guidelines.	**A**
2005	***Jaspart et al***	Vol. 2 – Progress of the scientific activities on joint components and assembly.	**A;N**
2010	***Wardenier et al***	Hollow sections in structural applications.	**A;E;N**
2012	***Jurcíková e Rosmanit***	FEM model of joint consisting RHS and HEA profiles.	**A;N**
2013	***Zhao***	Design, Analysis and Fabrication of welded structures: Static design procedure for welded hollow section joints – Recommendations.	**A**
2015	***Chen e Wu***	Behaviour of square hollow section brace - H-shaped steel chord T-joints under axial compression.	**A;E;N**

<u>Note</u>: **A**: Analytical; **E**: Experimental; **N**: Numerical

In 1977, Wardenier *et al* developed an investigation that describe the experimental results of the tests performed on joints usually applied on trusses, of K or N type with SHS, RHS or CHS braces and I or H chords. The investigations is merely experimental and it aims to create basic design recommendations and specifications for joints with these characteristics (Wardenier *et al*, 1977).

In 1982, Wardenier published a book, also referred to in the previous chapter with the name "Hollow section joints". This book presents an initial approach to the behavior and resistance of joints with hollow sections, based on analytical, simplified and experimental models. The experimental tests are performed by the author who develops some equations and design recommendations for this type of joints, subjected especially to static loads of compression, tension or bending (Wardenier, 1982).

Lachal & Petit developed a research paper called "Improvement of design and calculation methods for steel hollow sections to H type section welded joints", with the purpose to study two types of failure modes that can occur prematurely, under static loading, on welded joints between hollow and I or H sections. One of them is due to cracking near the connection of tension bracings with the flange of the chord and the other due to local buckling at the welded extremity of the compression bracings. In the first part it's described the models of joints of various types (X, K and N) with different sections and the experimental tests done to this joints, turning possible to state precisely the influence of various parameters of the joints. In the second part, the experimental results are compared with existing formulations, particularly those of French Standard NF P 22–255. Finally, some modifications are proposed to obtain better agreement of Standard with the experimental results and homogeneous security coefficients (Lachal & Petit, 1983).

Still in the 80's, more precisely in 1985, Koning & Wardenier developed an investigation designated by "The static strength of welded joints between structural hollow sections or between structural hollow section bracings and an H-section chord". The purpose of this investigations was to give more evidence to the existing experimental results in the literature, in the determination of the effective width mobilized in the flanges of H chords. The authors considered that the existing analytical models of that time were based on a very limited number of experimental tests. The obtained results by them were compared with other existing experimental tests and also with the analytical models based on IIW recommendations (1981), Eurocode 3 (1984) and Dutch RB'82 (1982). The type of joints considered were X, K and N joints with gap or overlap. In the end of the investigation the authors put forward, for this Norms, some modifications related to effective width criteria (Koning & Wardenier, 1985).

In the same year, 1985, included in the CIDECT project, Giddings lead an investigation entitled by "The development of recommendations for the design of welded joints between steel structural hollow sections and H-sections". This investigations is exclusively experimental and its purpose is to provide theoretical information in a large scale, in such a way that it would allow a development which covers all the required parameters needed for the design of joints with hollow sections and joints with open sections. As it's also clear that either in this investigation or in the others referred, the authors want to provide a base for future researches and development of European recommendations (Giddings, 1985).

In the beginning of the new millennium, in 2003, Weynand *et al* developed a research intended to demonstrate the applicability of the component method on the design of joints with hollow sections, in order to create an initial design philosophy common to structural joints independently of the type of sections that take part into the joint. The component method gives a lot of advantages in comparison with empirical approaches. The investigation is complemented with preliminary studies in the way the design Norms of those type of joints can be transferred to the component method. The joints studied in this project were T-joints between RHS chords, filled or not with concrete, and I braces (Weynand *et al*, 2003).

In 2005, Jaspart *et al*, conducted an investigation called "Development of a full consistent design approach for bolted and welded joints in building frames and trusses between steel members made of hollow and/or open sections – Application of the component method: Vol1 – Practical Guidelines". This research in inserted in CIDECT project and intends to join information to develop a unified approach in the design of joints independently of the type of connected sections, enlarging the field of application of the component method for joints with hollow sections. This investigation is exclusively analytical, where the authors do a deep research in the existing literature about relevant information to continue CIDECT project. The studied joints are T-joints with CHS or RHS chords, filled or not with concrete, and I braces, splices between hollow sections, welded joints between hollow sections and base column joints (Jaspart *et al*, 2015*a*).

Still in 2005, Jaspart *et al*, developed the second volume of the previous investigation, called "Vol.2 – Progress of the scientific activities on joint components and assembly". In this volume, as a follow-up of the first one, it's demonstrated a advantage of using the component method through the analytical and numerical study of a T-joint between a CHS chord and a plate acting as the brace, with the aim to verify the plastic mechanisms between the plate and the hollow member and develop an equation which correlates the analytical and numerical models, in this particular case. Whereas this research is a little bit limited joints with CHS chords, it is required, by the authors, future investigations to obtain models completely validated for the use of other type of hollow sections (Jaspart *et al*, 2005*b*).

Wardenier *et al*, developed a book called "Hollow sections in structural applications" under the shelter of IIW (2009) and CIDECT (2008 and 2009) recommendations. This book was already referred in this dissertation in subchapter 5.1, "Standard documents" (Wardenier, 2010).

In 2012, Jurčiková & Rosmani, developed a scientific paper where the authors want to study the behaviour of joints between hollow and open section. They study an N type joint with RHS braces and H chord. It was performed some experimental tests and a numerical modelling on this joint looking forward to do a validation of the results based on Force-displacement curves. They also did a verification of the results with the obtained using analytical models presented in the actual Norms (Jurčiková e Rosmani, 2012).

In 2013, Zhao developed a design guide of welded joints with hollow sections, called "Design Analysis and Fabrication of welded structures: Static design procedure for welded hollow section joints – Recommendations". This guide, inserted in IIW project, is consistent with the CIDECT design recommendations for joints, being useful for analytical advice (Zhao, 2013).

In the year of 2015, Chen & Wu accomplished an investigation which had the aim to study the behavior of a T joint between SHS (brace) and I (chord) sections, subjected to axial compression over the brace. The authors accomplished eight experimental tests whose geometrical parameters of the joints vary from each other and they added stiffeners on the bottom of the braces (top of chord flanges) in some of the joints to verify its different behavior with or without them. Subsequently, the authors made a numerical modelling of those joints and confront its results with the experimental results to validate them. The main purpose in the research is, therefore, to compare the obtained results with those obtained by EC 3-1-8 for the same joints. They concluded that, for the studied joints, it's unreliable to design these type of joints using the Eurocode 3 when the joints are subjected to axial compression (Chen & Wu, 2015).

3. THE JOINT ACCORDING TO EUROCODE 3 AND CIDECT

3.1. Introduction

Joint is said to be the location in which two or more elements converge. For design purpose, it is a set of basic components (weld seams, bolts, plates, flanges, web, etc.) necessary to represent the joint behavior on the transmission of relevant forces throughout the structure (CEN, 2010*b*).

The Chapter 7 of EC 3-1-8, includes detailed application recommendations for the resistance of joints under static loading, with uniplanar or multiplanar geometries, acting upon structures with CHS, RHS or SHS sections (Simões da Silva & Santiago, 2003). In subchapter 7.6 of this document there's the information for the design of joints between hollow and open sections.

The IIW and CIDECT recommendations are consistent with each other and they are the root for the design guide of Wardenier *et al* (2010). The design of joints between hollow and open sections can be done based on chapter 12 of this design guide.

The application rules are valuable for rolled profiles manufactured according to EN 10210-2 (CEN, 2006). The nominal thickness of hollow sections must be between 2,5 and 25 millimetres, unless special rules are taken to guarantee special properties along the thickness (Simões da Silva & Santiago, 2003).

3.2. Type of joints

Among the different type of joints with hollow sections, in this dissertation it's intended to study the uniplanar joints between hollow and open sections, whereby the open section is the chord and the hollow section is the brace.

In Figure 3.1 can be observed the distinct type of uniplanar joints between hollow and open sections. Is noteworthy that Y and N joints are similar to T or K joints, but they have variance on the angle between the chord and braces. This variance is taken into account in the resistance formulas, as will be shown afterwards.

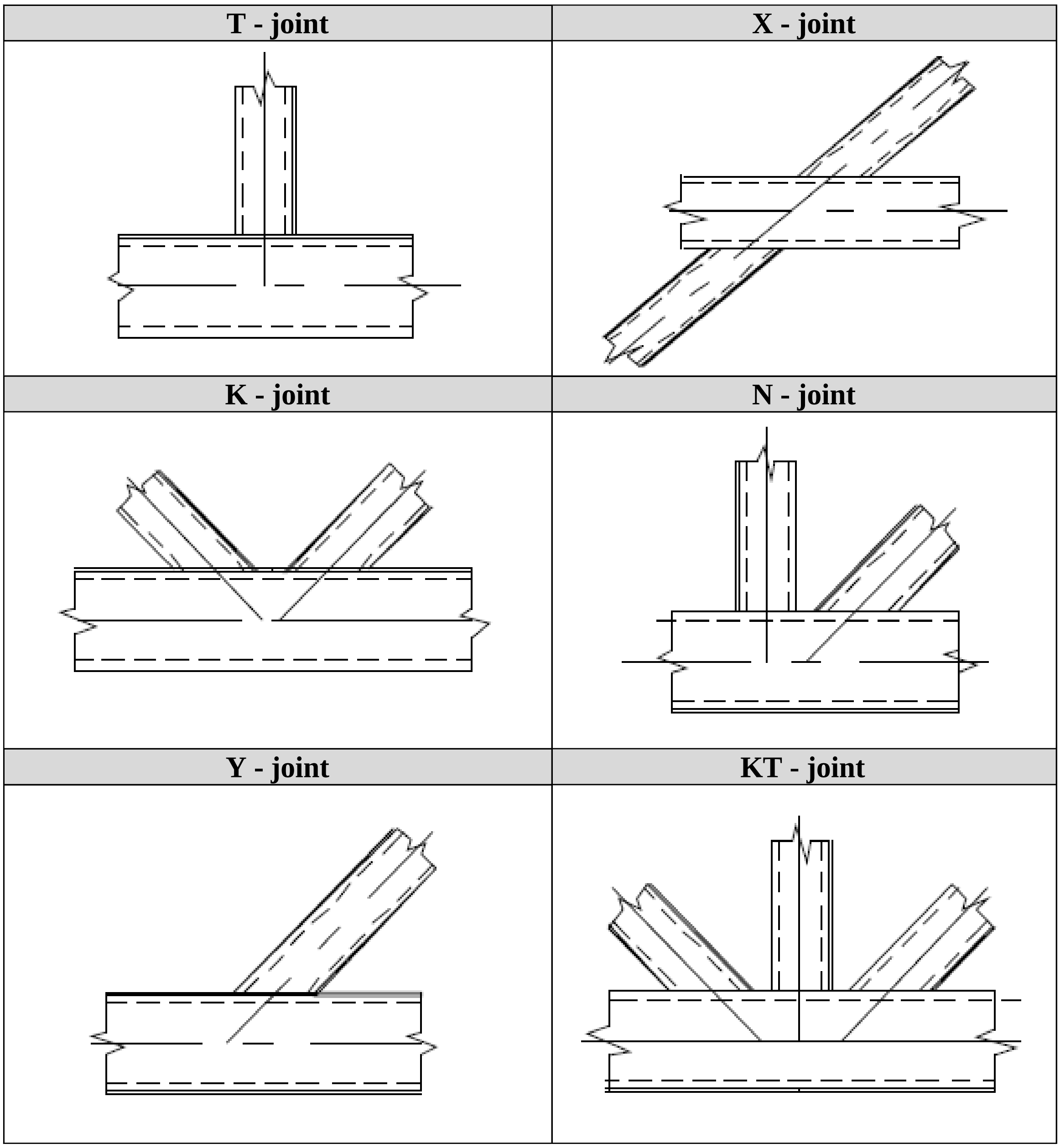


Figure 3.1 – Types of joints between hollow and open sections (CEN, 2010*b*)

According to Zhao (2013), the classification of the type of joints might vary depending on the load applied to the joints. This means that, for instance, if the central brace of the KT joint presented in Figure 3.1 is not subjected to any force, then, the joint can be classified as a K joint instead of a KT joint.

The classification of joints can also vary depending on the eccentricity of the brace axes in relation to the chord axis and, as already referred, regarding the applied loads (Figure 3.2).

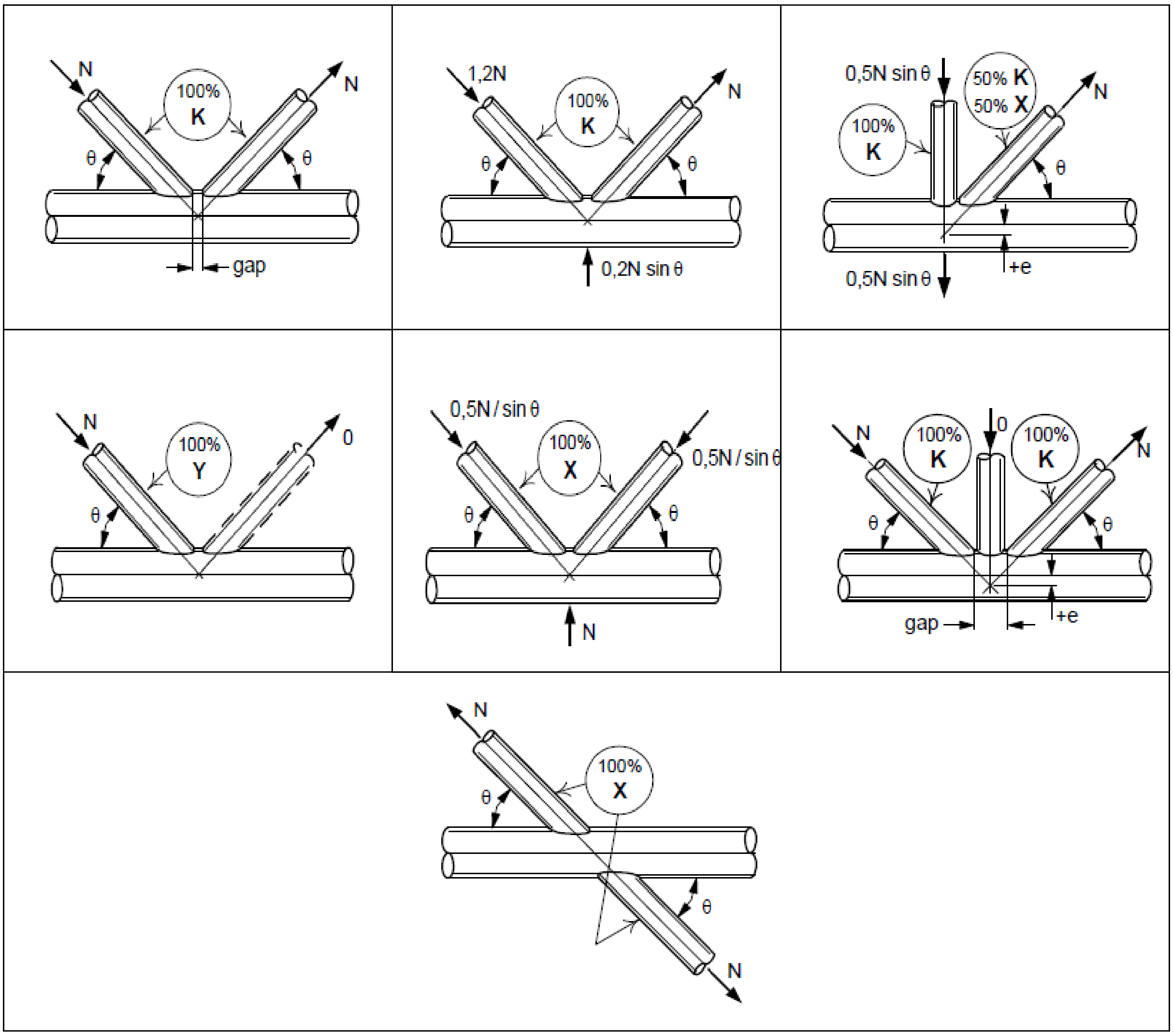


Figure 3.2 – Types of joints regarding the eccentricity and the applied loads to the joint (Wardenier *et al*, 2010)

3.3. Analytical models

This chapter holds a literature review of the existing analytical models in EC 3-1-8 and CIDECT recommendations, about joints between hollow and open sections for different types of joints.

For an easy understanding of the variables shown in the analytical models, in Figure 3.3 is presented, as an example, a K joint with the respective variables. The meaning of each variable can be seen in chapter "Symbols".

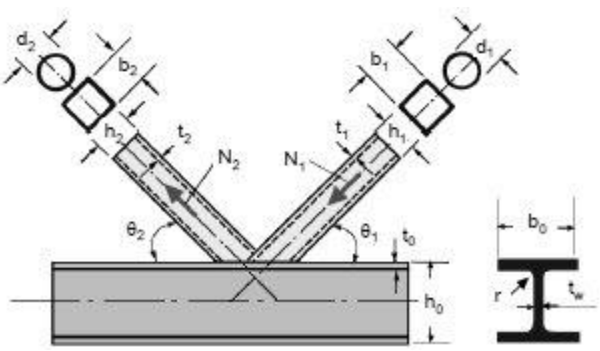


Figure 3.3 – Variables used in joints between hollow and open sections
(Wardenier *et al*, 2010)

3.3.1. General requirements

The EC 3-1-8 and CIDECT establish for hot finished hollow sections that their nominal value for yield stress should not exceed 460 N/mm^2. For elements in which the nominal value of the yield stress is higher than 355 N/mm^2, the design values of static resistance should be reduced by a coefficient of 0,9.

Beyond this requirements, the recommendations of CIDECT still recommend that in cases where the yield stress exceeds 0,8 of the ultimate stress, the yield stress should be considered equal to 0,8 of the ultimate stress.

The EC 3-1-8 considers that, in K-joints with gap or overlapped, the bending moments due to the eccentricity related to the chord axis could be neglected on the design of joints if the eccentricity is under the following limits: -0,55$\boldsymbol{h_0} \leq \boldsymbol{e} \leq$0,25$\boldsymbol{h_0}$.

CIDECT states that if the same eccentricity limit is $\mathbf{e} \leq 0{,}25\boldsymbol{h_0}$ or $\mathbf{e} \leq 0{,}25\boldsymbol{d_0}$, the bending moments due to the eccentricity must be taken into account in joint resistance. If these limits are exceeded, a part of the resulting moments at the joint must be distributed to the braces.

EC 3-1-8 and CIDECT also state some similar rules, such as:

- The angles between the chords and braces, as the angles between the adjacent braces should be bigger than 30º.
- Joints with gap between the braces should have a limit gap equal or inferior than $(\boldsymbol{t_1} + \boldsymbol{t_2})$, thus ensuring the existence of required space to implement an acceptable welding.
- In overlap joints, when the braces have different thicknesses and/or different resistance classes, the element with the lower $(\boldsymbol{t_i f_i})$ value should overlap the other one.
- When the overlap braces have different widths, the narrowest element must overlap the widest.
- In gap or overlap K joints, when there is eccentricity relative to the axis chord, a primary bending moment is produced which must be taken into account in the design of trusses.

3.3.2. Parameters included in the analytical models

The difference between the proposed parameters of EC 3-1-8 and CIDECT, is that CIDECT recommends one more equation related to the effective width for CHS braces when overlapped by another equal brace and presents different equations for RHS and CHS, instead of Eurocode that recommends the same equations for both type of braces.

The parameters included in the analytical models are presented in Table 3.1, based on CIDECT recommendations which seemed to fit better in this chapter due to its frequent and recent upgrades over the years, based on a large number of experimental researches which put these recommendations in a reliable place for joint designers.

Table 3.1 – Parameters included in the analytical models

<table>
<tr><th>RHS braces</th><th>CHS braces</th><th>Type of joint</th></tr>
<tr><td>$b_e = t_w + 2r + 7t_0 \frac{\sigma_{y0}}{\sigma_{yi}}$;
but $b_e \leq b_i + h_i - 2t_i$</td><td>$d_e = t_w + 2r + 7t_0 \frac{\sigma_{y0}}{\sigma_{yi}}$;
but $d_e \leq 0{,}5\pi(d_i - t_i)$</td><td rowspan="4">T,X and K and N with gap between braces</td></tr>
<tr><td>$b_w = \frac{h_i}{sen\ \theta_i} + 5(t_0 + r)$;
but $b_w \leq \frac{2t_i}{sen\ \theta_i} + 10(t_0 + r)$</td><td>$d_w = \frac{d_i}{sen\ \theta_i} + 5(t_0 + r)$;
but $d_w \leq \frac{2t_i}{sen\ \theta_i} + 10(t_0 + r)$</td></tr>
<tr><td colspan="2">$A_s = A_0 - (2 - \alpha)b_0 t_0 + (t_w + 2r)t_0$</td></tr>
<tr><td>$\alpha = \sqrt{\frac{1}{1 + \frac{(4g^2)}{(3t_0^2)}}}$</td><td>$\alpha = 0$</td></tr>
<tr><td>$b_{e,ov} = \frac{10}{\frac{b_j}{t_j}} \frac{\sigma_{yj}\, t_j}{\sigma_{yi}\, t_i} b_i$; but $\leq b_i$</td><td>$d_{e,ov} = \frac{12}{\frac{d_j}{t_j}} \frac{\sigma_{yj}\, t_j}{\sigma_{yi}\, t_i} d_i$; but $\leq d_i$</td><td>K and N with overlapped braces</td></tr>
</table>

3.3.3. Range of validity

For CHS and RHS braces under compression, CIDECT only recommends the use of sections of Class 1, but on the other side, EC3-1-8 allows sections of Class 1 or 2.

Another difference between the two standard documents, comes up with the cross section limit ratios for RHS braces under tension, because EC 3-1-8 recommends smaller limits comparing to CIDECT.

In Table 3.2 is summarized the information related to the range of validity based on the CIDECT recommendations for joints with no overlap.

Table 3.2 – Range of validity for welded joints between CHS, RHS and SHS braces and I or H chords

<table>
<tr><th colspan="3"></th><th>X - joint</th><th>T,Y and K and N with gap between braces</th></tr>
<tr><td rowspan="3">I- or H-section chord</td><td rowspan="2">Compression</td><td>Flange</td><td colspan="2">Class 1 or 2</td></tr>
<tr><td>Web</td><td>Class 1 and $d_w \leq 400\ mm$</td><td>Class 1 or 2 and $d_w \leq 400\ mm$</td></tr>
<tr><td colspan="2">Tension</td><td colspan="2">None</td></tr>
<tr><td rowspan="2">CHS braces</td><td colspan="2">Compression</td><td colspan="2">Class 1</td></tr>
<tr><td colspan="2">Tension</td><td colspan="2">$\frac{d_i}{t_i} \leq 50$</td></tr>
<tr><td rowspan="3">RHS braces</td><td colspan="2">Compression</td><td colspan="2">Class 1</td></tr>
<tr><td colspan="2">Tension</td><td colspan="2">$\frac{h_i}{t_i} \leq 40 \,;\, \frac{b_i}{t_i} \leq 40$</td></tr>
<tr><td colspan="2">Aspect ratio</td><td colspan="2">$0{,}5 \leq \frac{h_i}{b_i} \leq 2{,}0$</td></tr>
<tr><td colspan="3">Gap</td><td></td><td>$g \geq t_1 + t_2$
for K - joints</td></tr>
<tr><td colspan="3">Eccentricity</td><td></td><td>$g \geq 0{,}25h_0$
for K - joints</td></tr>
<tr><td colspan="3">Brace angle</td><td colspan="2">$\theta_i \geq 30^o$</td></tr>
<tr><td colspan="3">Yield stress</td><td colspan="2">$\sigma_{yi} \leq \sigma_{y0}$ and $\sigma_{yi} \leq 0{,}8\sigma_u$</td></tr>
</table>

Regarding the range of validity for overlapped joints, Table 3.3 brings together the information which considers more general rules, recommended by CIDECT, namely regarding cross section geometry ratios.

For these kind of joints, the ratio limit for RHS cross section recommended by CIDECT is bigger than the ratio recommended by EC 3-1-8.

Table 3.3 – Range of validity for overlap welded joints between CHS, RHS and SHS braces and I or H chords

<table>
<tr><th colspan="5">K - or N – overlap joints</th></tr>
<tr><td rowspan="2">General</td><td>$\frac{d_i}{d_0}$ and $\frac{d_j}{d_0} \geq 0{,}20$
$\frac{b_i}{b_0}$ and $\frac{b_j}{b_0} \geq 0{,}25$
$\frac{d_i}{b_0}$ and $\frac{d_j}{b_0} \geq 0{,}25$</td><td>$\frac{d_i}{d_0} \geq 0{,}20$
$\frac{d_i}{d_j} \geq 0{,}75$
$\frac{b_i}{b_j} \geq 0{,}75$</td><td>$t_i$ and
$t_j \leq t_0$
$t_i \leq t_j$</td><td>$\theta_i \ e \ \theta_j \geq 30^0$
$O_v \geq 25^0$
$\sigma_{yi} \leq \sigma_{y0}$
$\sigma_{yi} \leq 0{,}8\sigma_u$</td></tr>
<tr><td colspan="4">$O_{v,lim}$ = 60% if the hidden seam is not welded
$O_{v,lim}$ = 80% if the hidden seam is welded</td></tr>
<tr><td rowspan="3">I - or H - chord</td><td rowspan="2">Compression</td><td>Flange</td><td colspan="2">Class 1 or 2</td></tr>
<tr><td>Web</td><td colspan="2">Class 1 or 2 and
$d_w \leq 400 \ mm$</td></tr>
<tr><td>Tension</td><td colspan="3">None</td></tr>
<tr><td rowspan="2">CHS braces</td><td>Compression</td><td colspan="3">Class 1 or 2 and
$\frac{d_1}{t_1} \leq 50$</td></tr>
<tr><td>Tension</td><td colspan="3">$\frac{d_2}{t_2} \leq 50$</td></tr>
<tr><td rowspan="3">RHS braces</td><td>Compression</td><td colspan="3">Class 1 or 2
$\frac{h_1}{t_1} \leq 40$; $\frac{b_1}{t_1} \leq 40$</td></tr>
<tr><td>Tension</td><td colspan="3">$\frac{h_2}{t_2} \leq 40$; $\frac{b_2}{t_2} \leq 40$</td></tr>
<tr><td>Aspect ratio</td><td colspan="3">$0{,}5 \leq \frac{h_i}{b_i} \leq 2{,}0$ and $0{,}5 \leq \frac{h_j}{b_j} \leq 2{,}0$
$\frac{h_i}{b_i} = 1$ and $\frac{h_j}{t_j} = 1$</td></tr>
</table>

3.3.4. Effective width criteria

Due to the geometric characteristics and difference of stiffness on each part of I or H sections, comes up the need to consider an effective width. This concept is based on the non-uniform stresses and strains in the joint, more exactly between the edges and the central part of the flange in contact with the brace. This phenomenon can be observed in Figure 3.4.

The effective width is the perimeter of the hollow sections capable of transmitting the forces of imminent collapse. Therefore, in the case of a decrease on the effective width due to the increase of loading, it can occur a failure in the section between the tensioned hollow brace and the open chord can occur, or on the other hand, it can occur local buckling in the edge of the tensioned element (Simões da Silva& Santiago, 2003).

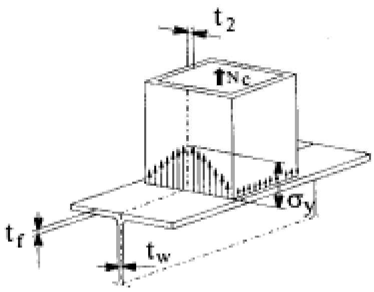


Figure 3.4 – Stress and deformation distribution in the edge of a RHS brace (Simões da Silva & Santiago, 2003)

There are some procedures that can be made to increase the effective width of this kind of joints, such as, through the use of reinforcements between the lower and superior flange of open sections, as noticed in Figure 3.5 (CEN, 2010*b*).

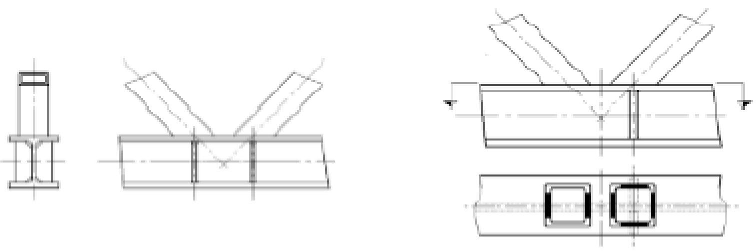


Figure 3.5 - Reinforcements for open section chords to increase the effective width (CEN, 2010*b*)

3.3.5. Failure Modes

- **<u>Local yield of the brace</u>**

In Figure 3.6 is shown the type of failure which corresponds to the local yield of the brace. As it can be seen from the cutting section A-A in the figure, when the brace is subjected to axial forces, the yield begins in the area of the brace which is closer to the web of the I or H profile. This is an occurrence that happens due to the decreasing of the effective width decrease, explained in the previous subchapter.

The Equation (3.1) reflects the resistance of a CHS, RHS or SHS brace when subjected to compression or tension.

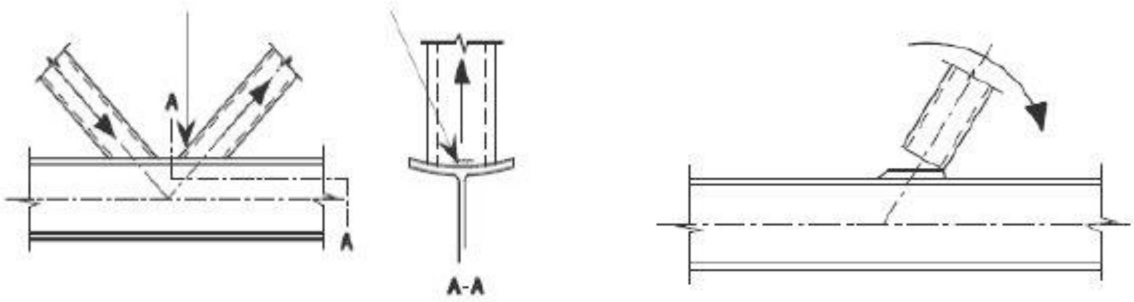


Figure 3.6 – Local yield of the brace (CEN, 2010*b*)

$$N_{1,Rd} = 2\sigma_{yi} t_i b_e \tag{3.1}$$

- **<u>Chord web failure</u>**

The Figure 3.7 shows the type of failure correspondent to the chord web failure.

The Equation (3.2) represents the resistance capacity of the chord web when a CHS, RHS or SHS brace is subjected to a tension or compression force.

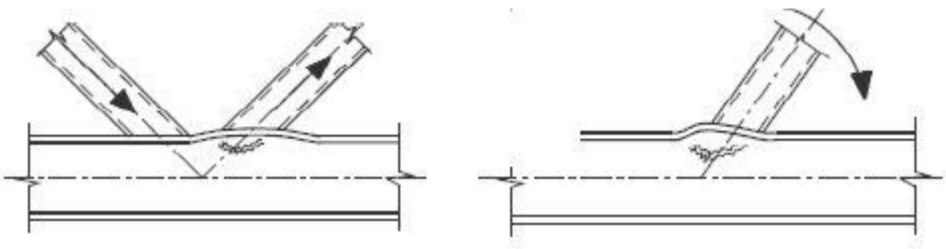


Figure 3.7 – Chord web failure (CEN, 2010*b*)

$$N_{i,Rd} = \frac{\sigma_{y0} t_w b_w}{sen\ \theta_i} \tag{3.2}$$

- **<u>Chord shear failure</u>**

In Figure 3.8 is shown the corresponding type of chord shear failure.

The Equation (3.3) reflects the resistance of a CHS, RHS or SHS brace when subjected to tension or compression forces. It should be noted that the value of 0,58 in the equation refers to the quotient $(1/\sqrt{3})$ which reflects the effect of the resistant shear stress of the chord.

The Equation (3.4) reflects the resistance of an I or H chord when subjected to shear forces.

According to Simões da Silva & Santiago (2003), this kind of failure is the most common failure mode in K or N joints with gap between their braces.

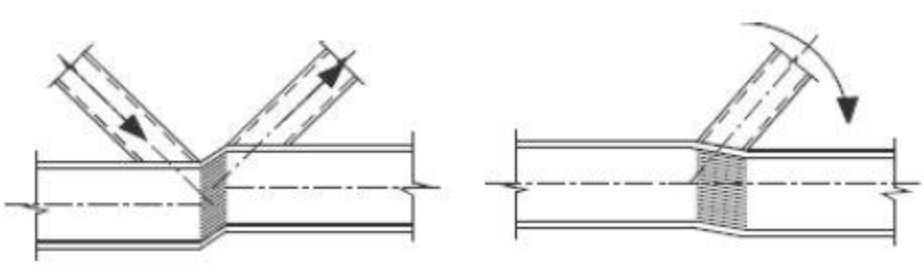


Figure 3.8 – Chord shear failure (CEN, 2010*b*)

$$N_{i,Rd} = \frac{0{,}58\sigma_{y0} A_s}{sen\ \theta_i} \tag{3.3}$$

$$N_{0,Rd} = (A_0 - A_s)\sigma_{y0} + A_s \sigma_{y0} \sqrt{1 - \left(\frac{V_{Ed}}{V_{pl,Rd}}\right)^2} \tag{3.4}$$

- **<u>Local yielding of overlapping brace</u>**

The CIDECT recommendations, in contrast to EC3-1-8, distinguish the effective width equations to be used in overlapped CHS or RHS braces, and consequently the equations are different for this parameter. These equations are presented in Table 3.4.

The Equation (3.5) represents the resistance of a CHS, RHS or SHS brace when subjected to tension or compression forces and when one of the braces is overlapped by a brace of the same type.

For joints with overlapping braces, the EC 3-1-8 only identifies this failure mode, for different overlapping percentages. CIDECT identifies more two failure modes (presented soon after) besides this one.

In Figure 3.9 is presented a type of joint with overlapping braces.

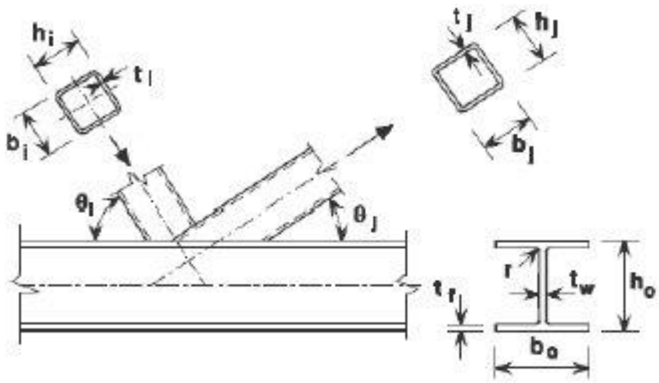


Figure 3.9 – Type of joint with overlapping braces
(CEN, 2010*b*)

$$N_{i,Rd} = \sigma_{yi} t_i l_{b,eff} \tag{3.5}$$

Table 3.4 - Effective perimeter of overlapping braces

	CHS braces	**RHS braces**
$\mathbf{25\% < O_v < 50\%}$	$l_{b,eff} = \frac{\pi}{4}(2d_i + d_{ei} + d_{e,ov} - 4t_i)$	$l_{b,eff} = \left(\frac{O_v}{50}\right) 2h_i + b_{ei} + b_{e,ov} - 4t_i$
$\mathbf{50\% < O_v < 100\%}$		$l_{b,eff} = 2h_i + b_{ei} + b_{e,ov} - 4t_i$
$\mathbf{O_v = 100\%}$	$l_{b,eff} = \frac{\pi}{4}(2d_i + 2d_{e,ov} - 4t_i)$	

- **Local chord member yielding with overlapping braces**

The Equation (3.6) expresses the I or H chord resistance when subjected to tension or compression forces combined with bending moment of joints with overlapping braces.

$$\left(\frac{N_0}{N_{pl,0}}\right)^c + \frac{M_0}{M_{pl,o}} \leq 1 \,;\, c = 1 \text{ for I or H section} \tag{3.6}$$

- **Brace shear when overlapped by another brace of the same type**

The Equation (3.7) consists on the CHS, RHS or SHS brace resistance to shear forces when overlapped by a brace of the same type.

$$N_i cos\theta_i + N_j cos\theta_j \leq V_s^* \,;\, \mathrm{O_{v,lim}} < \mathrm{O_v} < 100\% \tag{3.7}$$

The value of the overlap limit, $\mathrm{O_{v,lim}}$, may be obtained regarding Table 3.3.

- **Local yielding of brace when subjected to bending moment**

In Figure 3.10 is presented the referred type of joint submitted to bending moment.

The Equation (3.8) consists on the resistance of a RHS or SHS brace/beam when subjected to bending moment.

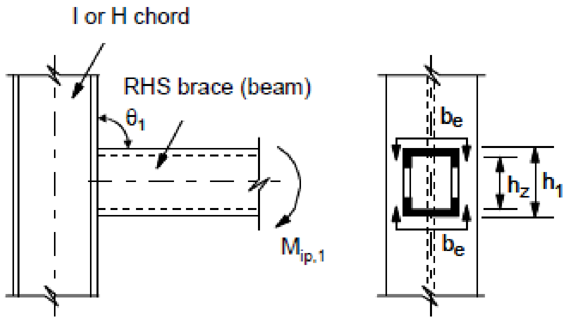


Figure 3.10 – T joint subjected to bending moment (Wardenier *et al*, 2010)

$$M_{ip,1,Rd} = \sigma_{y1} t_1 b_e h_z \tag{3.8}$$

- **Chord web failure when subjected to bending moment**

The Equation (3.9) consists on the resistance of a I or H chord/column when subjected to bending moment in a T joint like the one presented in Figure 3.10.

$$M_{ip,1,Rd} = 0{,}5\sigma_{y0}t_w b_w(h_1 - t_1) \tag{3.9}$$

4. CALIBRATION OF THE NUMERICAL MODEL

4.1. Introduction

Four decades ago computational analysis of structural joints was treated by some researchers as a non-scientific matter. Two decades later it was already a widely accepted addition or even extension of experimental and theoretical work. Nowadays, computational analysis, in particular computational mechanics and fluid dynamics, is commonly used as an indispensable design tool and a catalyst of many relevant research fields. The recommendation for design by advanced modelling in structural steel is already hidden but ready to be used in chapter 5 and Annex C off EN 1993-1-5 (CEN, 2010*c*) (Wald *et al*, 2014).

In a general manner, computational solution represents a great geometrical and mechanical solution comparing to real technical ways of experimental tests, when these real techniques are laborious and reveal expensive methods. The numerical set up behaves like a whole, divided by finite elements, with their own stiffness matrices. The assembly of these matrices will lead to the global stiffness matrix of the whole structure, assembled to ensure compatibility between the degrees of freedom and between the boundary conditions.

Considering these aspects, arises the need to apply a formal procedure called "Validation and Verification". Validation compares the numerical solution with experimental data, whereas verification uses comparison between computational solutions or experimental tests with highly accurate analytical or numerical solutions. To have better understanding, in contrast to numerical solutions used in the validation stage, the numerical solutions applied for verification can represent mathematical models with little physical importance. The verification on the analyst's side is based on the test agreement with existing correct results (Wald *et al*, 2014).

The validation and verification process is made based on the Chen & Wu (2015) research, referred on the last paragraph of subchapter 2.2. The authors analysed the behavior of T joints with H chord and SHS brace under compression forces, with and without stiffeners. For this dissertation, it was chosen a joint without stiffeners to be closer of its aim.

In Figure 4.1, it's represented the joint tested by Chen & Wu, and the respective arrangement of strain gauges (top and right side of the figure) and displacement transducers (bottom and right side of the figure). It should be pointed out that the transducers “D1 – D4” are not so relevant for this dissertation because they measure the chord flange deformations, instead of transducers “D5” and “D6”, which measure the vertical displacement of specimens (Chen & Wu, 2015).

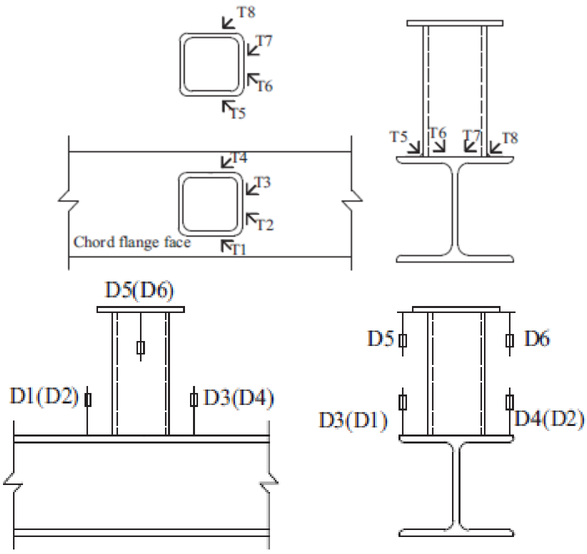



Figure 4.1 – Test set up and arrangement of strain gauges and displacement transducers (Chen & Wu, 2015)

The validation and verification results are presented in subchapter 4.3.

4.2. Description of the numerical model

In the following subchapters are described the choices related to the numerical modelling referring the relevant aspects for a correct and reliable modelling. Within the scope of the present dissertation and with the help of the finite element software ABAQUS, it was required to follow the next conditions:

- Type of finite element
- Material and imperfections
- Type of structural analysis
- Mesh
- Welds
- Support and loading conditions
- Limit deformation

The choices related to the previous conditions were made regarding the test set up presented in Figure 4.1. The dimensions of the test set up and the geometrical characteristics of the elements are presented in Figure 4.2 and Table 4.1.

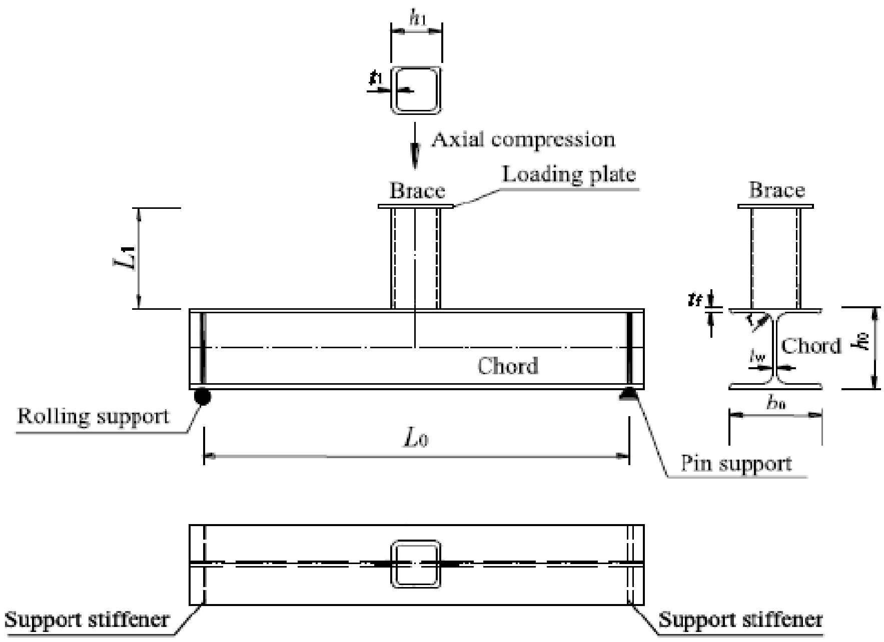


Figure 4.2 – Test set up of T joints with H chord and SHS brace (Chen & Wu, 2015)

Table 4.1 – Geometrical characteristics and test set up dimensions

T-joint of Chen e Wu			
Chord (mm)		***Brace (mm)***	
b_0	200	h_1	120
h_0	200		
t_f	12	t_1	2,8
t_w	8		
r	16	L_1	250
L_0	1040		

4.2.1. Type of finite elements

Figure 4.3 shows the three families of most used elements on numerical modelling with finite elements.

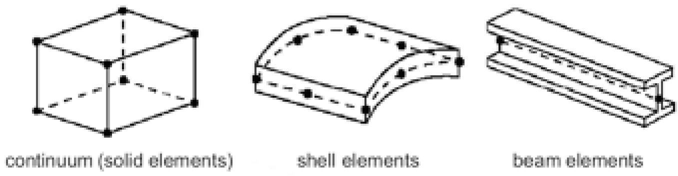


Figure 4.3 – Families of finite elements (ABAQUS 6.13, 2013)

A numerical model with solid elements can be time-consuming, particularly for bodies with a bid size and tri-dimensional problems. To reach a profitable solution in terms of time, shell elements and beam elements are a good solution. However, to produce acceptable solutions, the smaller dimensions of the beam or shell elements should have a relation of (1/10) with the bigger dimensions. Usually the smaller dimension belongs to the thickness (ABAQUS 6.13, 2013).

Facing some doubts when choosing the type of elements which would fit better on the joint to be studied in this dissertation, a total of eight numerical models were performed, four with shell elements and the other four with solid elements. The mesh was refined in each model to achieve the one that could give closer results to the experimental data.

4.2.2. Material and Imperfections

The material of which the joint is made is steel. This was defined as an isotropic material which is a material with the same characteristics in all directions or, in other words, a material with symmetrical characteristics in relation to one arbitrary orientation plane (Dias da Silva, 2004).

The material behavior was simulated through a bilinear stress-strain curve, different for each element of the referred joint.

In Figure 4.4 is presented the type of curve and in Table 4.2 can be seen the steel properties of each element of the joint tested by Chen & Wu (2015).

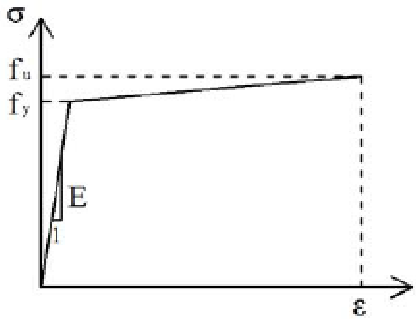


Figure 4.4 – Bilinear stress-strain curve
(Wald *et al*, 2014)

Table 4.2 - Mechanical properties of steel used in Chen & Wu tests

Mechanical properties		
	Chord	***Brace***
Young Modulus - ***E (Gpa)***	206	200
Yield stress - $\boldsymbol{\sigma_y}$ **(*MPa*)**	292	363
Ultimate stress - $\boldsymbol{\sigma_u}$ **(*MPa*)**	438	428
Final strain - $\boldsymbol{\varepsilon_f}$(**%**)	25,54	23,73
Poisson's ratio - $\boldsymbol{\nu}$	0,29	0,31

The non-linearity of the material is taken into account using the plastic behavior, visible by the steel properties of each element of the joint on Table 4.2. This behavior allows to consider the yield criteria of von Mises within an elastic-plastic constitutive law of the material.

The geometrical non-linearity, is considered within the analysis type and influenced by the material type (liquid or solid) on the selection of a mathematical formulation to integrate the points of the mesh. The software ABAQUS, gives two different kind of formulations for this purpose, the Lagrangian and the Eulerian formulation. The Lagrangian formulation, is more appropriate for solid elements analysis and the Eulerian formulation, is more current in fluid analysis.

Even though, to be well understandable how the phenomenon of non-linearity is treated on the modelling, it has to be noticed how this is included on the software. This is considered within each load increment through the Lagrangian formulation. This means that, in the end of each load increment, or group of increments, the coordinates of the structure are updated to the deformed configuration. In the next increment, these coordinates start to belong to an “undisturbed” configuration and the stresses already installed due to the previous increments are treated as initial stresses referent to a deformation equal to zero. This process goes on, increment by increment, until the end of the analysis. In finite elements, the mesh keeps connected to the integration points during all the process if the correct features are applied (Dias da Silva, 2002).

The load increments are intervals of loads, useful in a way that it allows to analyse the joint behavior in a restrict number of intervals demanded by the user, according to the joint to be analysed and regarding the deformation capacity of the material.

Thus, as the applied loads on the joint are slowly enough, it is acceptable to consider the inertia forces negligible and a static analysis on the joint can be used (Azevedo, 2003).

Considering all these referred aspects, the type of analysis considered in the numerical modelling was a static analysis with an elatic-plastic steel behavior and a Lagrangian formulation for the integration points of the mesh to take into account the geometrical non-linearity.

4.2.3. Mesh

The choice of the finite element mesh is directly related to the degrees of freedom, because the degrees of freedom are associated with each node of the mesh. By setting the mesh or the number of nodes, it is necessary to do a “Quality *vs* Time” reflection, because the greater the number of nodes, the better the quality of the results will be, but on the other hand, the computational cost will be higher. The degrees of freedom of each node also vary depending on the boundary conditions between the elements, the supporting conditions, loading conditions and the choice of the type of elements (solid or shells) admitted for the numerical model.

The type of the mesh elements (quadratic or triangular) are an important aspect to take into account for the refinement process, being necessary for such process, to have in consideration the shape of the mesh for the structural elements (undistorted or distorted).

In Figure 4.5 can be observed on the top of the figure different type of meshes. On the bottom of the figure are presented the two different shapes of shell or solid elements.

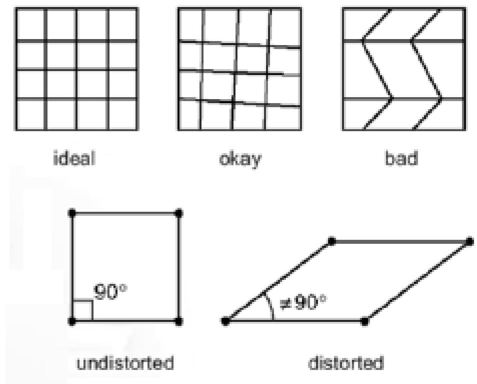


Figure 4.5 – Type of finite element meshes and shape of shell or solid elements (ABAQUS 6.13, 2013)

In this particular case, as the shell or solid elements do not turn the mesh distorted, the solution was to use quadratic elements or four nodes for shell elements and three-dimensional elements of eight nodes for solid elements. The size of the mesh elements varies within the mesh refinement given to each of the different modellings.

The stiffness, mass and volume of an element are calculated numerically through the called "integrations points". These points influence the element's behavior, which can be analysed by a full or reduced integration. The difference between these types of integration is on the number of points needed to integrate the polynomials of the matrices required to develop the finite element method.

In the library of ABAQUS, there are different numerical techniques to integrate certain amount of points. In this dissertation, the Simpson integration technique is used, which is valid for shells or solids since they are physically homogenous and have a non-linear material behavior (ABAQUS 6.13, 2013).

In Figure 4.6 are represented elements with different number of integration points regarding its integration order. This integration order is related to the shape of mesh elements as shown in the figure (ABAQUS 6.13, 2013).

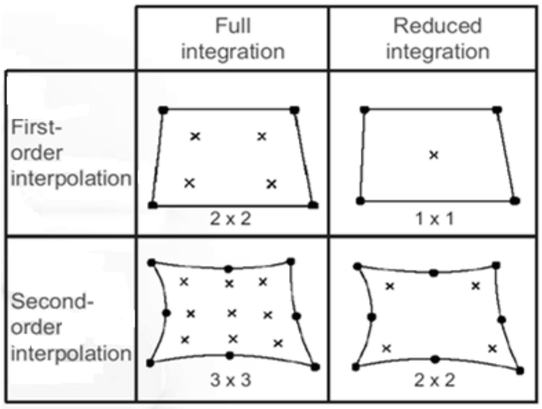


Figure 4.6 – Integration points (ABAQUS 6.13, 2013).

After considering all these aspects, it is necessary to do the mesh refinement in order to approach as much as possible the numerical results with the experimental data.

In this step of mesh refinement, became important to do some partitions on the joint model, in such a way to obtain more discretized meshes in some particular areas, as those near the intersection between the two structural elements of the joint. These areas are the joint regions with high stress concentration.

4.2.4. Welds

The welds between the structural elements, were not implemented in the finite element model by considering their negligible effect on the behavior of T-joints when subjected to compression (Chen & Wu, 2015).

4.2.5. Support and loading conditions

Considering the Figure 4.2, the support conditions were modelled with the same conditions of the experimental tests.

This means that for a pinned support, the horizontal and vertical degrees of freedom in $\boldsymbol{x}$ and $\boldsymbol{y}$ axis respectively, were constrained. For the other pinned support, in addition to the same constrains, it was also constrained the horizontal degree of freedom in $\boldsymbol{z}$ axis. The axis system and support conditions can be observed in Figure 4.7.

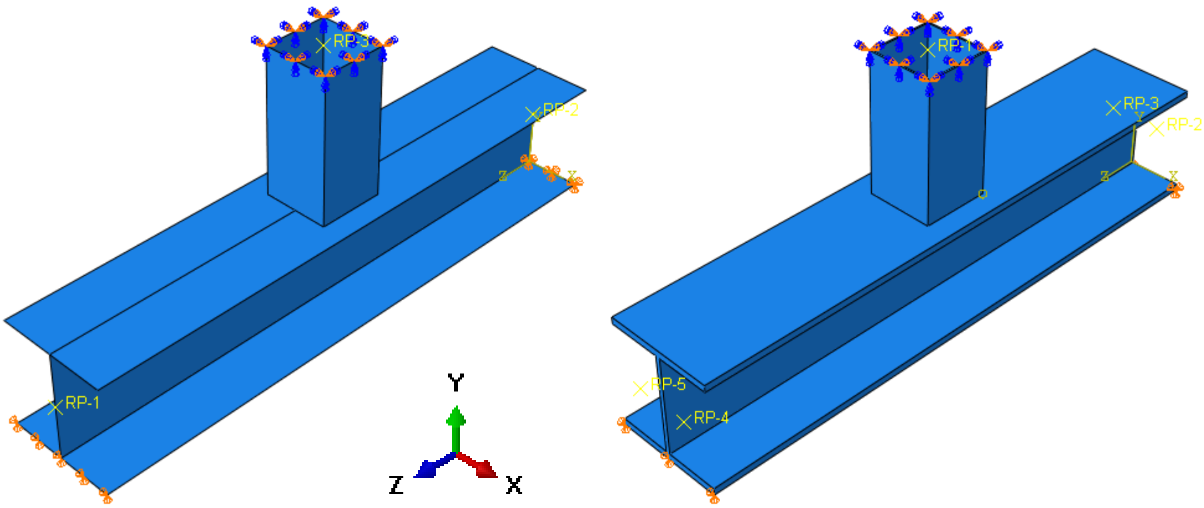


Figure 4.7 – Axis system and support conditions of the numerical modelling for shell and solid elements, respectively

Apart from the support stiffeners, there is also a loading plate at the top of the brace. The load plate was restricted to all the degrees of freedom except for the vertical translation in **y** axis (Chen & Wu, 2015).

The support stiffeners and the loading plate, were simulated as rigid bodies through the introduction of reference points which restrict the movement of the structural element components in its location to ensure a convenient numerical analysis. The convenient location for these reference points is in the centre of mass of the bodies (Chen & Wu, 2015).

The compression load is applied by a concentrated force at the reference point of the loading plate. The value of this force is irrelevant, since the importance is on the number of increments considered in the type of analysis, a matter already discussed previously.

The contact between the chord and the brace is established using a “master-slave” algorithm, available in ABAQUS library. This interaction between the two structural elements does not allow them to penetrate each other when subjected to compression forces.

4.2.6. Maximum deformation

The numerical modelling of Chen & Wu (2015) presents a peak-load for the joint under research , although, in cases where is not visible a peak-load, the authors report that is required to use a deformation limit equivalent to 3% of the width of the chord (b_0), which in this case is equal to 6 millimetres of maximum deformation.

This criteria is also referred by CIDECT, that recommend a deformation limit of 3% of the width of the chord (b_0) in cases where do not happen visually cracks or the ultimate load capacity (peak load) is not well defined and consequently have large deformations (Wardenier *et al*, 2010).

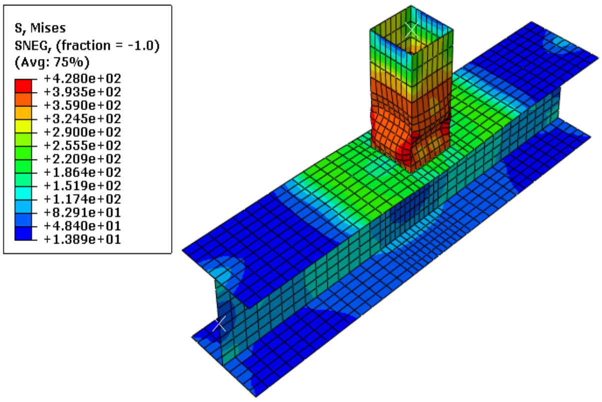
4.3. Analysis of the results

4.3.1. Validation

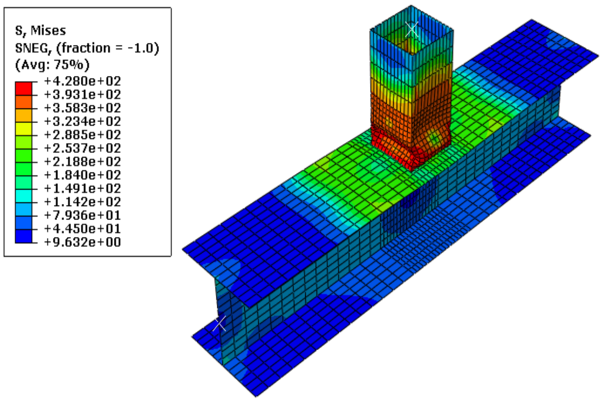
Based on all the information given in Chapter 4, the validation and verification of the numerical results can be done. The verification is made in the next chapter and it has to be pointed out that, in this process (Table 4.3), the resistance capacity based on analytical models, is obtained by the Equation (3.1) of this dissertation.

In Figure 4.8 and 4.9, depicts the joints modelled with shell and solid elements, respectively, and the equivalent von Mises stresses under compression.

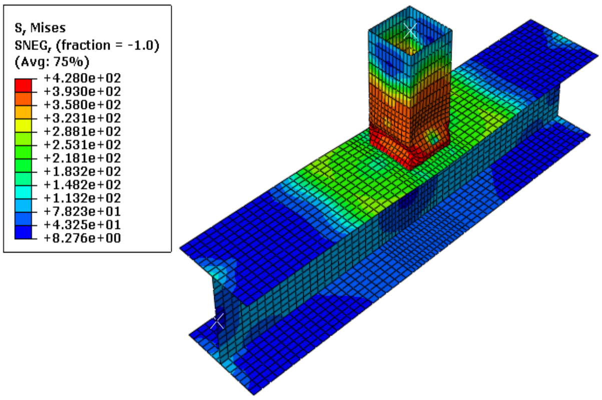
It can be seen in both figures the refinement of finite element meshes made.



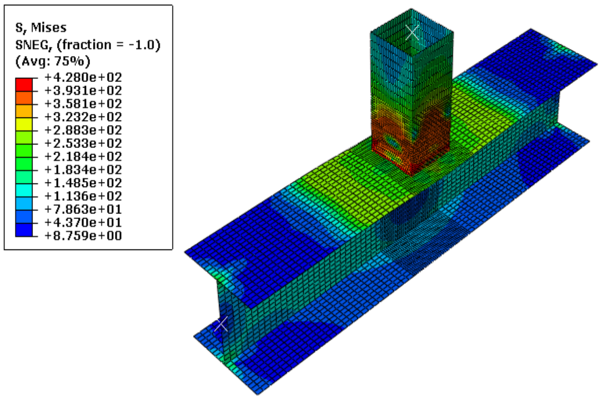
a) Mesh 30x30-15



b) Mesh 30x30-10

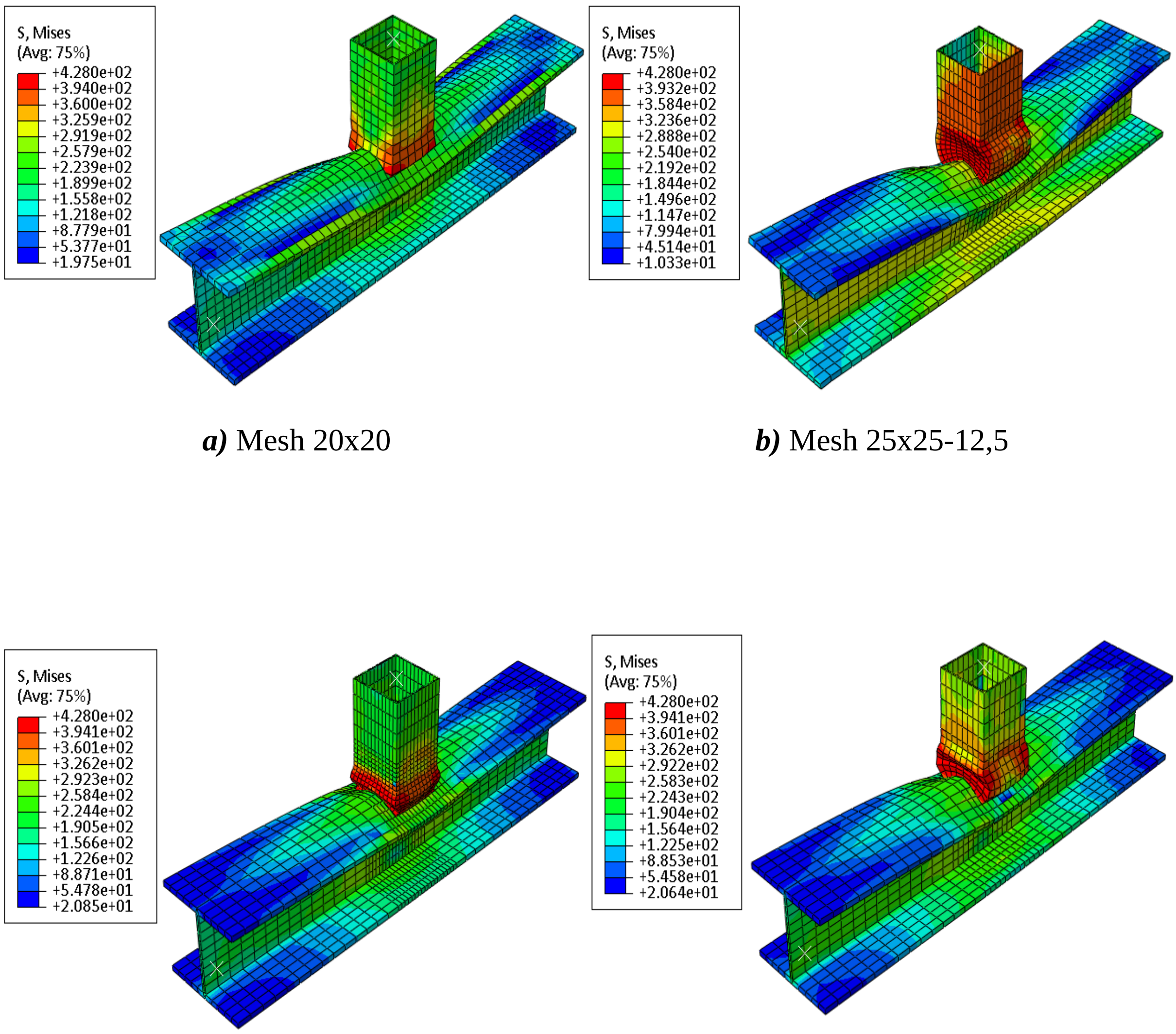


c) Mesh 20x20-10



d) Mesh 15x15-5

Figure 4.8 – Refinement of finite element meshes and respective von Mises stresses regarding shell elements: ***a)***, ***b)***, ***c)*** and ***d)***.



a) Mesh 20x20

b) Mesh 25x25-12,5

c) Mesh 30x30-10

d) Mesh 30x30-15

Figure 4.9 – Refinement of finite element meshes and respective von Mises stresses regarding solid elements: ***a)***, ***b)***, ***c)*** and ***d)***.

In Figure 4.10 and 4.11 are represented the Force-Displacement curves of the numerical models for shell and solid elements,respectively.

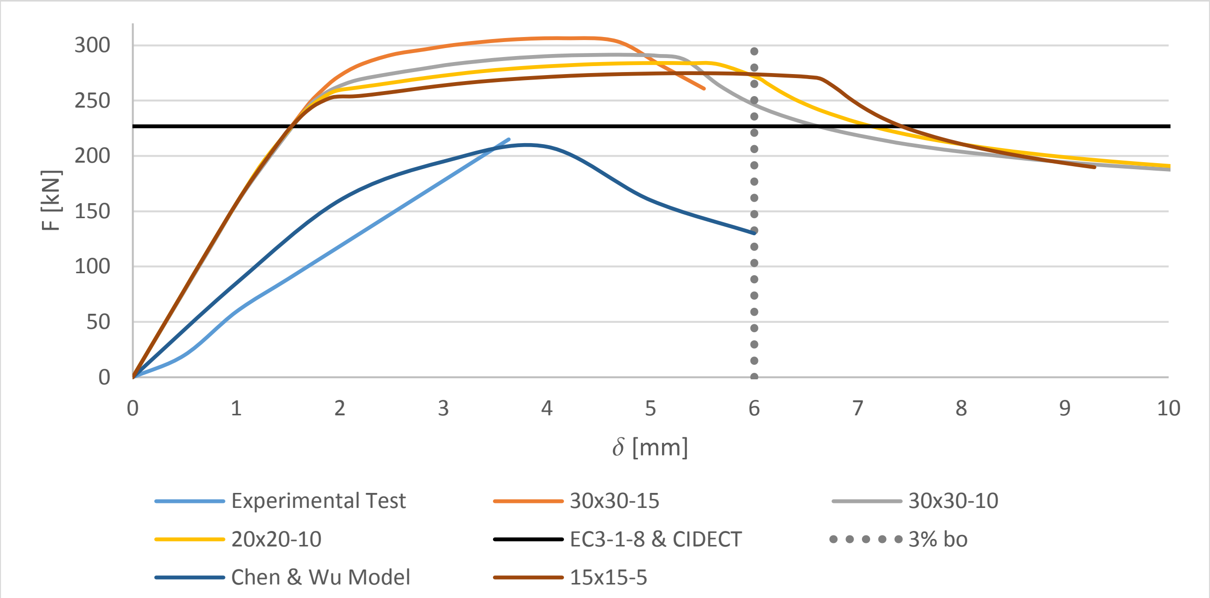


Figure 4.10 - Force-Displacement curves for shell elements with different refined meshes

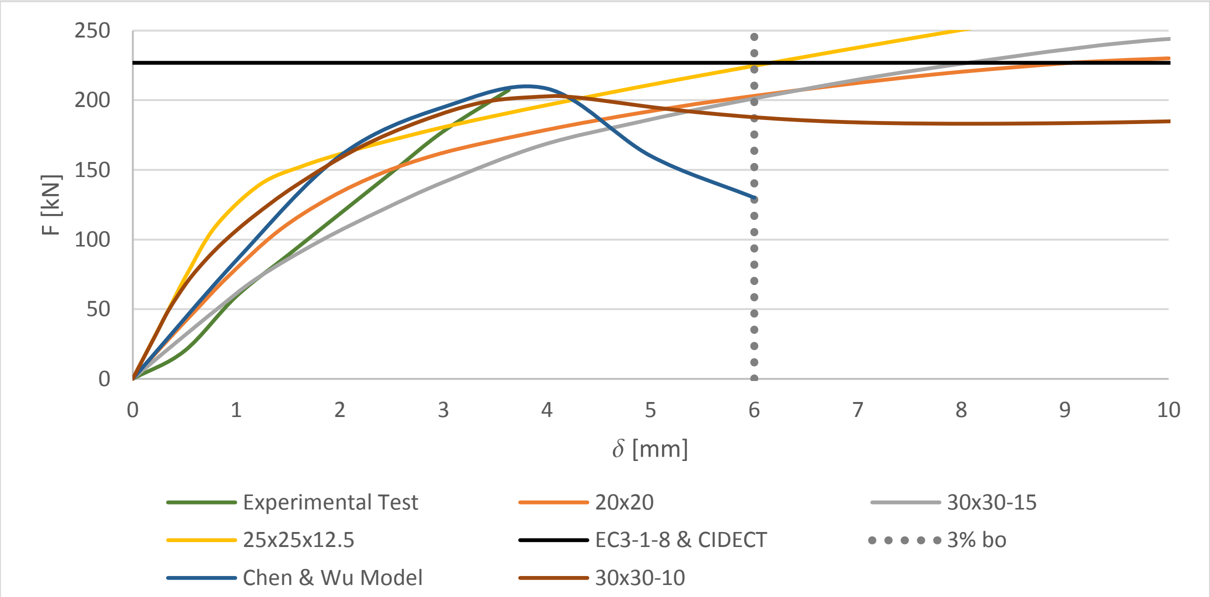


Figure 4.11 - Force-Displacement curves for solid elements with different refined meshes

As can be seen in Figures 4.10 and 4.11, the use of shell elements is not that which is closer to the experimental results.

The modelling with shell elements includes more discretized meshes comparing to those used with solid elements, but the meshes of the solid elements are composed by three-dimensional elements with 8 nodes, a fact that does not make it necessary to do such a wide mesh refinement as that done in shell elements.

Usually, shell elements give good approximations with experimental results, particularly in structures with large dimensions (one dimension significantly small, about 1/10 of the other dimensions), however, for this specific case, the solid elements gave the closest results against the experimental tests. Regarding the Chen & Wu research, the authors also performed the numerical study with solid elements.

This modelling with solid elements, apart from being the one closest to the experimental results, was also the one closer to the numerical modelling performed by Chen & Wu (2015), with the name of “Chen & Wu Model” in Figure 4.10 and 4.11.

In terms of validation, the mesh 30x30-10 was the mesh which showed the best numerical results, showing a peak-load nearby the peak-load of Chen & Wu Model and nearby the ultimate experimental load.

4.3.2. Verification

The experimental data which can be used for validation should be treated separately and in a different way comparing to the solutions applied for verification. The reasons for that are unavoidable errors and uncertainties associated with the result of experimental measurement. As the accurate solution is usually unknown (eventually for simplified cases) the user can only deal with estimates of errors. Uncertainty can be though as a parameter associated with the result of a measurement (solution) that characterizes the dispersion of the values and could reasonably be attributed to the measure (Wald *et al*, 2014).

The limitations of experimental validation increase the importance of verification which is supposed to deliver evidence that mathematical models are properly implemented and the numerical solution is correct with respect to the mathematical model (Wald *et al*, 2014).

The verification of the results is presented on Table 4.3, where the measurement errors between the ultimate loads of numerical solutions with analytical and experimental solutions are also evaluated.

Table 4.3 - Verifications of the numerical solutions with analytical and experimental solutions

Type of finit elements	Mesh	EC3-1-8; CIDECT	Experimental	Numerical	Numerical Error comparing to:	
			N_u (kN) Compression	N_u (kN) Compression	EC3-1-8; CIDECT	Experimental
Shell elements	**a)** 30x30-15	226,8	207,2	306,4	35,10%	47,88%
	b) 30x30-10			291,47	28,51%	40,67%
	c) 20x20-10			284	25,22%	37,07%
	d) 15x15-5			273,77	20,71%	32,13%
Solid elements	**a)** 20x20			202,89	-10,54%	-2,08%
	b) 25x25-12,5			224,46	-1,03%	8,33%
	c) 30x30-10			185,91	-18,03%	-10,28%
	d) 30x30-15			201,09	-11,34%	-2,95%

Analysing the obtained errors, it can be concluded that the solid elements represent again the best option, whether comparing to analytical or experimental results.

Considering the different meshes used for these elements, one that offers the closest results comparing to analytical results is the mesh ***b)*** 25x25-12,5 of Figure 4.9. The smaller numerical error comparing to experimental results is the one that belongs to the mesh ***a)*** 20x20 of Figure 4.9.

However, confronting the validation results, it can be seen that the mesh which is closer to the experimental results for solid elements is the mesh ***c)*** 30x30-10 of Figure 4.9, but as the model with this mesh is the one that shows a peak-load before the maximum deformation (3% of b_0), the load at this limit is smaller comparing to the peak-load, so, that's why the error is -10,28% or otherwise it would be smaller.

Looking to Figures 4.8 and 4.9, all the cases are consistent with each other in terms of failure modes, which occur by local yielding of the brace. By comparison with the Figures 4.12 and 4.13, it can be concluded that, there are reliability between the numerical failure modes and the experimental failure mode.

It should be noticed that, in Figure 4.12, in spite of being visible de deformation of the chord flanges, these components don't reach the plastic deformation, so, this means that this deformation is recoverable. The only components which reach the plastic deformation are the walls of the braces, a phenomenon that shows reliability comparing to the joint of Chen & Wu (2005).

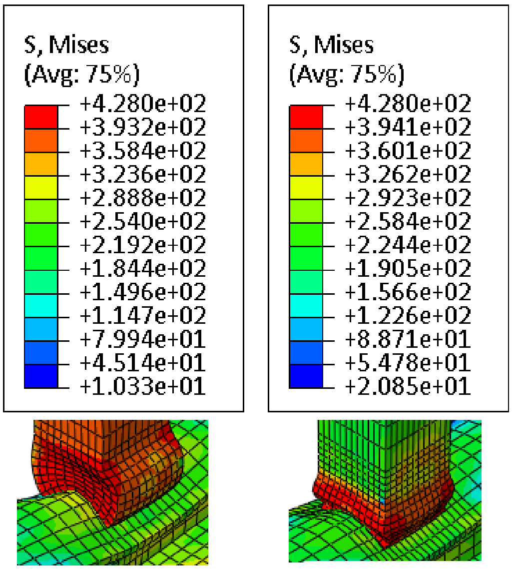


Figure 4.12 - Numerical failure modes of mesh 25x25-12,5 and 30x30-10 respectively, in solid elements

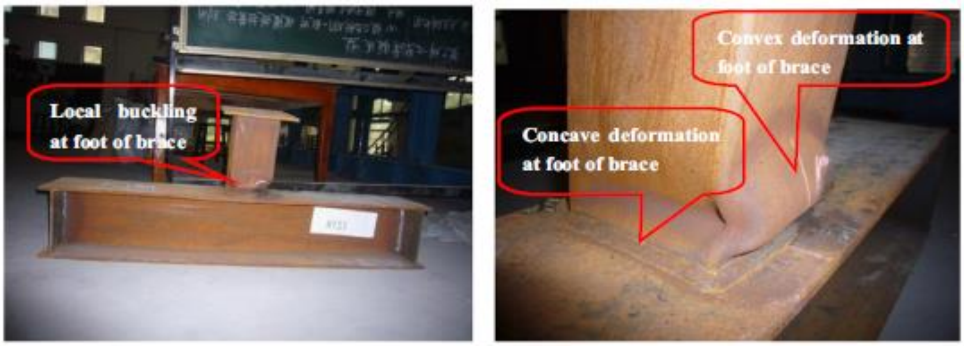


Figure 4.13 – Failure mode of the joint of Chen & Wu (Chen & Wu, 2015)

5. NUMERICAL ANALYSIS

5.1. Framework

Within the purpose of this dissertation, it was suggested by the Department of Steel and Timber Structures of the University of Prague, the numerical study of welded T-joints between hollow (brace) and open (chord) sections.

These joints were previously tested by them in their laboratories. The data provided about the joints were the geometry, type of loading, failure load, failure mode and steel resistance. As the Force-Displacement curves were not provided by them, the purpose of this chapter is not a validation, but a verification of the experimental results with analytical results.

To complement this analysis, it was also performed a numerical modelling of these joints in order to verify the similarity between the numerical failure modes and the analytical and experimental failure modes.

In the absence of data about the welds between the structural elements, the effect of them was neglected in the resistance of the joints, such as in the numerical model of Chen & Wu (2005). This can be explained by the welds resistance, which can be at least equal to the steel resistance of the structural members, but most of the times it is bigger. The failure modes due to the weld resistance are always by brittle failures, so, regarding the purpose of this dissertation, it is not intended to reach a brittle failure but a ductile behavior of the joint, which means despising the weld's effect is acceptable for this case.

However, even considering this, further investigations should be taken regarding the effect of the welds in the resistance of this type of joints. In Figure 5.1 is shown how the effective width of the chord web is taken into account in the resistance of T-joints with hollow (brace) and open (chord) sections. Looking to this figure, mainly to the right side of it, it's visible how the effect of the welds can increase the effective width of the chord web if they are taken into consideration in the analysis.

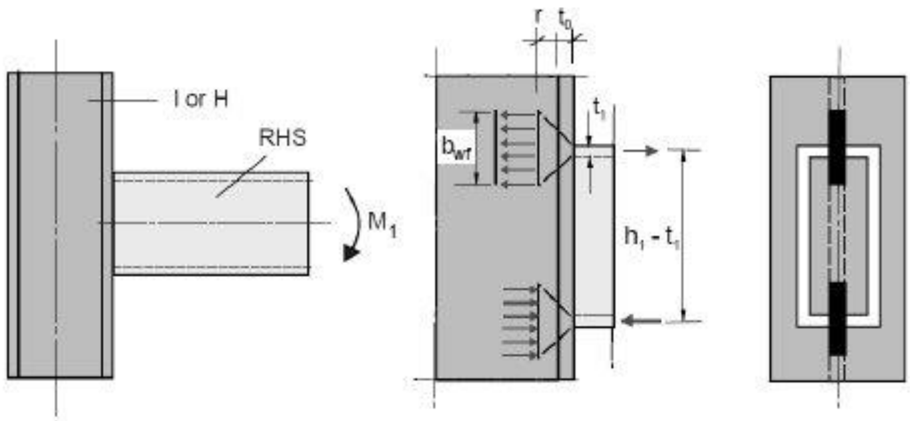


Figure 5.1 - Effective width of the chord (Wardenier *et al*, 2010)

In Table 5.1 the structural elements used in the different joints are presented. Numbered joints with the letter "***A***" are the joints tested under compression. On the other hand, joints numbered with the letter "***B***" are the joints tested under bending.

Table 5.1 - Summary of the analysed joints

	Joint	Chord	Brace
Compression	***A1***	HEA 140	RHS 150x100x12,5
	A2	HEA 140	RHS 150x100x5
	A3	HEA 140	RHS 140x80x4
	A4	IPE 270	RHS 150x100x12,5
Bending	***B1***	HEA 140	RHS 150x100x12,5
	B2	HEA 140	RHS 150x100x5
	B3	HEA 140	RHS 140x80x4
	B4	IPE 270	RHS 150x100x12,5

In Tables 5.2 and 5.3 are presented the geometrical properties of the structural elements of the joints. The parameters $\boldsymbol{L_0}$ and $\boldsymbol{L_1}$ represent the length of the chord and the brace respectively.

Table 5.2 - Geometrical properties of the structural elements which act as chords

Chord		
Properties	***HEA 140***	***IPE 270***
h_0 (mm)	133	270
b_0 (mm)	140	135
$t_{w,0}$ (mm)	5,5	6,6
$t_{f,0}$ (mm)	8,5	10,2
r (mm)	12	15
A_0 (cm^2)	31,42	45,95
A_{vz} (cm^2)	10,12	22,14
$W_{el,0}$ (cm^3)	155,4	428,9
$W_{pl,0}$ (cm^3)	173,5	484
d_w (mm)	92	219,6
L_0 (mm)	1050	1250

Table 5.3 - Geometrical properties of the structural elements which act as braces

Brace			
Properties	***RHS 150x100X12,5***	***RHS 150x100X5***	***RHS 140x80X4***
h_1(mm)	150	150	140
b_1 (mm)	100	100	80
t_1 (mm)	12,5	5	4
A_1(cm^2)	49,5	23,4	16,5
$W_{el,1}$(cm^3)	163	95,9	61,4
$W_{pl,1}$(cm^3)	220	117	75,5
r_1 (mm)	37,5	10	8
r_2 (mm)	25	5	4
L_1 (mm)	500	500	500

The numerical modelling of these joints is done based on the validation and verification study, done in Chapter 4.

Both non-linearity, material and geometrical, are implemented in numerical models. The material behavior is considered through a bilinear steel curve (Figure 4.4), with a yield stress ($\boldsymbol{\sigma_y}$) of 355 *MPa*, a ultimate stress ($\boldsymbol{\sigma_u}$) of 490 *MPa* and a ultimate strain ($\boldsymbol{\varepsilon_f}$) of 0,3, or in other words, of 30%. The Young Modulus ($\boldsymbol{E}$) is equal to 210 *GPa* and the Poisson's ratio ($\boldsymbol{\nu}$) has the value of 0,3.

As the purpose of this chapter is a verification of the obtained results, considering the results of Table 4.3, solid elements with the refined mesh 25x25-12,5 were used for this numerical study.

The stiffeners located at the support regions and the loading plate, were simulated through rigid bodies with reference points in its centres of mass.

In Figures 5.2 and 5.3 are presented the tests set up for the compression and bending tests respectively. The loads were always applied as concentrated forces as shown in the same figures with the letter "**F**".

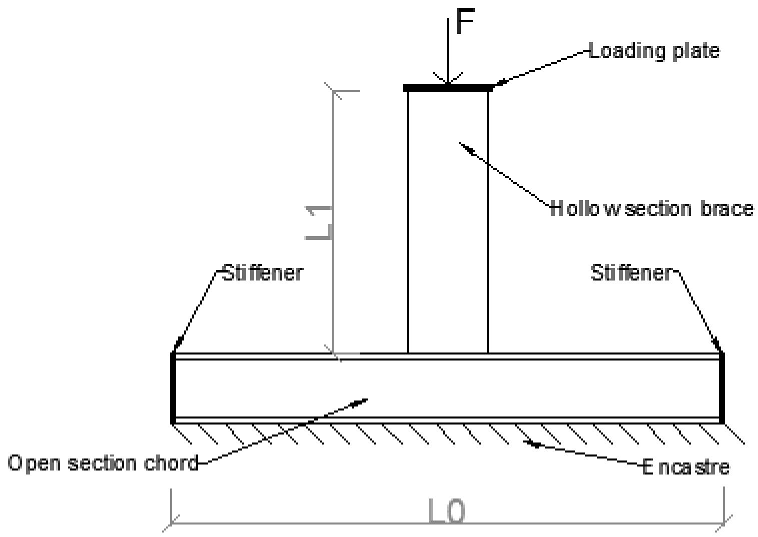


Figure 5.2 - Test set up for joints under compression

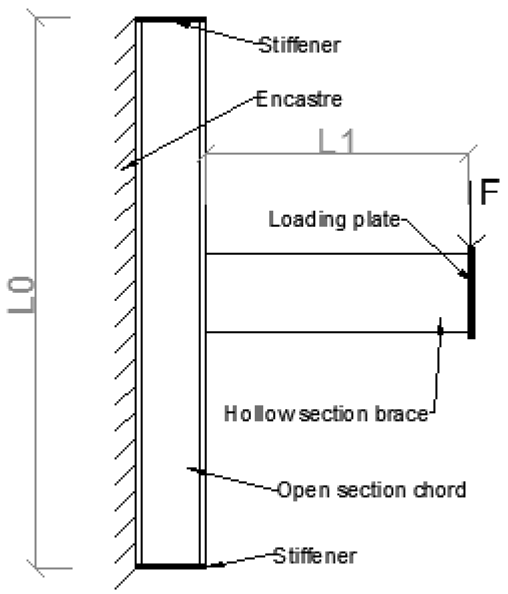


Figure 5.3 - Test set up for joints under bending

5.2. Analysis of the results

In this subchapter a comparison/relation between the resistance capacities obtained analytically and experimentally is made. It's aimed to establish a ratio between these results, which the closer they would be to the unit, more parity exists between them.

To compare the failure modes, the numerical model of these joints was performed. The numerical failure modes are presented in Figures 5.4 to 5.7. Comparing to the experimental tests, there are no differences regarding the failure modes, but comparing to the analytical failure modes, the joint ***B*2** has different solutions. For this joint, the analytical failure mode is by the chord web, in spite of the experimental or numerical failure mode which is by local failure of the brace. This comparison between the failure modes is also pointed out afterwards in Table 5.4.

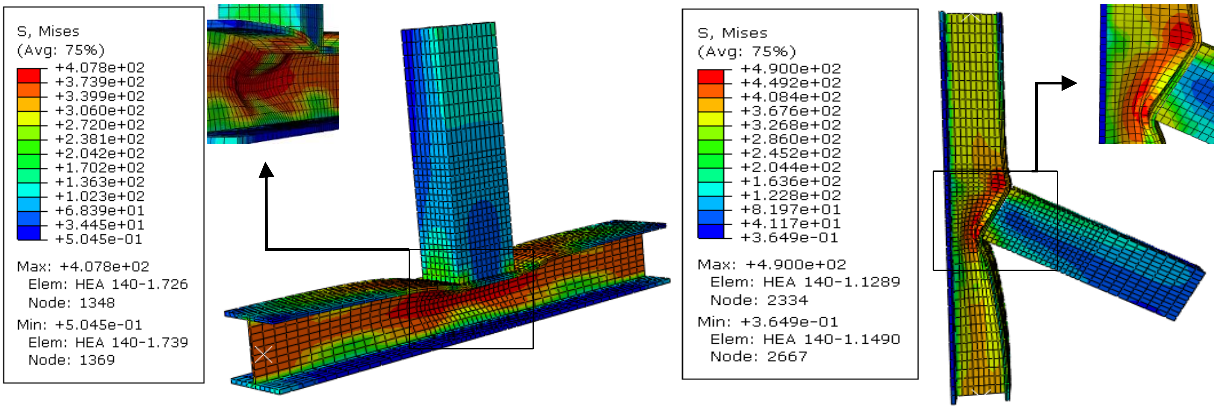


Figure 5.4 - Failure Mode of ***A*1 (left)** and ***B*1 (right)**

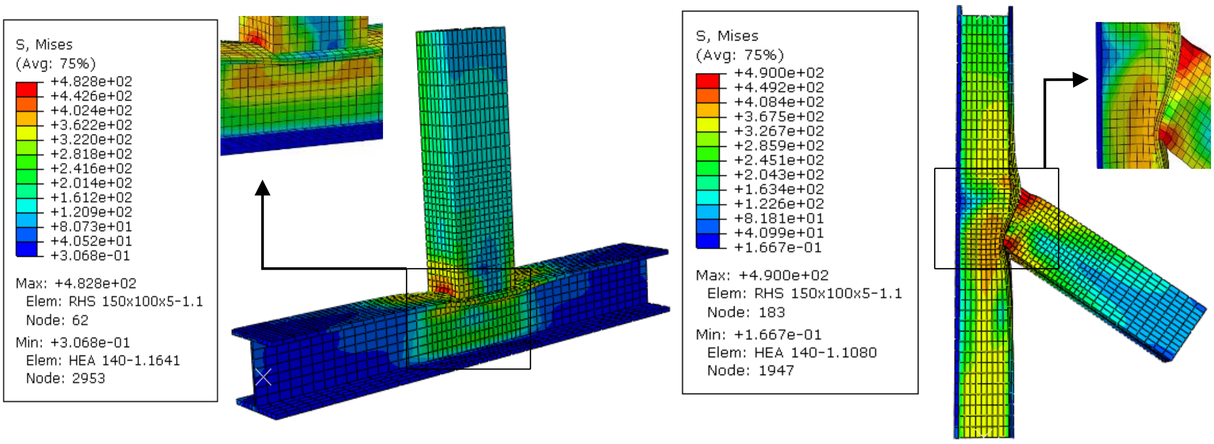


Figure 5.5 - Failure Mode of ***A*2 (left)** and ***B*2 (right)**

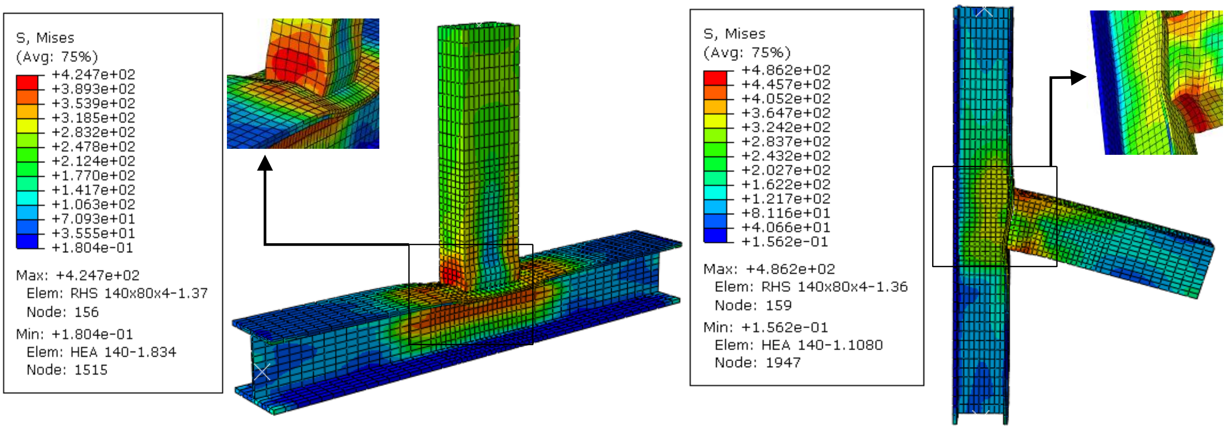


Figure 5.6 - Failure Mode of ***A*3 (left)** and ***B*3 (right)**

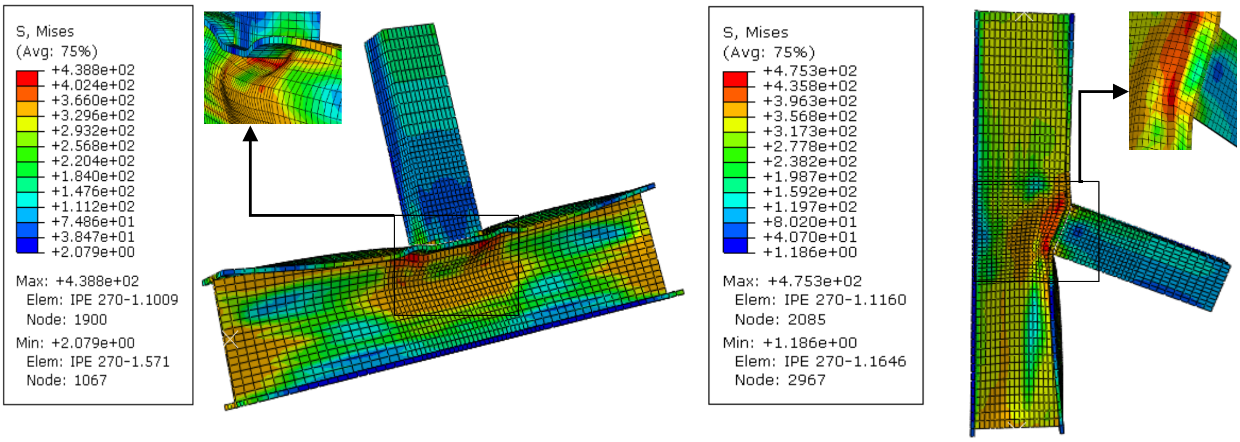


Figure 5.7 - Failure Modes of ***A4* (left)** and ***B4* (right)**

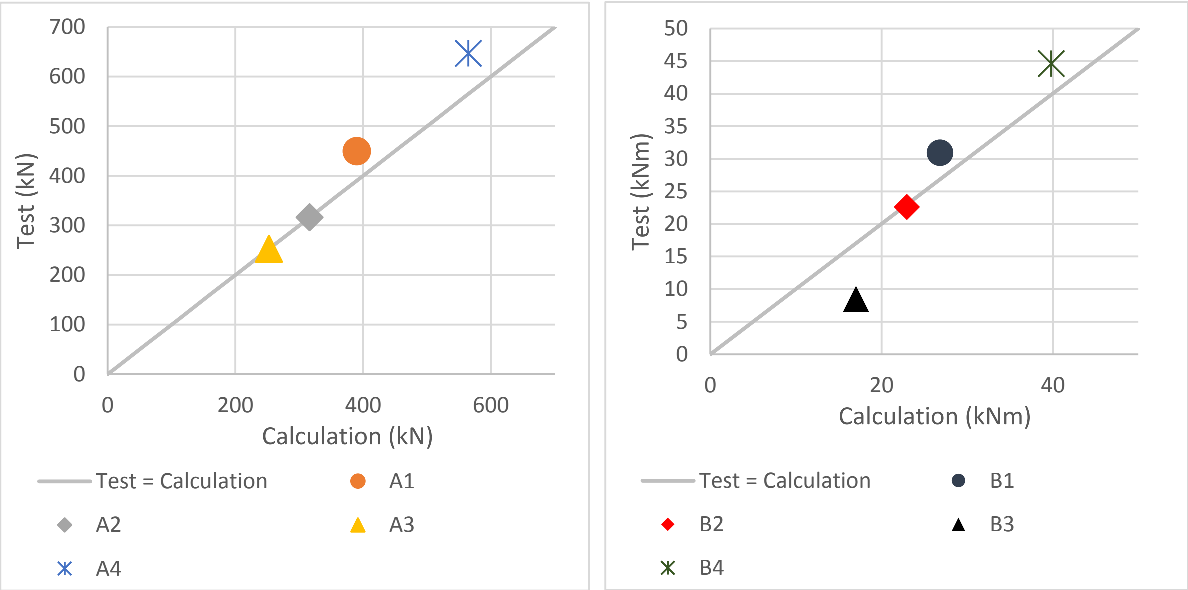
The comparison of the results can be observed in Table 5.4 and Figure 5.8.

In the common used standards documents is not exactly defined a limit for the deformation capacity of these type of joints when subjected to compression or bending. Also in the absence of information regarding the maximum deformation of the tested joints, it was applied the failure load of these tests in the numerical models. This is the reason why the numerical and experimental values in Table 5.4 have the same value.

Table 5.4 – Verification of the results considering the experimental tests of the Department of Steel and Timber Structures of Czech Technical University of Prague

	Joint	EC3-1-8 ; CIDECT		Experimental tests		Numerical		Ratio Calculation/ Experimental
Compression N_u(kN)	***A1***	390,50	CWF	449,10	CWF	449,10	CWF	0,87
	A2	315,95	LBF	316,00	LBF	316,00	LBF	1,00
	A3	252,76	LBF	252,80	LBF	252,80	LBF	1,00
	A4	564,66	CWF	646,70	CWF	646,70	CWF	0,87
Bending M_u(kNm)	***B1***	26,85	CWF	30,87	CWF	30,87	CWF	0,87
	B2	22,97	CWF	22,60	LBF	22,60	LBF	1,02
	B3	17,03	LBF	8,47	LBF	8,47	LBF	2,01
	B4	39,82	CWF	44,60	CWF	44,60	CWF	0,89

CWF – Chord Web Failure; LBF – Local Brace Failure



a) Test ***vs*** Calculation (Joints under compression)

b) Test ***vs*** Calculation (Joints under bending)

Figure 5.8 - Comparison between Test and Calculation results: **a)** and **b)**.

According to Wald *et al* (2014), the necessity for results verification is useful due to unavoidable errors and uncertainties associated with the result of experimental measurement. In a general manner, the ratios are relatively close to the unit, which means there's a good approach between the experimental tests and the analytical models.

Although, considering the ratio of the joints subjected to bending, there is one ratio that needs special attention (***B*3**). This ratio is far enough from the unit, which means that the experimental result is not closer to the analytical result as expected.

Thus, ***B*3**'s ratio can be thought like an error or uncertainty associated with the experimental measurement, or in other way, an uncertainty associated with the analytical formulas which are empirical and have lack of physical meaning. This phenomenon is one of those which can lead to the aim of this study, which is to get knowledge on the behavior of joints between hollow (brace) and open (chord) sections and try to give some physical meaning to these type of joints.

6. COMPONENT METHOD

6.1. Introduction

The component method, which origin is owed to Zoetemeijer in the year of 1974, is a simplified model produced throughout the rendering of a steel joint in its basic components which reproduce, beyond the joint geometry, the behavior of each part of the joint (resistance and deformation). This method allows to join to the traditional solutions, the verification of deformation compatibility, and consequently, the estimate stiffness of a joint. The components can be subjected to tension, compression or shear forces (Simões da Silva e Santiago, 2003).

The application of the component method has three stages:

- Listing the active components in a joint;
- Characterization of the behavior of each component;
- Assembly of all the components to get the global behavior of the joint based on the internal distribution of forces.

In Figure 6.1, can be observed a representative example of the background knowledge for the component method in a T-joint or, in other words, a beam to column joint between two open sections. In this figure is presented the path of the internal forces which establish the balance with the external forces of shear and bending. The spring distribution simulates the behavior of each component of the joint.

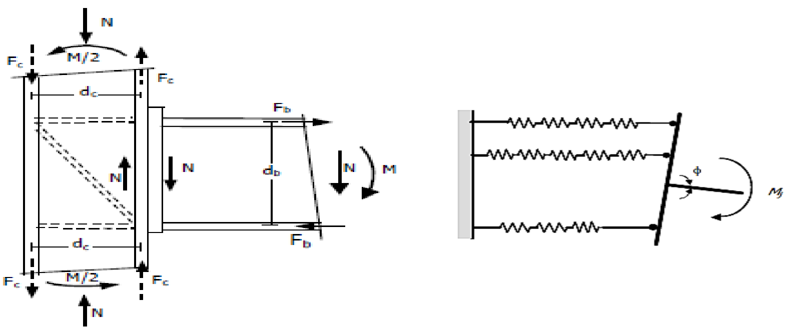


Figure 6.1 – Balance between internal and external forces and components simulated by springs (Simões da Silva e Santiago, 2003)

According to Jaspart *et al* (2005a), the assembly process of the various components intends to express that the forces acting upon all the joint are distributed in such a way that:

- The internal forces of the components are balanced with the external forces.
- The resistance of each component is not exceeded.
- The deformation capacity of each component is not exceeded.

However, the component method has some limitations because it was developed only for the design of joints between open sections. This has been a matter of investigations over the last decades with the aim to apply this method to all types of joints, apart from the type of elements which set up the joint.

Thereby, in this chapter is intended to study the possibility to apply the component method to welded joints with hollow (brace) and open (chord) sections.

6.2. Application of the component method to welded joints between open sections subjected to bending, according to EC3-1-8

6.2.1. Description of the welded T-joints between open sections

Call upon the numerical modelling of joints $\boldsymbol{B}\mathbf{1}$, $\boldsymbol{B}\mathbf{2}$, $\boldsymbol{B}\mathbf{3}$ and $\boldsymbol{B}\mathbf{4}$ studied in Chapter 5 of this dissertation, a numerical modelling of the same type of joints was done, but in this chapter, the braces were changed to open sections.

These open section braces were performed in a manner that their flanges would have the same geometrical properties of the RHS smaller walls (width $\boldsymbol{b_1}$) of joints of Chapter 5 (Figure 6.2). The decision of "removing" the bigger RHS walls (width $\boldsymbol{h_1}$) was based on the effective width criteria of the brace walls, explained on Subchapter 3.3.4. This allows to check the importance of RHS walls in the resistance of the joint and, consequently, see if the component method can be applied to joints with this sections, considering this criteria.

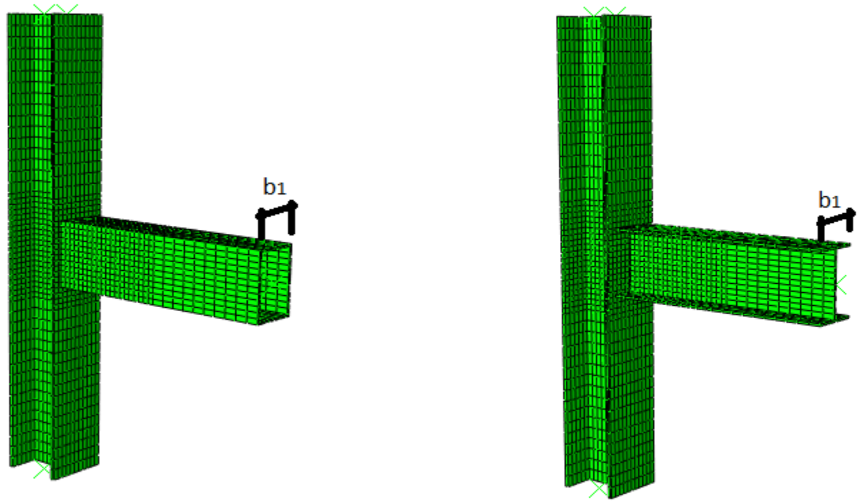


Figure 6.2 – Modification of an hollow section brace to an open section brace for behavior comparison

In Table 6.1 are presented the structural elements of the analysed joints in Chapter 5 (***B*1**, ***B*2**, ***B*3** and ***B*4**) and in this Chapter (***B*1_v2**, ***B*2_v2**, ***B*3_v2** and ***B*4_v2**).

In Figures 5.2 and 6.3 are presented the tests set up with their correspondent dimensions.

In Table 6.2 are presented the geometrical characteristics of the open section braces. The elements that act as chords and the hollow members that act as braces were already presented previously in Tables 5.2 and 5.3.

Table 6.1 - Summary of the analysed joints

	Joint	Chord	Brace
H or I Chord RHS Brace	***B1***	HEA 140	RHS 150x100x12,5
	B2	HEA 140	RHS 150x100x5
	B3	HEA 140	RHS 140x80x4
	B4	IPE 270	RHS 150x100x12,5
H or I Chord I Brace	***B1*_ v2**	HEA 140	I 150x100x12,5x5,5
	***B2*_ v2**	HEA 140	I 150x100x5x4
	***B3*_ v2**	HEA 140	I 140x80x4x4
	***B4*_ v2**	IPE 270	I 150x100x12,5X5,5

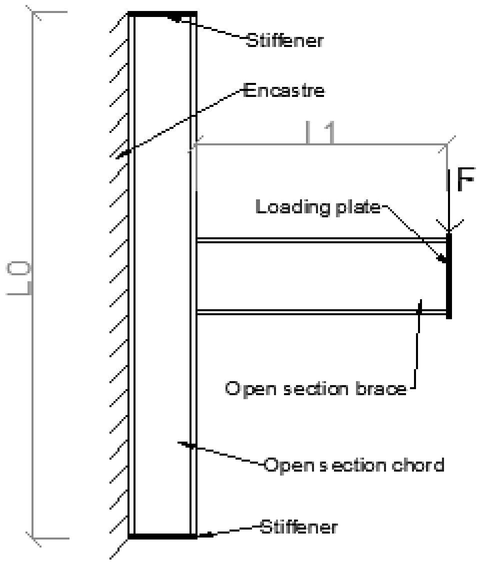


Figure 6.3 – Numerical test set up for joints between open sections

Table 6.2 - Geometrical properties of the open sections braces

	Brace		
Properties	***I 150x100x12,5x5,5***	***I 150x100x5x4***	***I 140x80x4x4***
h_1 (mm)	152	150	140
b_1 (mm)	100	100	80
$t_{1,f}$ (mm)	12,5	5	4
$t_{1,w}$ (mm)	5,5	4	4
A_1 (cm^2)	49,5	23,4	16,5
L_1 (mm)	500	500	500

6.2.2. Application of the component method

According to EC3-1-8, beam to column joints (T-joints) can be represented by a rotational spring joining the axes of the connected elements at an intersection point, as shown in Figure 6.4 (CEN, 2010a).

The spring properties can be expressed by a design relation Moment-Rotation, which represents the relation between the bending moment and the respective rotation between the two connected elements. The obtained curve with this relation should allow the user to define the three structural properties of a joint: Bending capacity, Rotational stiffness and Rotational capacity (CEN, 2010a).

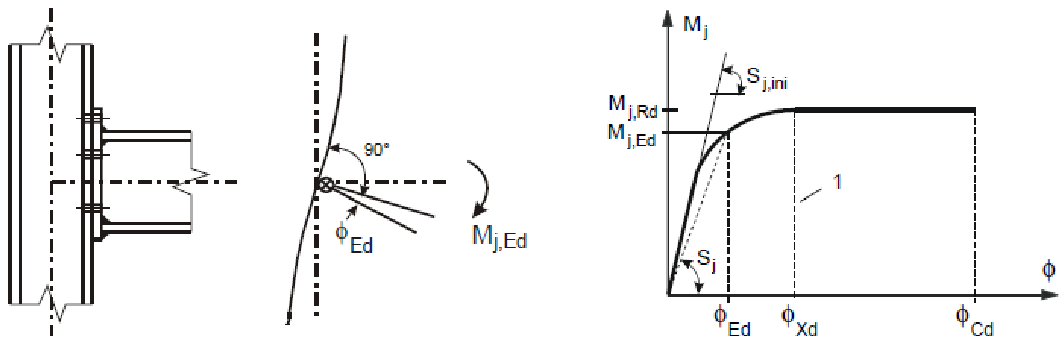


Figure 6.4 – Characteristic Moment-Rotation behavior of a joint (CEN, 2010*b*)

Soon afterwards is explained in a briefly manner, the application of the component method for welded joints between two open sections, according to EC 3-1-8.

The EC 3-1-8 gives application rules for the design of the resistance, stiffness factor and rotational capacity for each component of the referred joints (Figure 6.5).

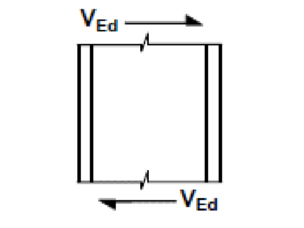
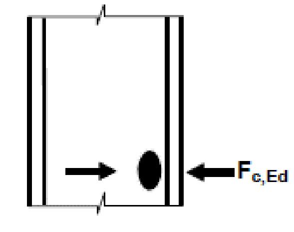
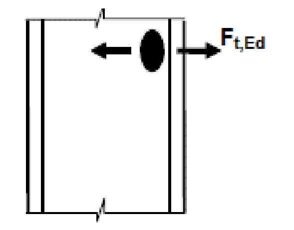
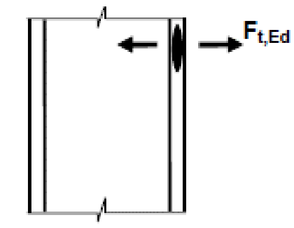
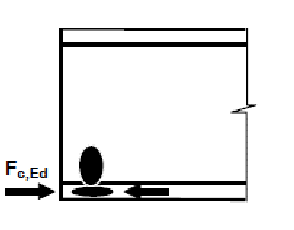
Column web subjected to shear	
Column web subjected to transversal compression	
Column web subjected to transversal tension	
Column flange subjected to bending	
Beam web and flange subjected to compression	

Figure 6.5 - Basic components of welded beam to column joints between open sections (CEN, 2010*b*)

In spite of not considering the welds in the Figure 6.5 because, as already explained, their effect on the resistance is admitted as negligible, it has to be referred that the Eurocode also gives recommendations to know the behavior of this component, in terms of resistance and stiffness capacity.

Relating to the stiffness factors to be considered for welded joints with beam in one side, it just needs to take into consideration the factors k_1, k_2 and k_3, referred to the respective components:

chord web subjected to shear, chord web subjected to transversal compression and chord web subjected to tension.

Hereafter, it is explained how the resistance and stiffness is taken into account in each component.

- **<u>Chord web subjected to shear forces</u>**

Resistance

The design methods of this component are acceptable if the column web slenderness respects the Equation (6.1).

$$\frac{d_c}{tw} \leq 69\varepsilon \tag{6.1}$$

The design value of the plastic resistance to shear forces of a column web not reinforced ($V_{wp,Rd}$), liable to an acting value ($V_{wp,Ed}$), should be obtained by the equations (6.2) and (6.3), respectively.

In Figure 6.6 are presented the shear forces acting upon the column web borders.

$$V_{wp,Rd} = \frac{0{,}9\sigma_{y,wc}A_{vc}}{\sqrt{3}\gamma_{M0}} \tag{6.2}$$

$$V_{wp,Ed} = \frac{(M_{b1,Ed}-M_{b2,Ed})}{z} - \frac{(V_{c1,Ed}-V_{c2,Ed})}{2} \tag{6.3}$$

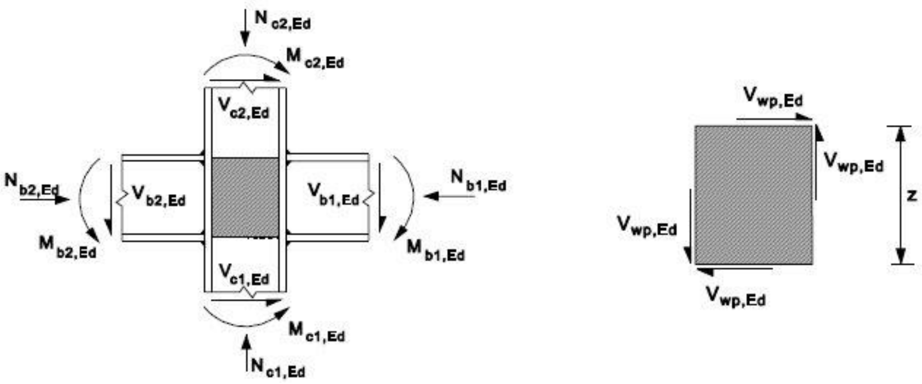


Figure 6.6 - Shear forces acting on column web borders (CEN, 2010*b*)

Stiffness

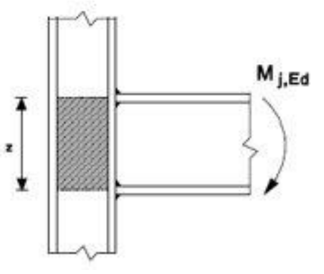
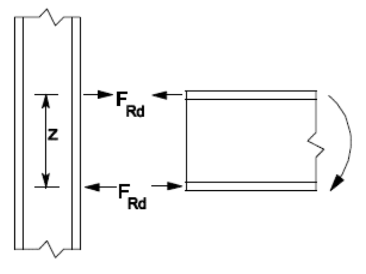
The stiffness factor of this component is given by the Equation (6.4).

In Figure 6.7 is presented the information related to the compression centre, torque arm and forces distribution.

$$k_1 = \frac{0{,}38 A_{vc}}{\beta z} \tag{6.4}$$

It should be noted that, for design purposes, the parameter β considers the relation between the moments at the right and left side of the column. As the present case only has one beam in one of the sides, this parameter has a value equal to the unit (CEN, 2010a).

Figure 6.7 - Compression centre, torque arm and distribution of forces (CEN, 2010*b*)

Welded joint	Compression centre	Torque arm	Distribution of forces
	Aligned with the medium thickness of the beam flange	$z = h - t_{fb}$	
		h height of the connected beam	
		t_{fb} thickness of the beam flange	

➢ **Column web subjected to transversal compression**

Resistance

The design value of the resistance of the web considering that it's not reinforced, when subjected by a transversal compression should be determined by the Equation (6.5).

$$F_{c,wc,Rd} = \frac{w k_{wc} b_{eff,c,wc} t_{wc} \sigma_{y,wc}}{\gamma_{M0}} \text{ but } F_{c,wc,Rd} \leq \frac{w k_{wc} \rho b_{eff,c,wc} t_{wc} \sigma_{y,wc}}{\gamma_{M1}} \tag{6.5}$$

w is the reduction factor to take into account eventual integration effects with the shear forces in the column web, which for $\beta = 1$, has the next equation:

$$w = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc} t_{wc}/A_{vc})^2}}$$

k_{wc} is the reduction factor, usually considered equal to 1. It can, therefore, be omitted in preliminary calculations.

$b_{eff,c,wc}$ is the effective width of the compressed column web.

$$b_{eff,c,wc} = t_{fb} + 2\sqrt{2}a_b + 5(t_{fc} + s)$$

The parameter a_b refers to the thickness of the welds. In this dissertation, the effect of the welds was neglected in the numerical analysis done previously. So, to achieve a reliable compatibility of the results in the comparison to be done afterwards, this parameter was also neglected in this analytical analysis.

s for a rolled I or H section column: $s = r_c$

ρ is the reduction factor for plate buckling:

if $\overline{\lambda_p} \leq 0{,}72 :\ \rho = 1$;

if $\overline{\lambda_p} > 0{,}72 :\ \rho = (\overline{\lambda_p} - 0{,}2)/\overline{\lambda_p}^2$;

$\overline{\lambda_p}$ is the plate slenderness:

$$\overline{\lambda_p} = 0{,}932\sqrt{\frac{b_{eff,c,wc}d_{wc}\sigma_{y,wc}}{Et_{wc}^2}}$$

$$d_{wc} = h_c - 2(t_{fc} + r_c)$$

Stiffness

The stiffness factor of this component is given by Equation (6.6).

$$k_2 = \frac{0{,}7b_{eff,c,wc}t_{wc}}{d_{wc}} \tag{6.6}$$

- **Column web subjected to transversal tension**

Resistance

The design value of the resistance of the not reinforced column web, when subjected to transversal tension, should be determined regarding the Equation (6.7).

$$F_{t,wc,Rd} = \frac{wb_{eff,t,wc}t_{wc}\sigma_{y,wc}}{\gamma_{M0}} \tag{6.7}$$

Stiffeness

The stiffness factor of this component is given by the Equation (6.8).

$$k_3 = \frac{0{,}7 b_{eff,c,wc} t_{wc}}{d_{wc}} \tag{6.8}$$

- **Column flange subjected to bending**

Resistance

In a welded joint, the design value of the resistance of the not reinforced column flange, when subjected to bending due to tension or compression forces acting in the beam flanges, should be determined by the Equation (6.9).

$$F_{fc,Rd} = \frac{b_{eff,fc} t_{fb} \sigma_{y,fb}}{\gamma_{M0}} \tag{6.9}$$

$b_{eff,fc} = t_w + 2s + 7kt_f$

$k = \left(\frac{t_f}{t_p}\right)\left(\frac{\sigma_{y,f}}{\sigma_{y,p}}\right)$; $k \leq 1$. In Figure 6.8 are characterized the parameters t_f, t_p and $b_{eff,fc}$.

$\sigma_{y,f}$ is the yield stress of the flange of the I or H section.

$\sigma_{y,p}$ is the yield stress of the plate welded to the I or H section. For this case, this parameter is equal to the yield stress of the column web.

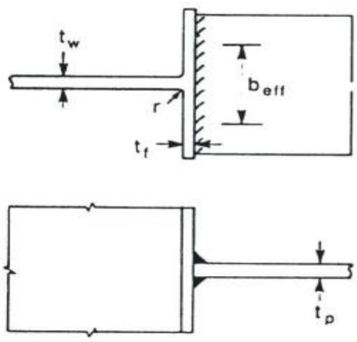


Figure 6.8 - *tf*, *tp* and *beff,fc* parameters for not reinforced T-joints (CEN,2010*b*)

- **<u>Flange and web of the beam subjected to compression forces</u>**

<u>Resistance</u>

The design value of the resistance of the flange and web of the beam when subjected to compression, is determined by the Equation (6.10).

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{(h-t_{fb})} \tag{6.10}$$

h is the height of the beam

$M_{c,Rd}$ is the design value of the bending capacity of the cross section of the beam, reduced if required to take into account the shear forces acting upon the beam. This value is calculated considering the recommendations of EN 1993-1-1 (CEN, 2010*a*).

$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl}\sigma_y}{\gamma_{M0}}$; for sections of Class 1 or 2

$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min}\sigma_y}{\gamma_{M0}}$; for sections of Class 3

If the shear forces acting upon the beam are less than half of the plastic resistant capacity to shear, its effect under $M_{c,Rd}$ can be despised. Otherwise, $M_{c,Rd}$ should be considered with the next equations:

$$M_{c,Rd} = \left(\frac{\left[W_{pl,y} - \frac{\rho A_W}{4t_W}\right]\sigma_y}{\gamma_{M0}}\right)$$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

6.2.3. Behavior of the T-joints between open sections

Regarding the exposed information, it’s possible to verify the weakest component of each joint considering the one with the minor resistant capacity. The bending capacity of the joints may be obtained by multiplying the value of this resistance by the torque arm (***z***) (Figure 6.7), expressed by the Equation (6.11)

To obtain the global stiffness of each joint, the assembly of the stiffness of each component is done, for welded beam to column joints (k_1, k_2, k_3) based on Equation (6.12).

$$M_{j,Rd} = zF_{Rd} \tag{6.11}$$

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} \tag{6.12}$$

In Table 6.3 is presented the summary of the results in terms of resistance and stiffness of the welded T-joints between open sections, studied in this Chapter.

Table 6.3 - Resistance capacity and Stiffness of the joints, designed according to EC3-1-8

	EC 3-1-8 - Component method				
	Joints	Resistance capacity			Stiffness
		F_{Rd} (kN)	$M_{j,Rd}$ (kNm)	Weakest component	S_j (kN/m)
H or I Chord I Brace	***B1_* v2**	182,86	25,14	CWTC	4503,79
	***B2_* v2**	157,98	21,72	CFB	4709,05
	***B3_* v2**	126,38	17,38	CFB	4224,35
	***B4_* v2**	293,60	40,37	CWTC	2727,36

F_{Rd} – Resistance capacity of the weakest component of each joint (Figure 6.7); $M_{j,Rd}$ – Bending capacity (Figure 6.7 and Equation 6.11); S_j – Stiffness of the joints (Equation 6.12); CWTC – Column web subjected to transversal tension or transversal compression; CFB – Column web subjected to bending.

As the material and test set up dimensions are the same of the analysed joints in Chapter 5 subjected to bending, a brief comparison of the analytical results is made to check the difference between the resistance of the two type of joints.

The bending capacity of these joints (H or I Chord and RHS brace) is once more presented in Table 6.4. In this table, the component of the joint where the failure occurred, was called the weakest component because, in contrast to the component method for joints with open section (Table 6.3), the resistance capacity is not obtained for each component but for the whole joint through empirical formulas. These formulas are those expressed in Equations (3.8) and (3.9) of this dissertation.

Table 6.4 - Bending capacity of the analysed joints in Chapter 5 subjected to bending

	EC 3-1-8 and CIDECT - Empirical formulas		
	Joints	Resistance capacity	
		$M_{j,Rd}$ (kNm)	Weakest component
H or I Chord RHS Brace	***B1***	26,85	CWF
	B2	22,97	CWF
	B3	17,03	LBF
	B4	39,82	CWF

The weakest component symbols are defined in Table 5.4.

In Figure 6.9 is presented the referred analytical comparison between the results of the bending capacity of the joints of Tables 6.3 and 6.4, both of them designed according to EC 3-1-8.

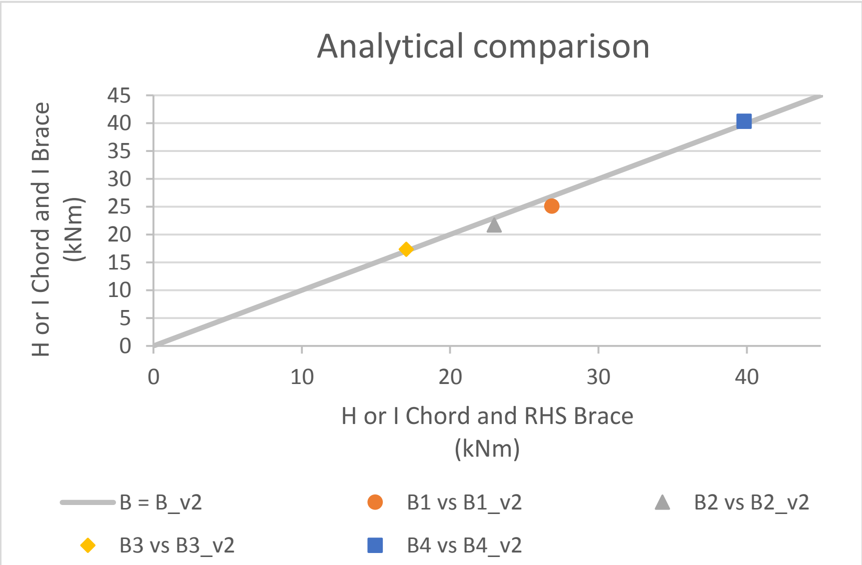


Figure 6.9 - Analytical comparison between the joints of Chapter 5 and Chapter 6

Regarding the Figure 6.9, comes up the conclusion that the possibility to apply the component method, foremost performed for joints between open sections, is viable for joints between hollow (brace) and open (chord) sections. It can also be concluded that the wider walls of the RHS braces are not so relevant for the joint resistance.

6.3. Numerical analysis

6.3.1. T-joints between open sections *vs* T-joints between hollow (brace) and open (chord) sections

In a rough way, the component method is frequently represented throughout the finite element method (Jaspart *et al*, 2000). The finite element method, applied with the appropriate numerical tools is, for some joints, less labour intensive and has a bigger range of application comparing to the component method (Sabatka *et al*, 2014).

In this Subchapter is made a comparison of the numerical curves Moment-Rotation of the joints of Chapter 5 (***B*1**, ***B*2**, ***B*3** and ***B*4**) with the joints of Chapter 6 (***B*1_v2**, ***B*2_v2**, ***B*3_v2** and ***B*4_v2**).

The numerical modelling of the T-joints between open sections (***B*1_v2**, ***B*2_v2**, ***B*3_v2** and ***B*4_v2**) it's completely identical to the numerical modelling done for T-joints between hollow and open sections (***B*1**, ***B*2**, ***B*3** and ***B*4**), in terms of dimensions, type of analysis, finite elements, mesh, boundary conditions, support conditions and loading. The only thing that changes is the type of elements that act as braces. The properties of the modelled joints can be observed in Tables 5.2, 5.3, 6.1 and 6.2.

From Figure 6.10 to 6.13 are shown the obtained failure modes for the welded T joints between open sections.

Comparing these numerical failure modes with the analytical failure modes of Table 6.3, the finite element (numerical) model reveals to be a reliable source for benchmark studies, when the lack of experimental tests in this area is utmost. So, the numerical and analytical failure modes are consistent with each other.

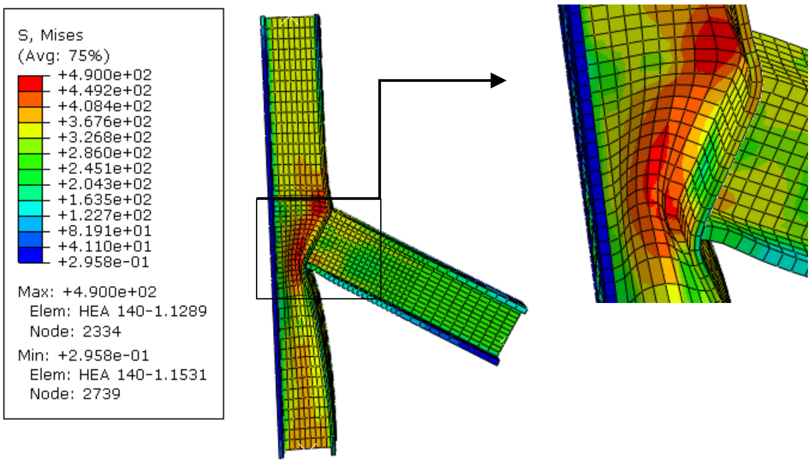


Figure 6.10 – Chord web failure of joint ***B*1_v2**

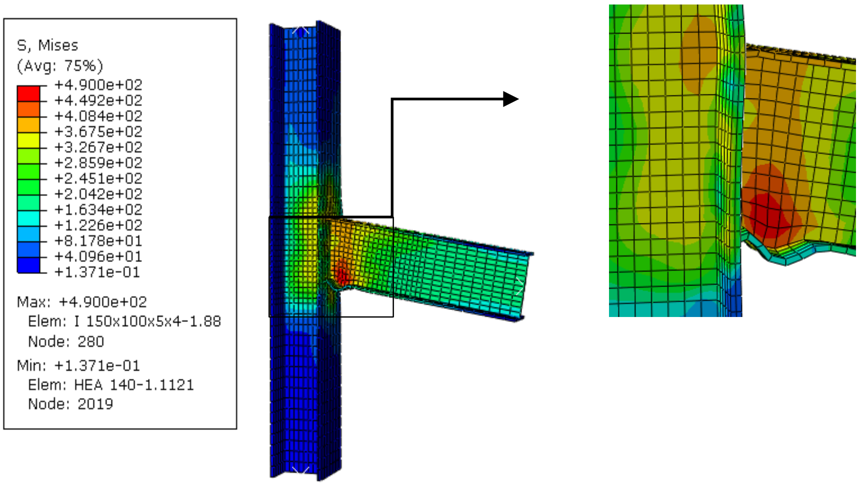


Figure 6.11 – Local brace failure ***B*2_v2**

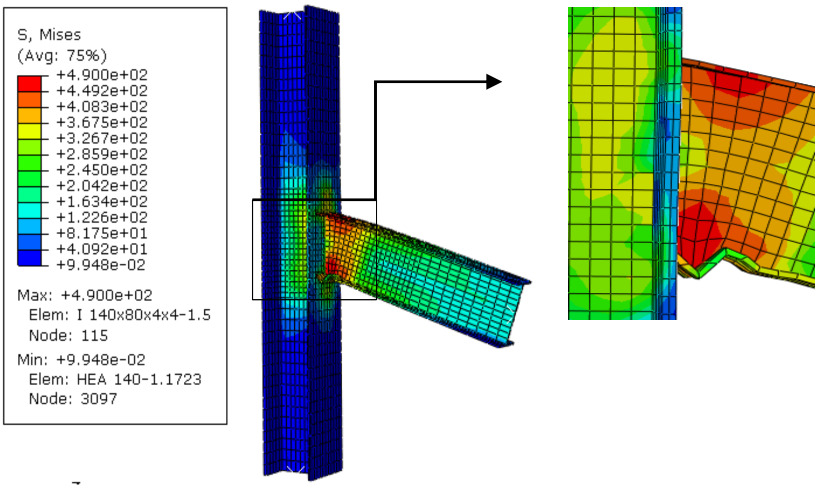


Figure 6.12- Local brace failure ***B*3_v2**

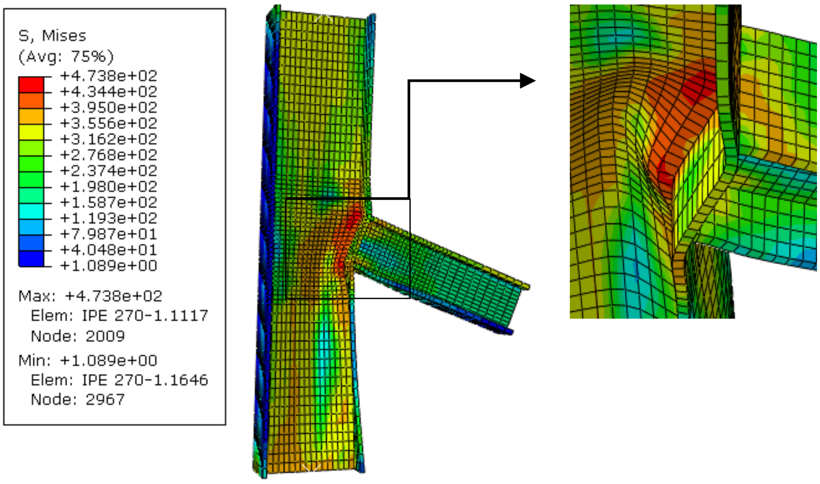


Figure 6.13 – Chord web failure ***B*4_v2**

Having no information about the rotation limit of the joints tested by the Department of Steel and Timber Structures of University of Prague (***B*1**, ***B*2**, ***B*3** and ***B*4**), the resistance capacity is ascertain to a rotation limit equivalent to 0,015 rads, according to EC 3-1-8 for not reinforced welded beam to column joints.

The Figures 6.14 to 6.17 show the referred numerical comparison of the Moment-Rotation curves.

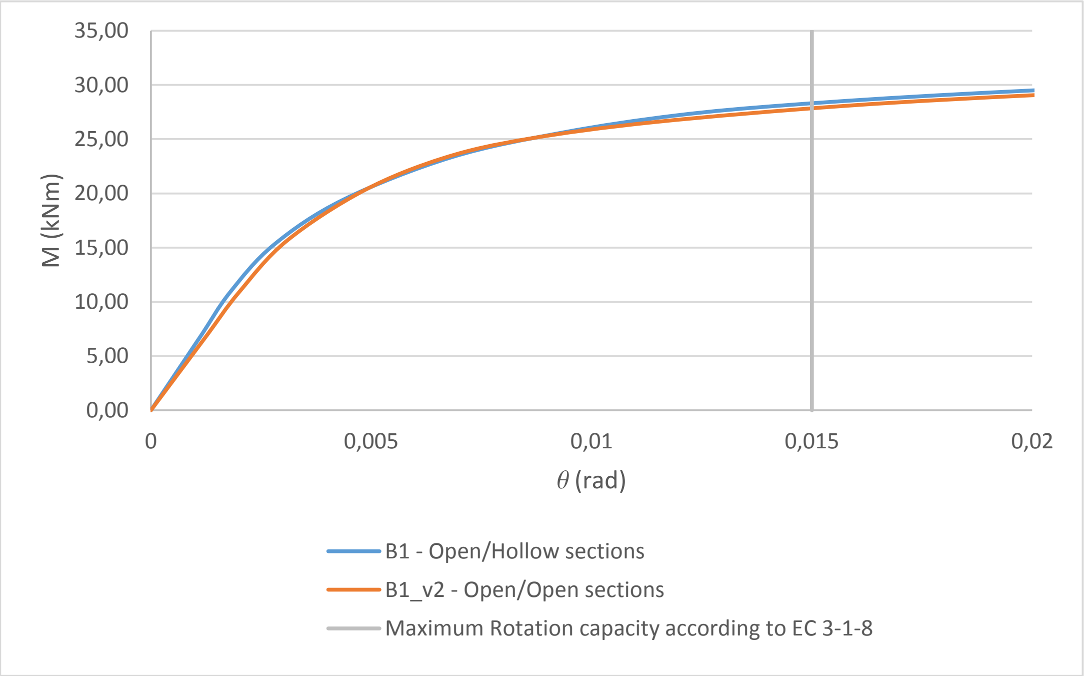


Figure 6.14 - Moment-Rotation curves, ***B*1** *vs* ***B*1_v2**

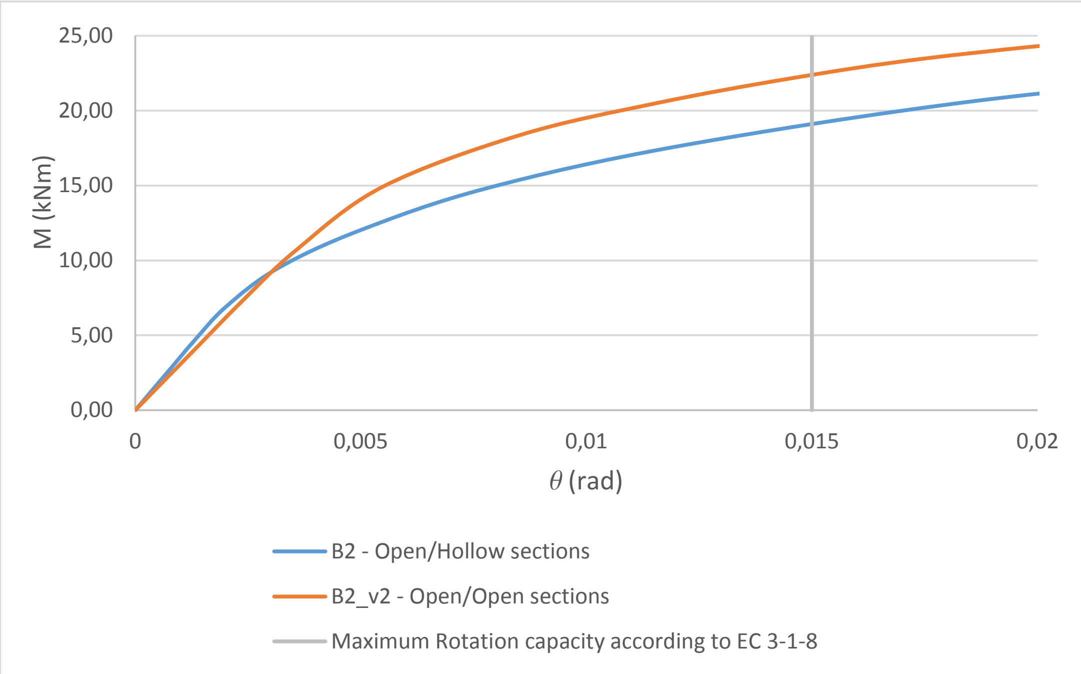


Figure 6.15 - Moment-Rotation curves, ***B2*** *vs* ***B2_v2***

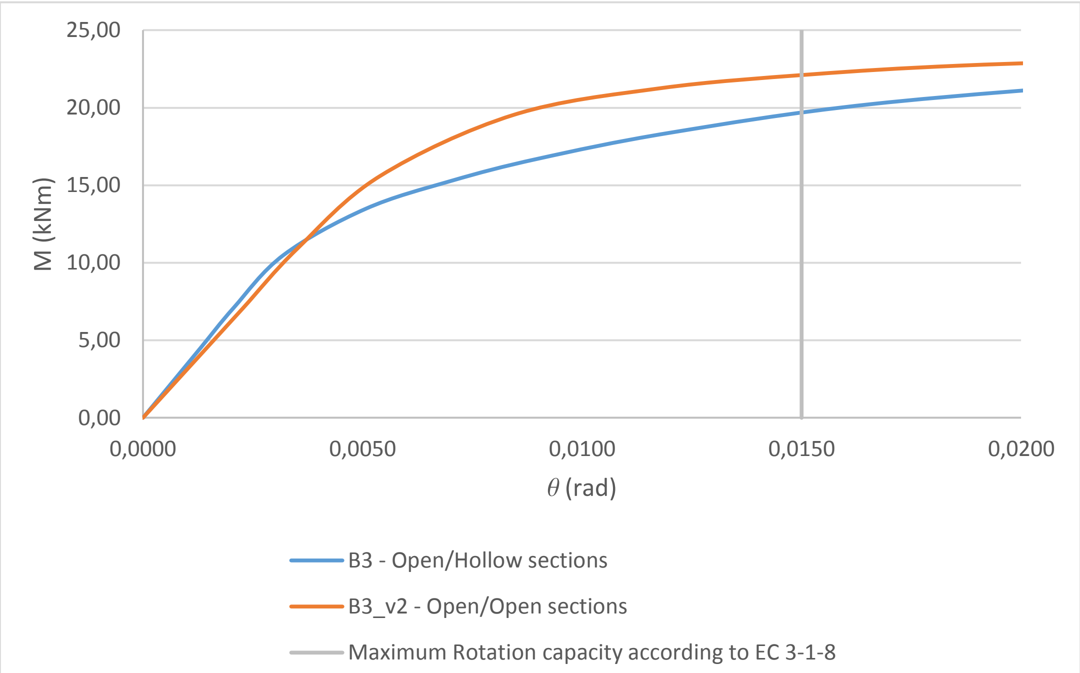


Figure 6.16 - Moment-Rotation curves, ***B3*** *vs* ***B3_v2***

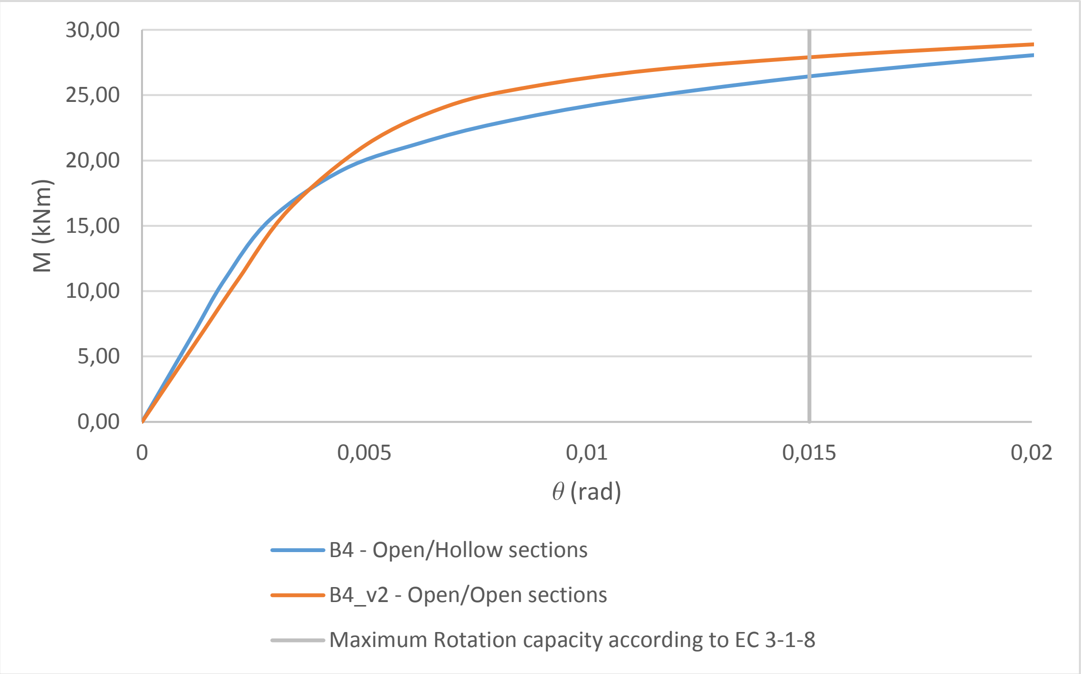


Figure 6.17 - Moment-Rotation curves, ***B4*** *vs* ***B4*****_v2**

In Table 6.5 are presented the numerical resistance solutions for the T-joints analysed above (subchapter 6.3.1) for a rotation limit equivalent to 0,015 rads. In Figure 6.18 are presented the ratios of these resistances.

Table 6.5 - T-joints between open sections *vs* T-joints between hollow and open sections (Numerical bending capacity)

Numerical		**Numerical**		**Ratio**
Joints	$M_{j1,Rd}$ (kNm)	Joints	$M_{j2,Rd}$ (kNm)	$M_{j1,Rd}/M_{j2Rd}$
B1	28,29	***B1*****_ v2**	27,81	1,02
B2	18,99	***B2*****_ v2**	22,37	0,85
B3	19,60	***B3*****_ v2**	22,10	0,89
B4	26,43	***B4*****_ v2**	27,89	0,95

$M_{j1,Rd}$ – Bending capacity of joints between hollow and open sections; $M_{j2,Rd}$ – Bending capacity of joints between open sections

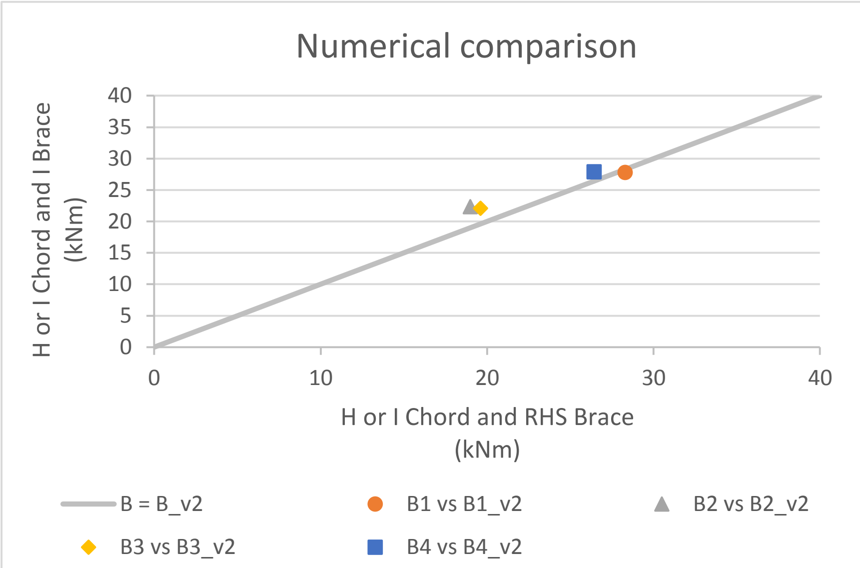


Figure 6.18 - Analytical comparison between the joints of Chapter 5 and Chapter 6

Observing these results seems that the numerical curves of the two types of joints are very similar. This fact allows to conclude that, the behavior of the T-joints with an open section chord and an hollow section brace is similar to the behavior of the T-joints with two open sections in terms of resistance capacity (Figure 6.18), stiffness and rotational capacity (Figures 6.14 to 6.17).

7. CONCLUSION AND FURTHER DEVELOPMENTS

7.1. Conclusion

The objective of the present dissertation was to increase the knowledge on the structural behavior of welded joints between hollow and open sections. Considering the obtained results, it can be concluded that:

- The analytical models for joints between open sections (chord) and hollow sections (brace), have lack of physical meaning.
- In T-joints between H or I sections (chord) and RHS sections (brace), the walls of the RHS profile which are parallel to the web of the H or I section, do not affect meaningfully the resistance of the whole joint, when subjected to bending.
- The component method can be applied to T-joints between open sections (chord) and hollow sections (brace), considering the studied moment-rotation curves.
- To achieve the entire application of the component method for T-joints between open sections (chord) and hollow sections (brace), the existing empirical formulas on EC 3-1-8 or CIDECT, can represent the resistance of each component of the joints, where each formula represents a component.

7.2. Further developments

Along the development of this dissertation, it was noticed the necessity for more information in some fields of application where some further investigations could be taken, such as:

- Include the welds in the numerical models to verify and study its effect on the behavior of joints with hollow sections.
- Use true stress-true strain steel curves in the numerical models, regarding the steel used in the respective experimental tests.
- Development of a parametrical study in these type of joints to understand the existing empirical formulas and try to give them some physical meaning.
- Perform more experimental tests on these type of joints.
- Bring up more reliable numerical and benchmark studies to lead the research on this subject to an utmost and profitable level.

REFERENCES

ABAQUS 6.13 (2013). Abaqus/CAE User's Manual version 6.13, Dassault Systémes Simulia Corp., Providence, RI, US

Arup@, http://www.arup.com/Projects/London_2012_ArcelorMittal_Orbit.aspx. (http://www.arup.com). Acess in April 11, 2015.

Azevedo, Álvaro F.M. (2003) "Método dos elementos finitos", 1ª edição, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal

CAU/RJ@. http://www.caurj.org.br/?p=7267&print=print. Serviço Público Federal Conselho de Arquitetura e Urbanismo do Rio de Janeiro. (http://www.caurj.org.br). Acess in April 1, 2015.

CEN, European Committee for Standardization (2010*a*). "EN 1993-1-1, Eurocode 3 – Design of steel structures – Parte 1-1: General rules and rules for buildings". Brussels, Belgium.

CEN, European Committee for Standardization (2006). "EN 10210-2, Hot finished structural hollow sections of non-alloy and fine grain steels – Part 2: Tolerances, dimensions and sectional properties". Brussels, Belgium.

CEN, European Committee for Standardization (2010*b*). "EN 1993-1-8, Eurocode 3 – Design of steel structures – Part 1-8: Design of joints". Brussels, Belgium.

CEN, European Committee for Standardization (2010*c*). "EN 1993-1-5, Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements". Brussels, Belgium.

Chen, Y., Wu, Y. (2015) "Behaviour of square hollow section brace – H – shaped steel chord T – joints under axial compression" Elsevier, Thin-Walled structures 89 192 – 204, School of urban construction,Yangtze University, Jingzhou, China, College of civil engineering, Huaqiao University, Xiamen, China

CISC@. http://www.cisc-icca.ca/solutions-centre/publications/publications-old/technical-publications/hollow-structural-section-connections-and-trusses#.VfrUiN9Viko. (http://www.cisc-icca.ca/). Acess in September 17, 2015.

Dias da Silva, V. (2002). “Introdução á Análise Não Linear de Estruturas”. Faculdade de Ciências e Tecnologia da Universidade de Coimbra, Departamento de Engenharia Civil, Coimbra, Portugal.

Dias da Silva, V. (2004) “Mecânica e Resistência dos Materiais”, 3ª Edição, Coimbra, Portugal

English Heritage@. (2015). (http://www.english-heritage.org.uk/siteSearchResults?showTotals=true&terms=Iron%20Bridge&mode=Site&Attempt=First). (http://www.english-heritage.org.uk/). Acess in September 17, 2015.

Freitas, P. (2013). “Configurações inovadoras de ligações de emenda de barras metálicas de secção tubular”. Tese de Mestrado, Departamento de Engenharia civil da Universidade de Coimbra, Coimbra, Portugal.

Giddings, T.W. (1985). “The development of recommendations for the design of welded joints between steel structural hollow sections and H-sections”. Final report, Report Eur 9462 EN, Comission of the European Communities. London, England.

IIW@. (http://www.iiwelding.org/TheIIW/Organization/Pages/WGStand.aspx). (http://www.iiwelding.org). Acess in September 26, 2015.

Jaspart, J. P., Pietrapertosa, C., Weynand, K., Busse, E., Klinkhammer, R. (2005a). “Development of a full consistent design approach for bolted and welded joints in building frames and trusses between steel members made of hollow and/or open sections – Application of the component method; Volume 1 – Pratical Guidelines”. CIDECT, Comité International pour le déveleppement et l’étude de la construction tubulaire. Report: 5BP, University of Liège, Liège, Belgium, PSP Technologien Gmbh, Aachen, Germany.

Jaspart, J. P., Pietrapertosa, C., Weynand, K., Busse, E., Klinkhammer, R. (2005b) “Development of a full consistent design approach for bolted and welded joints in building frames and trusses between steel members made of hollow and/or open sections – Application of the component method Volume 2 – Progress of the scientific activities on joint components and assembly” CIDECT Report: 5BP. University of Liège, Liège, Belgium, PSP Technologien Gmbh, Aachen, Germany

Jaspart, J.P. (2000). “General report: session on connections”. Journal of Construction Steel Research, vol. 55, pp. 69-89. Department MSM, University of Liège. Liège, Belgium.

Jurcikova, A., Rosmanit, M. (2012) “FEM model of a joint consisting RHS and HEA profiles” Elsevier, Procedia Engineering 40 183 – 188, Steel structures and bridges, Faculty of civil engineering VSB-Technical University of Ostrava, Ostrava, Czech Republic

Koning, C.H.M., Wardenier, J. (1985). “The static strength of welded joints between structural hollow sections or between structural hollow sections and H-sections; Part 3: Joints between structural hollow section bracings and an H-sections chord”. TNO-IBBC Report nr. Bi-85-39/63.4.3470. Stevin Report 6-84-24. Department of Civil Engineering, Delft University of Technology, Netherlands.

Lachal, A., Petit, L. (1983). “Improvement of design and calculation methods for steel hollow sections to H type section welded joints”. CECA program: The Development of recommendations for the design of welded joints steel structural hollow sections or between steel structural hollow sections and H sections. Structures Laboratory I.N.S.A. in Rennes, France.

LNEC@. http://www.lnec.pt/pt/servicos/normalizacao-e-regulamentacao/normalizacao/. (http://www.lnec.pt/). Acess in September 17, 2015.

Martifer Group@, http://www.martifer.pt/pt/portfolio/detalhes.php?sec=4&tipo=3&id=29 , (http://www.martifer.pt/). Acess in April 16, 2015)

Sabatka, L., Kabelác, J., Kolaja, D., Pospísil, M. (2014). “Structural analysis and design of steel connections using CBFEM method”. Journal of Civil Engineering and Architecture. Brno, Czech Republic.

Simões da Silva, L., Santiago, A. (2003).“Manual de ligações metálicas”. Research project: Continuing Education in Structural Connections. CMM, Associação Portuguesa de Construção Metálica e Mista, Coimbra, Portugal.

Wald, F., Kwasniewski, L., Gödrich, L., Kurejková, M. (2014) “Validation and verification procedures for connection design in steel structures”, Resear project MERLION of Czech Republic Technical No. TA02010159, Czech Technical University of Prague, Department of steel and timber structures, Prague, Czech Republic

Wardenier, J. (1982). “Hollow section joints”. Department of Civil Engineering, Delft University of Technology, Netherlands.

Wardenier, J., Koning, C.H.M., van Douwen, A.A. (1977). “Behaviour of axially loaded K- and N type joints with bracings of structural hollow sections and I- profile as chord”. CECA-convention/nr. 6210-SA/3/604. IIW Doc. XV–401–77. Stevin report nr. 6–77–8. TNO–IBBC report nr. BI–77–42/35.3.51210. CIDECT report nr. 77/12(5Q). Department of Civil Engineering, Delft University of Technology, Netherlands.

Wardenier, J., Koning, C.H.M., van Dowen, A.A. (1977). “Behaviour of axially loaded K- and N type joints with bracings of structural hollow sections and an I-profile as chord”. IIW Doc. XV – 401 – 77. Stevin report nr. 6 – 77 – 8. TNO – IBBC report nr. BI – 77 – 42/35.3.51210. CIDECT report nr. 77/12(5Q).Department of Civil Engineering, Delft University of Technology, Netherlands.

Wardenier, J., Packer, J.A., Zhao, X.-L., van der Vegte, G.J. (2010). “Hollow sections in structural applications”. Geneva, Suiça.

Weynand, K., Jaspart, J. P., Ly, L. (2003). “Application of the component method to joints between hollow and open sections”. CIDECT, Comité International pour le déveleppement et l’étude de la construction tubulaire. Research project 5BM. PSP Technologien Gmbh, Aachen, Germany, University of Liège, Liège, Belgium.

Zhao, X.-L. (2013). “Comission XV Design, Analysis and Fabrication of welded structures, XV-1439-13 ISO/FDIS 14346: Static design procedure for welded hollow-section joints – Recommendations”. IIW, International institute of welding, Clayton, Australia