Chapter 1

INTRODUCTION

1.1 GENERAL

1.1.1 Aims of the book

The aim of the present book is threefold:

- To provide designers with practical guidance and tools for the design of steel joints;
- To point out the importance of structural joints on the response of steel structures and to show how the actual behaviour of joints may be incorporated into the structural design and analysis process;
- To illustrate the possibilities of producing more economical structures using the new approaches offered in the European code of practice for the design of steel structures, Eurocode 3 (EN 1993) as far as structural joints are concerned.

The organisation of the book reflects the belief that, in addition to the sizing of the members (beams and columns), consideration should also be given to the joint characteristics throughout the design process. This approach, despite the novelty it may present to many designers, is shown to be relatively easy to integrate into everyday practice using currently available design tools.

Hence the present book addresses design methodology, structural analysis, joint behaviour and design checks, at different levels:

- Presentation and discussion of concepts;
- Practical guidance and design tools.

1.1.1.1 The traditional common way in which joints are modelled for the design of a frame

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

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

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

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

1.1.1.2 The semi-continuous approach

The rotational behaviour of actual joints is well recognised as being often intermediate between the two extreme situations, i.e. rigid or pinned. In sub-chapter 1.2, the difference between joints and connections will be introduced. For the time being, examples of joints between one beam and one column only will be used. Let us now consider the bending moments and the related rotations at a joint (Figure 1.1).

When all the different parts in the joint are sufficiently stiff (i.e. ideally infinitely stiff), the joint is rigid, and there is no difference between the respective rotations at the ends of the members connected at this joint (Figure 1.1a). The joint experiences a single global rigid-body rotation which is the nodal rotation in the commonly used analysis methods for framed structures.

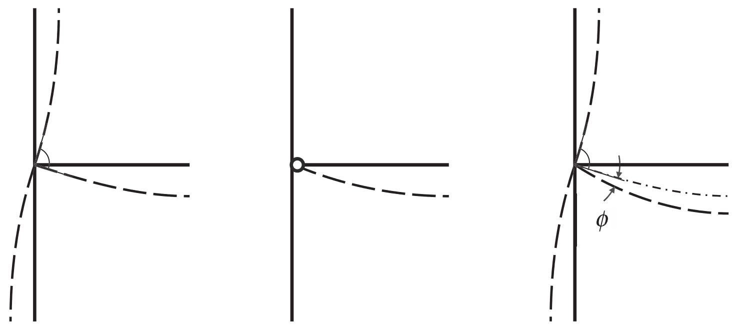
Should the joint have only negligible stiffness, then the beam will behave just as a simply supported beam, whatever the behaviour of the other connected member(s) (Figure 1.1b). This is a pinned joint.

For intermediate cases (non-zero and finite stiffness), the moment transmitted will result in a difference ϕ between the absolute rotations of the two connected members (Figure 1.1c). The joint is classified as semi-rigid in these cases.

The simplest way for representing this concept is a rotational (spiral) spring between the ends of the two connected members. The rotational stiffness S_j of this spring is the parameter that links the transmitted moment M_j to the relative rotation ϕ, which is the difference between the absolute rotations of the two connected members.

When this rotational stiffness S_j is zero, or when it is relatively small, the joint falls back into the pinned joint class. In contrast, when the rotational stiffness S_j is infinite, or when it is relatively high, the joint falls into the rigid joint class. In all the intermediate cases, the joint belongs to the semi-rigid joint class.

For semi-rigid joints the loads will result in both a bending moment M_j and a relative rotation ϕ between the connected members. The moment and the relative rotation are related through a defined behaviour depending on the joint properties. This is illustrated in Figure 1.2 where, for the sake of simplicity, an elastic response of the joint is assumed in view of the structural analysis to be



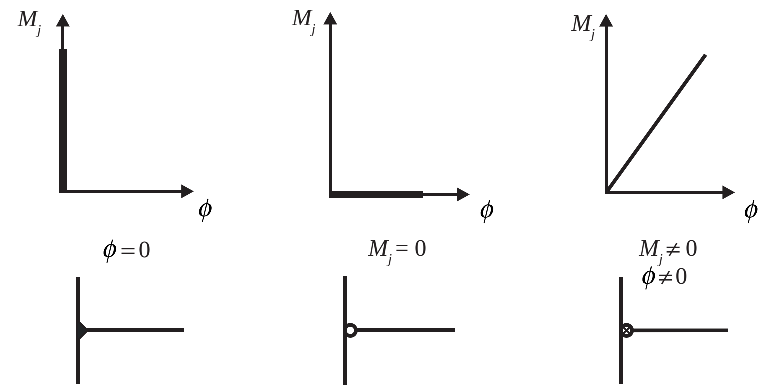
(a) Rigid Joint (b) Pinned joint (c) Semi-rigid joint

Figure 1.1 – Classification of joints according to stiffness

performed (how to deal with non-linear behaviour situations will be addressed later on, especially in chapter 2).

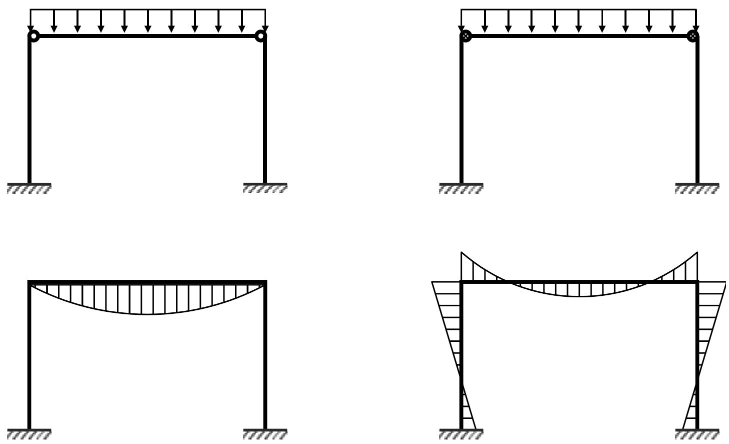
It has to be understood that the effect, at the global analysis stage, of having semi-rigid joints instead of rigid or pinned joints is to modify not only the displacements, but also the distribution and magnitude of the internal forces throughout the structure.

As an example, the bending moment diagrams in a fixed-base simple portal frame subjected to a uniformly distributed load are given in Figure 1.3 for



(a) Rigid joint (b) Pinned joint (c) Semi-rigid joint

Figure1.2 – Modelling of joints (case of elastic global analysis)



Pinned joints Semi-rigid joints

Figure 1.3 – Elastic distribution of bending moments in a simple portal frame

two situations, where the beam-to-column joints are respectively either pinned or semi-rigid. The same kind of consideration holds for deflections.

1.1.1.3 The merits of the semi-continuous approach

Both the EN 1993 requirements and the desire to model the behaviour of the structure in a more realistic way leads to the consideration of the semi-rigid behaviour when necessary.

Many designers would stop at that basic interpretation of EN 1993 and hence would be reluctant to confront the implied additional computational effort involved. Obviously a crude way to deal with this new burden will be for them to design joints that will actually continue to be classified as being pinned or fully rigid. However, those properties will have to be proven at the end of the design process; in addition, such joints will certainly be uneconomical in a number of situations.

It shall be noted that the concept of rigid and pinned joints still exists in EN 1993. It is accepted that a joint which is almost rigid or, on the contrary, almost pinned, may still be considered as being truly rigid or truly pinned in the design process. How to judge whether a joint can be considered as rigid, semi-rigid or pinned depends on the comparison between the joint stiffness and the beam stiffness, which depends on the second moment of area and length of the beam.

The designer is strongly encouraged to go beyond this "all or nothing" attitude. Nowadays, it is possible, and therefore of interest, to consider the benefits to be gained from the semi-rigid behaviour of joints. Those benefits can be brought in two ways:

1. The designer decides to continue with the practice of assuming – sometimes erroneously – that joints are either pinned or fully rigid. However, EN 1993 requires that proper consideration be given to the influence that the actual behaviour of the joints has on the global behaviour of the structure, i.e. on the accuracy with which the distribution of forces and moments and the displacements have been determined. This may not prove to be easy when the joints are designed at a late stage in the design process since some iteration between global analysis and design checking may be required. Nevertheless, the following situations can be foreseen:

- For a joint to be assumed rigid, it is common practice to introduce, for instance, column web stiffeners in a beam-to-column joint. EN 1993 now provides the means to check whether such stiffeners are really necessary for the joint to be both rigid and have sufficient resistance. There are practical cases where they are not needed, thus permitting the adoption of a more economical joint design;
- When joints assumed to be pinned are later found to have fairly significant stiffness (i.e. to be semi-rigid), the designer may be in a position to reduce beam sizes. This is simply because the moments carried out by the joints reduce the span moments and deflections in the beams.

2. The designer decides to give consideration, at the preliminary design stage, not only to the properties of the members but also to those of the joints. It will be shown that this new approach is not at all incompatible with the frequent separation of the design tasks between those who have the responsibility for conceiving the structure and carrying out the global analysis and those who have the responsibility for designing the joints. Indeed, both tasks are very often performed by different people, indeed, or by different companies, depending on national or local industrial habits. Adopting this novel approach towards design requires a good understanding of the balance between, the costs and the complexity of joints and, the optimisation of the structural behaviour and performance through the more accurate consideration of joint behaviour for the design as a whole. Two examples are given to illustrate this:

   - It was mentioned previously that it is possible in some situations to eliminate stiffeners, and therefore to reduce costs. Despite the reduction in its stiffness and, possibly, in its strength, the joint can still be classified as rigid and found to have sufficient strength. This is possible for industrial portal frames with rafter-to-column haunch joints, in particular, but other cases can be envisaged;
   - In a more general way, it is worth considering the effect of adjusting the joint stiffness so as to strike the best balance between the cost of the joints, and the cost of the beams and columns. For instance, for braced frames, the use of semi-rigid

joints, which are probably more costly than the pinned joints, enables reducing the beam sizes. For unbraced frames, the use of less costly semi-rigid joints, instead of the rigid joints, leads to increased column sizes and possibly beam sizes.

Of course the task may seem difficult, and this is why the present book, in chapter 9, is aimed at providing the designer with a set of useful design strategies. The whole philosophy could be summarised as "*As you must do it, so better make the best of it*".

Thus, EN 1993 now offers the designer the choice between a conservative attitude, where however something may often be gained, and an innovative attitude, where the economic benefits may best be sought.

It is important to stress the high level of similarity that exists between response of member cross sections and that of structural joints. This topic is addressed in the following section.

1.1.1.4 A parallel between member cross sections and joints

Member cross section behaviour may be considered by means of a moment-rotation curve, $M - \theta$, for a simply supported beam loaded at mid span (M here is the bending-moment at mid-span, and θ is the sum of rotations at the span ends). Joint behaviour will be considered through a similar relationship, but with $M = M_j$ being the bending moment transmitted by the joint, and $\theta = \phi$ being the relative rotation between the connected members. Those relationships have a similar shape as illustrated in Figure 1.4.

According to EN 1993, member cross sections are divided into four classes, especially according to their varying ability to resist local instability, when partially or totally subject to compression, and the consequences this may have on the possibility for plastic redistribution. Therefore, their resistance ranges from the full plastic resistance (class 1 and 2) to the elastic resistance (class 3) or the sub-elastic resistance (class 4).

The specific class to which a cross section belongs defines the assumptions on:

– The behaviour to be idealised for global analysis (i.e. class 1 will allow the formation of a plastic hinge and the redistribution of internal forces in the frame as loads are increased up to or beyond the design loads);

– The behaviour to be taken into account for local design checks (i.e. class 4 will imply that the resistance of the cross section is based on the properties of a relevant effective cross section rather than those of the gross cross section).

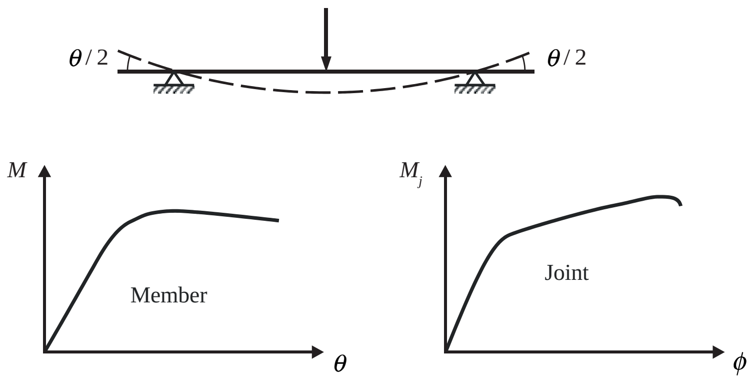


Figure 1.4 – Moment vs. rotation characteristics for member cross section and joint

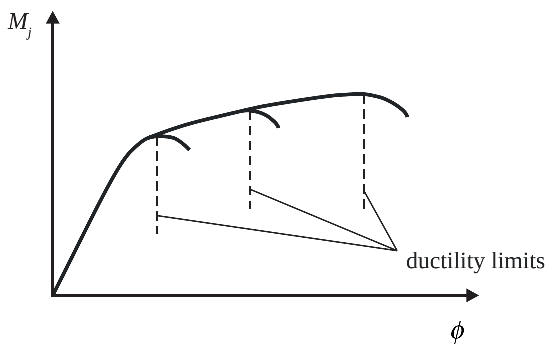


Figure 1.5 – Ductility or rotation capacity in joints

The classification of a cross section is based on the width-to-thickness ratio of the steel component walls of the section. Ductility is directly related to the amount of rotation during which the design bending resistance will be sustained. This results in a so-called rotation capacity concept.

In a similar manner, joints are classified in terms of ductility or rotation capacity, see Figure 1.5. This classification is a measure of their ability to resist premature local instability, and premature brittle failure (especially due to bolt or weld failure) with due consequences on the type of global analysis allowed.

The practical interest of such a classification for joints is to check whether an elastoplastic global analysis may be conducted up to the formation of a plastic collapse mechanism in the structure, which implies the formation of hinges in some of the joints.

As will be shown, this classification of joints by ductility, although not explicitly stated in EN 1993, may be defined from the geometric and mechanical properties of the joint components (bolts, welds, plate thickness, etc.). Joints may therefore be classified according to both stiffness, and ductility.

Additionally, joints may be classified according to their strength, as full-strength or partial-strength, according to their resistance compared to the resistance of the connected members. For elastic design, the use of partial-strength joints is well understood. When plastic design is used, the main use of this classification is to anticipate the possible need to allow a plastic hinge to form in the joint for global analysis. In order to allow increase of loads, a partial-strength joint may be required to act as a plastic hinge from the moment its bending resistance is reached. In that case, the joint must also have sufficient ductility (plastic rotation capacity).

The final parallel to be stressed between joints and members is that the same kind of link exists between the global analysis stage and the ultimate limit states design checking stage. The latter has to extend to all aspects that were not implicitly or explicitly taken into account at the global analysis stage. Generally speaking, it can be stated that the more sophisticated the global analysis is, the simpler the ULS design checks required.

The choice of global analysis will thus depend not only on what is required by EN 1993 but also on personal choices, depending on specific situations, available software tools, etc. A particular choice means striking a balance between the amount of effort devoted to global analysis and the amount of effort required for the check of remaining ULS, see Figure 1.6.

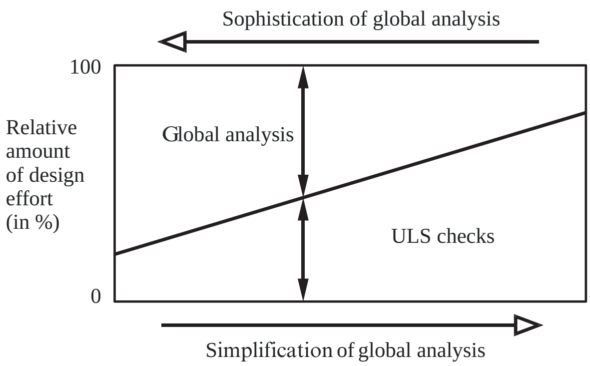


Figure 1.6 – Schematic of the proportion of effort for global analysis and for ULS checks

UK Specific Comment

UK practice normally considers only two main forms of joint modelling, simple and continuous. Ideally frames designed on the basis of simple joint modelling should use joints between members that possess negligible rotational stiffness, and transmit the beam reactions in shear into the columns without developing significant moments. These joints may be treated as perfect pins. Members can then be designed in isolation, either as (predominantly) axially loaded columns, or as simply supported beams. Joints designed within the principles of continuous modelling are capable of transmitting significant moments and are able to maintain the original angle between adjacent members virtually unchanged. This form of construction of steel frames is particularly advantageous when beam deflections are critical or if bracing systems are not possible.

To comply with one or the other design assumption, steelwork connections are detailed as simple or moment-resisting joints. Because joints in frame structures are often treated by adopting a degree of standardisation, the designer usually details the connections in accordance with the principles in the series of SCI Green Books, which present this material in the form of step-by-step design procedures and tables covering standard arrangements.

Other approximate analysis methods continue to be used among British designers, the most representative being the wind-moment method for the design of unbraced multi-storey steel frames. The method assumes that (i) under gravity loads the beam-to-column joints act as pinned connections, and (ii) under horizontal wind loads these joints are rigid, although it lacks transparency. This method can be seen as a manifestation of the semi-continuous principles. Wind moment connection details can be found in SCI (1995a). With the now widespread use of software in design offices for rigorous second-order analysis, it is doubtful if the use of the wind-moment method should be advocated any longer, other than for initial sizing.

1.1.2 Brief description of the contents of the book

This book is divided into three main parts:

– The first part covered by chapter 2 is a recall of the available methods of structural analysis and design, but with a special focus on the integration of the joint response into the whole design process;

– The second part includes chapter 3 to chapter 8. It concerns the characterisation of the joint properties in terms of resistance, stiffness, ductility. Chapter 3 to chapter 7 concentrate on the response of joints under static loading while chapter 8 provides information about the extension of the previously described joint characterisation methods to fire, seismic and fatigue loading;
– In the third part, covered by chapter 9, design opportunities to optimise the structural frames and the constitutive joints are presented, and guidelines are provided on how to implement them into daily practice.

Practical design recommendations and tools are presented throughout the whole book. This is intended to help the reader to "materialise" the possibly new concepts and the way to "operate" them in their design activities. In order to achieve a full consistency with the European norms, the notations adopted in the book are basically those suggested in EN 1993. The design rules implemented in the codes are not all repeated in the present book, which is aimed at commenting the joint related aspects of EN 1993, but certainly not at "replacing" the normative documents.

Finally, it has to be mentioned that only one load case is generally considered in the book, when referring to the design of a specific joint or frame, whereas, in actual projects, several load combination cases have obviously to be dealt with.

1.1.3 Types of structural systems and joints covered

No particular type of structure is excluded but the book is focusing more particularly on steel building frames. As far as joints are concerned, the configuration and the loading conditions addressed in this book are, strictly speaking, those directly covered by part 1-8 of EN 1993, EN 1993-1-8 (CEN, 2005c); however, as the Eurocode design principles are also valid for many other types of joints and loading conditions, the scope the present book extends to the following:

– Member cross sections: open H or I rolled and welded sections, as well as tubular sections;
– Connections with mechanical fasteners;
– Welded connections;

– Beam-to-column joints, beam-to-beam joints, and column or beam splices: simple and moment-resisting joints with welded or bolted steel connections (with or without haunch) and using various connecting elements (end plates, angles, fin plates) and stiffening systems;
– Column bases: bolted end plate connections;
– Lattice girder joints;
– Joints under static and non-static loading, including fire, seismic and fatigue loading.

1.1.4 Basis of design

EN 1993-1-8 is one of the twelve constitutive parts of EN 1993. Accordingly, its basis for design is fully in line with EN 1993, in terms of basic concepts, basic variables, ULS, SLS, durability and sustainability. Reference will therefore be done here to the introductory chapter of the ECCS Design Manual “Design of steel structures” (Simões da Silva *et al*, 2010) where the here-above mentioned aspects are extensively discussed.

1.2 DEFINITIONS

One of the advantages of the steel construction lies in the prefabrication of steel pieces. These pieces, according to the needs or the transportation requirements, will have to be assembled either on site or directly in the workshop. The connections resulting from this fabrication and erection process will sometimes be there just for constructional reasons; but in many cases, they will have to transfer forces between the connected pieces and so play an important structural role. One speaks then about “structural connections, which are a particular focus of this book.

In the past, forging was used as a convenient way to assemble steel pieces together. This was achieved through the melting of the steel at the interface between the connected pieces. Nowadays, in modern constructions, use is made of welds and mechanical fasteners, including various bolt types (e.g. TC bolts, injection bolts, flow drill bolting), pins, or nails. Beside the connectors, a great variety of configurations is also required in daily practice, sometimes as a result of the architectural demand or for economic reasons. This leads to a number of situations to which the practitioner is likely to be faced. Moreover, various loading situations (e.g. static, dynamic, seismic, fire) and structural systems (e.g. buildings, bridges, towers) are also to be considered.

In this book, an exhaustive consideration of all the existing connection and joint configurations is obviously not practical; therefore the authors have tried to deliver the most updated and complete information in the domain by structuring chapter 3 to chapter 8 as follows:

- Chapter 3: Connections with mechanical fasteners;
- Chapter 4: Welded connections;
- Chapter 5: Simple joints;
- Chapter 6: Moment-resisting joints;
- Chapter 7: Lattice girder joints;
- Chapter 8: Joints under various loading situations.

Chapter 3 and 4 refer to basic joints where mainly tension and/or shear forces are to be transferred between the connected pieces. The most important design property is their level of resistance.

Chapter 5 is devoted to connections aimed at transferring shear forces, possibly together with axial forces, between two elements, but additionally allow free rotation between these elements. The word “simple” refers to this specific ability.

Chapter 6 concentrates on moment-resisting joints where a combination of axial/shear forces and moments are to be carried over by the joint. Their rotational stiffness plays an important role in terms of global structural response. In the case of plastic frame analysis, their rotational capacity will also have to be assessed.

In chapter 7, lattice girder joints are discussed, and, in chapter 8, the extension of the static design rules to fire, fatigue or earthquake situations will be discussed.

The design of the basic connections (covered in chapters 3 and 4) and of lattice girder joints according to the Eurocodes is quite similar to what practitioners have been used to apply in the last decades at the national level while new concepts are now available for the design of simple and moment-resisting joints. The next pages will allow the designer to familiarise with important definitions used in the book.

1.2.1 Joint properties

The specific case of building frames is addressed below, just for the sake of illustration.

Building frames consist of beams and columns, usually made of H, I or tubular shapes that are assembled together by means of connections. These connections are between two beams, two columns, a beam and a column, or a column and the foundation (Figure 1.7).

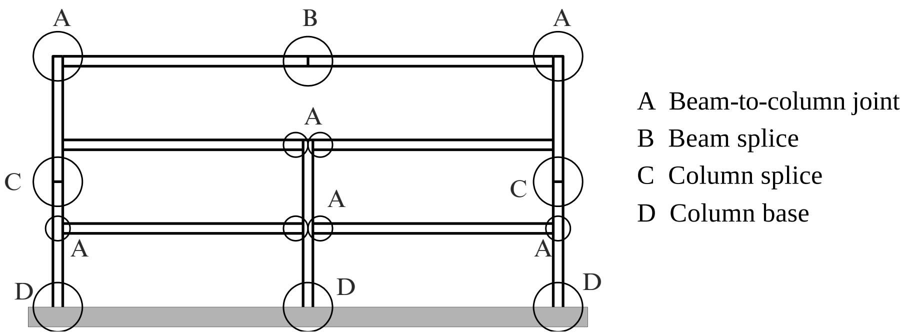
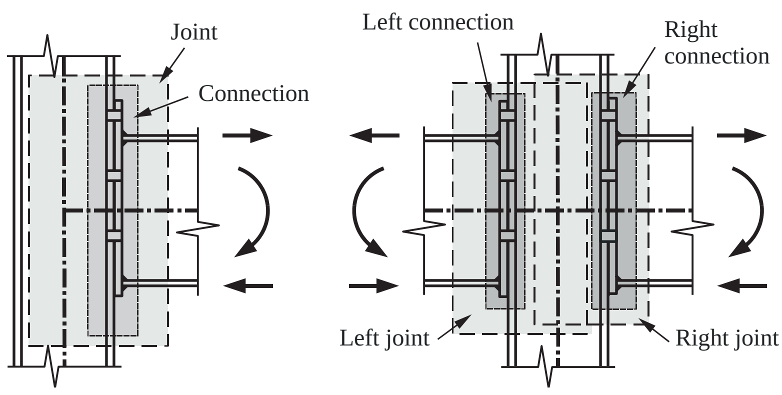


Figure 1.7 – Different types of joints in a building frame



(a) Single sided joint configuration (b) Double sided joint configuration

Figure 1.8 – Joints and connections

A *connection* is defined as the set of the physical components which mechanically fasten the connected elements. The connection is assumed to be concentrated at the location where the fastening action occurs, for instance at the beam end/column interface in a beam-to-column joint. When the connection as well as the corresponding zone of interaction between the connected members are considered together, the wording *joint* is used (Figure 1.8a).

Depending on the number of in-plane elements connected together, single-sided and double-sided joint configurations are defined (Figure 1.8). In a double-sided configuration (Figure 1.8b), two joints – left and right – have to be considered.

The definitions illustrated in Figure 1.8 are valid for other joint configurations and connection types.

As explained previously, the joints, which are traditionally considered as rigid or pinned, and designed accordingly, possess, in reality, their own degree of flexibility resulting from the deformation of all the constitutive components. Section 1.2.2 describes the main *sources of joint deformation*. Sub-chapter 2.2 provides information on how to *model* the joints in view of frame analysis. This modelling depends on the level of joint flexibility. In sub-chapter 2.3, the way in which the shape of the non-linear joint deformation curves may be *idealised* is given. Section 1.6.2 refers to the *component method* as a general tool for the prediction of the main joint mechanical properties in bending. The concept of *joint classification* is introduced in sub-chapter 2.4. Finally, it is commented on the *ductility classes* of joints in sub-chapter 2.5. This aspect, expressed in terms of rotational capacity, will be of particular importance in cases where a plastic structural analysis is carried out.

1.2.2 Sources of joint deformation

As stated in sub-chapter 1.1, the rotational behaviour of the joints may affect the local and/or global structural response of the frames. In this sub-chapter, the sources of rotational deformation are identified for beam-to-column joints, splices and column bases.

It is worth mentioning that the rotational stiffness, the resistance and the rotation capacity are likely to be affected by the shear force and/or the axial force acting in the joint. These shear and axial forces may obviously have contributions to the shear and axial deformation within the joint. However, it is known that these contributions do not affect significantly the frame response. Therefore, the shear and axial responses of the joints, in terms of rotational deformation, are usually neglected in applications as buildings.

1.2.2.1 Beam-to-column joints

1.2.2.1.1 Major axis joints

In a major axis beam-to-column joint, different sources of deformation can be identified. For the particular case of a single-sided joint (Figure 1.9a and Figure 1.10a), these are:

- *The deformation of the connection* that includes the deformation of the connection elements: column flange, bolts, end plate or angles, and the load-introduction deformation of the column web resulting from the transverse shortening and elongation of the column web under the compressive and tensile forces F_b acting on the column web. The couple of forces F_b are statically equivalent to the moment M_b at the beam end. These deformations result in a relative rotation ϕ_c between the beam and column axes; this rotation, which is equal to $\theta_b - \theta_c$ (see Figure 1.9a), results in a flexural deformation curve $M_b - \phi_c$.
- *The shear deformation of the column web panel* associated with the shear force V_{wp} acting in this panel. It leads to a relative rotation γ between the beam and column axes; this rotation makes it possible to establish a shear deformation curve $V_{wp} - \gamma$.

The deformation curve of a connection may obviously be influenced by the axial and shear forces possibly acting in the connected beam.

Similar definitions apply to double-sided joint configurations (Figure 1.9b and Figure 1.10 b). For such configurations, two connections and a sheared web panel, forming two joints, must be considered.

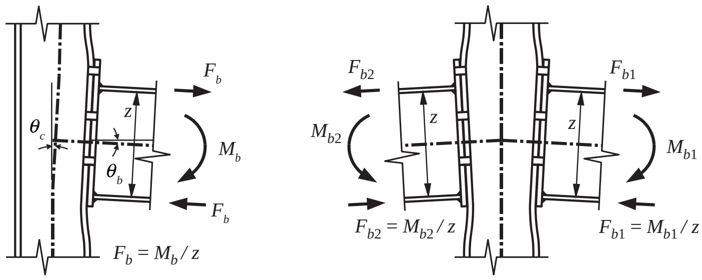
To summarise, the main sources of deformation which must be taken into account in a beam-to-column major axis joint are:

Single-sided joint configuration:

- The connection deformation $M_b - \phi_c$ characteristic;
- The column web panel shear deformation $V_{wp} - \gamma$ characteristic.

Double-sided joint configuration:

- The right hand side connection deformation $M_{b1} - \phi_{c1}$ characteristic;
- The left hand side connection deformation $M_{b2} - \phi_{c2}$ characteristic;
- The column web panel shear deformation $V_{wp} - \gamma$ characteristic.



(a) Single sided joint configuration (b) Double sided joint configuration

Figure 1.9 – Sources of joint deformation

The deformation of the connection elements and load-introduction is only due to the couple of forces transferred by the flanges of the beam, equivalent to the beam end moment M_b. The shear deformation of the column web panel results from the combined action of these equal but opposite forces and of the shear forces in the column at the level of the beam flanges. Equilibrium equations of the web panel provide the shear force V_{wp} (for the sign convention, see Figure 1.10, where the directions indicated by the arrows are positive):

$$V_{wp} = \frac{M_{b1} - M_{c2}}{z} - \frac{V_{c1} - V_{c2}}{2} \tag{1.1}$$

If the beneficial effect of the shear force in the column is neglected, Eq. (1.1) simplifies as follows:

$$V_{wp} = \frac{M_{b1} - M_{b2}}{z} \tag{1.2}$$

In both formulae, z is the lever arm of the resultant tensile and compressive forces in the connection(s). Chapter 6 explains how to derive the value of z.

1.2.2.1.2 Minor axis joints

A similar distinction between *web panel* and *connection* must also be made for a minor axis joint (Figure 1.11). The column web exhibits a so-called out-of-plane deformation while the connection deforms in bending, as in a major axis joint. However, no load-introduction deformation is involved.

In the double-sided joint configuration, the out-of-plane deformation of the column web depends on the bending moments experienced by the right and left connections (see Figure 1.12):

$$\Delta M_b = M_{b1} - M_{b2} \tag{1.3}$$

For a single-sided joint configuration (Figure 1.11), ΔM_b is M_b.

1.2.2.1.3 Joints with beams on both major and minor column axes

A three-dimensional (3-D) joint is characterised by the presence of beams connected to both the column flange(s) and web (Figure 1.13). In such joints, a shear deformation (see 1.2.2.1.1) and an out-of-plane deformation (see 1.2.2.1.2) of the column web develop simultaneously.

The loading of the web panel appears therefore as the superimposition of the shear loading given by Eq. (1.1) or (1.2) and the out-of-plane loading given by Eq. (1.3).

The joint configuration of Figure 1.13 involves two beams only; configurations with three or four beams can also be found.

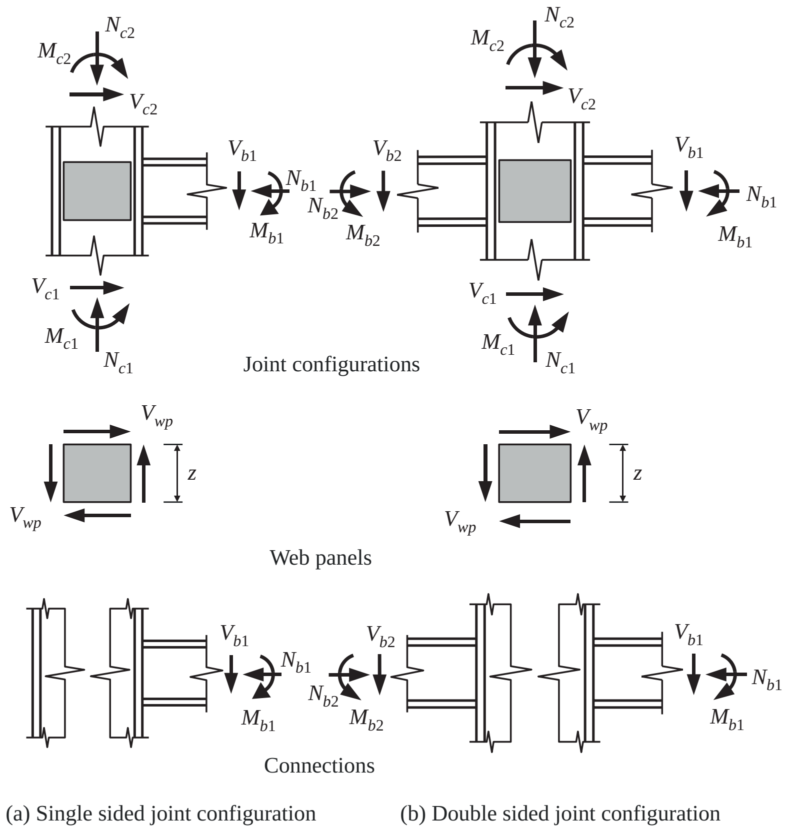


Figure 1.10 – Loading of the web panel and the connections

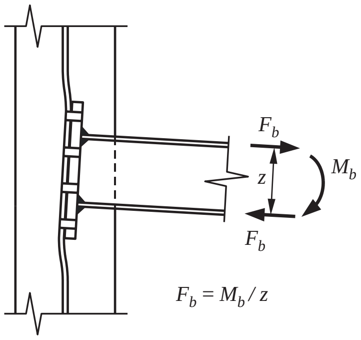


Figure 1.11 – Deformation of a minor axis joint

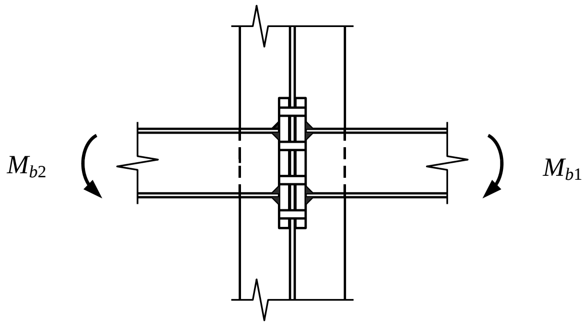


Figure 1.12 – Loading of a double-sided minor axis joint

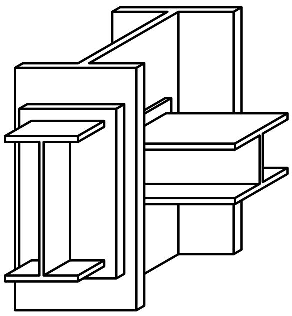


Figure 1.13 – Example of a 3-D joint

1.2.3 Beam splices and column splices

The sources of deformation in a beam splice (Figure 1.14) or in a column splice (Figure 1.15) are less than in a beam-to-column joint; indeed they are concerned with connections only. The deformation is depicted by the sole curve $M_b - \phi_c$.

This curve corresponds to the deformation of the whole joint, i.e. the two constituent connections (left connection and right one in a beam splice, upper connection and lower one in a column splice).

In a column splice where the compressive force is predominant, the axial force affects the mechanical properties of the joint significantly, i.e. its rotational stiffness, its strength and its rotation capacity. The influence, on the global frame response, of the axial deformation of splices is however limited; therefore it may be neglected provided that specific requirements given in sub-chapter 6.7 are fulfilled.

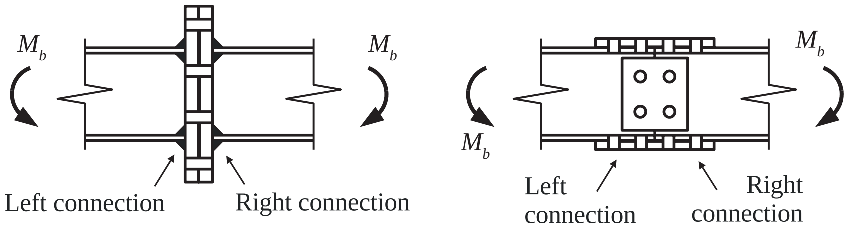


Figure 1.14 – Deformation of a beam splice

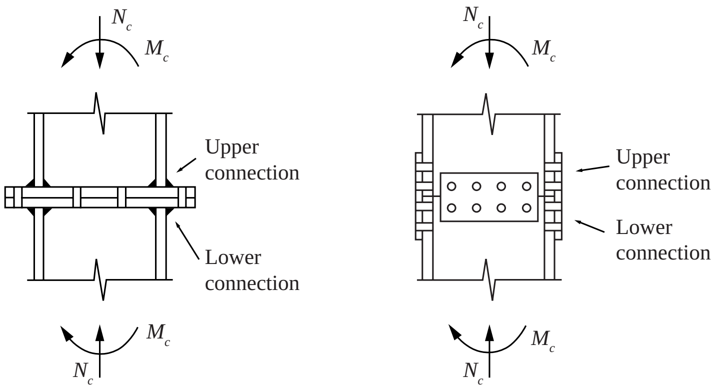


Figure 1.15 – Deformation of a column splice

1.2.4 Beam-to-beam joints

The deformation of a beam-to-beam joint (Figure 1.16) is similar to that of a minor axis beam-to-column joint; the loadings and the sources of deformation are similar to those expressed in 1.2.2.1.2 and can therefore be identified.

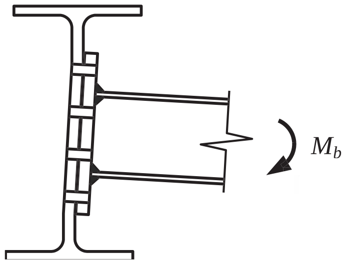


Figure 1.16 – Deformation of a beam-to-beam joint

1.2.5 Column bases

In a column base, two connection deformabilities need to be distinguished (Figure 1.17):

- The deformation of the connection between the column and the concrete foundation (*column-to-concrete connection*);
- The deformation of the connection between the concrete foundation and the soil (*concrete-to-soil connection*).

For the column-to-concrete connection, the bending behaviour is represented by a $M_c - \phi$ curve, the shape of which is influenced by the ratio of the bending moment to the axial load at the bottom of the column. For the connection between the concrete foundation and the soil, two basic deformation curves are identified:

- A $N_c - u$ curve which mainly corresponds to the soil settlement, u, due to the axial compressive force in the column, N_c; in contrast with the other types of joint, this deformation curve may have a significant effect on the frame behaviour;
- A $M_c - \phi$ curve characterising the rotation of the concrete block in the soil.

As for all the other joints described above, the deformation due to shear forces in the column may be neglected in the case of column bases.

The column-to-concrete connection and concrete-to-soil connection $M_c - \phi$ characteristics are combined in order to derive the rotational stiffness at the bottom of the column and conduct the frame analysis and design accordingly.

Similar deformation sources exist in column bases subjected to biaxial bending and axial force. The connection $M_c - \phi$ characteristics are then defined for both the major and the minor axes.

1.2.6 Hollow section joints

Design rules in EN 1993-1-8 for hollow section joints were originally developed and published as design recommendations by the International Committee for Research and Technical Support for Hollow Section Structures (CIDECT). Chapter 7 of EN 1993-1-8 gives application rules to determine the static design resistances of joints in lattice structures.

Design formulae for hollow section joints are based on semi-empirical investigations in which analytical models were fitted with test results. The complex geometry of the joints, local influences of the corners of rectangular sections and residual stresses, for instance due to welding, lead to non-uniform stress distributions. Strain hardening and membrane effects also influence the local structural behaviour. Simplified analytical models which consider the most relevant parameters were developed. In contrast to the design models for open section joints, where the so-called component method (see section 1.6.2) is used, the design of hollow section joints is based on the study of failure modes. Note that the terminology is not fully consistent here. In the previous sections, the terms *joint configuration*, *joint* and *connection* respectively are defined. For the design of hollow section joints, different types of joints are covered in EN 1993, see Table 1.1. However, here the word “joint” means *joint* and *joint configuration* at the same time. Hence, the resistance of a joint is also the resistance of the whole node, i.e. joint configuration, typically expressed by the resistances of the braces, see Figure. 1.18.

1.3 MATERIAL CHOICE

The design provisions given in the Eurocodes are only valid if material or products comply with the reference standards given in the appropriate parts of the Eurocodes. Both, in EN 1993-1-1 (CEN, 2005a) and in EN 1993-1-8 (CEN, 2005c), material and product standards are listed in sections 1.2. EN 1993-1-1, as a master document for the other parts of EN 1993, covers the

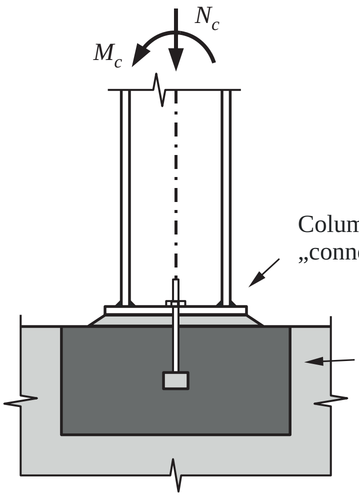
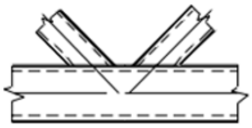
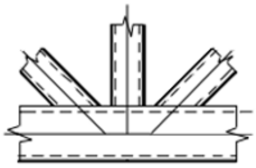
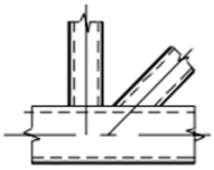
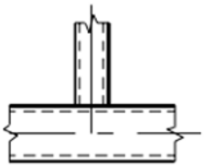
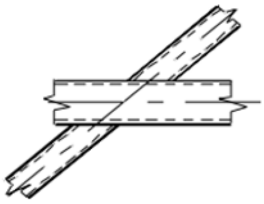
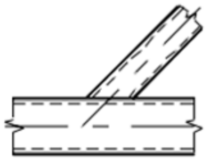
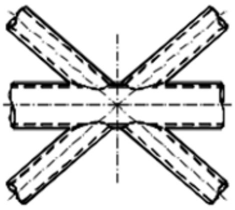
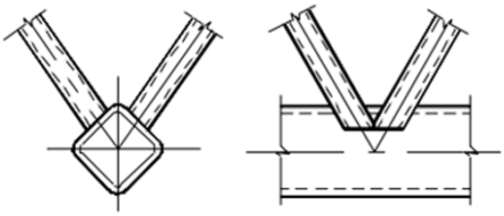
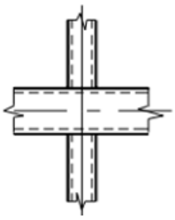
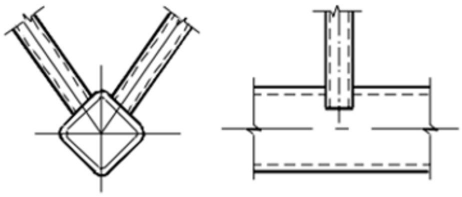
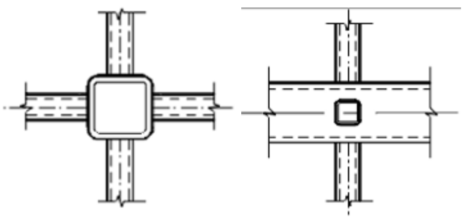


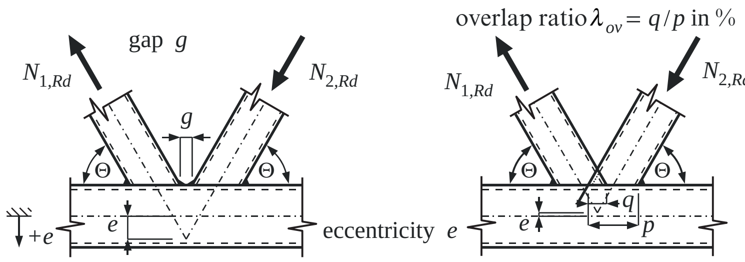
Figure 1.17 – The two connections in a column base

design of steel structures fabricated from steel material conforming to the steel grades listed in Table 3.1 of EN 1993-1-1. According to this table, also for EN 1993-1-8, only steel grades from S235 to S460 are covered. For hot rolled structural steel (I or H sections, and welded build-up sections), reference is made to EN 10025 (CEN, 2004d). For structural hollow sections, reference is made to EN 10210 (CEN, 2006c and 2006d) for hot finished sections and to EN 10219 (CEN, 2006a and 2006b) for cold formed sections. Especially with regard to the design of joints where, in many cases, a local plastic redistribution of stresses will be assumed to determine the resistance of a joint, sufficient ductility of the material is required. Steel conforming to steel grades listed in Table 3.1 of EN 1993-1-1 can be assumed to have sufficient ductility.

Due to the fabrication process, the nominal values of the yield strength f_y and also of the ultimate strength f_u depend on the thickness of the elements. It should be noted that different values for the relation between the material thickness and the material strength are given in the delivery condition speci fied in the product standards (for example, EN 10025) and in EN 1993. EN 1993 allows to define the strength values either by adopting the values directly from the product standards, or by using simplified values given in Table 3.1 of EN 1993-1-1. The user should check the National Annex where a choice could have been made.

Table 1.1 – Types of hollow section joints in lattice structures

 K joint	 KT joint	 N joint
 T joint	 X joint	 Y joint
 DK joint	 KK joint	
 X joint	 TT joint	
 DY joint	 XX joint	



(a) K gap joint

(b) K overlap joint

Figure 1.18 – Definition of design resistance, gap and overlap of a K joint

To avoid brittle fracture of tension elements, the material should have sufficient facture toughness. EN 1993-1-10 (CEN, 2005e) provides rules to select the appropriate steel grade. Several aspects should be taken into account for the choice of material:

– Steel material properties: the yield strength depending on the material thickness $f_y(t)$ and the toughness quality, which is expressed in terms of T_{27J} or T_{40J};
– Member characteristics: the member shape, detail and element thickness (t), stress concentrations according to the details in EN 1993-1-9 (CEN, 2005d) as well as fabrication flaws (for example through-thickness);
– Design situations: design value at lowest member temperature as well as maximum stresses for permanent and imposed actions derived from the design residual stress should be considered. If relevant, assumptions for crack growth for fatigue loading during an inspection interval, the strain rate for accidental actions and the degree of cold forming.

In steel assemblies where the plate is loaded in tension perpendicular to its plane, i.e. in the through-thickness direction, the material must have adequate through-thickness properties to prevent lamellar tearing, see Figure 1.19. EN 10164 (CEN, 2004e) defined quality classes in terms of Z-values.

EN 1993-1-10 gives guidance on the quality class to be used where steel with improved through-thickness properties is necessary. This is for example the case of welded beam-to-column joints (where the column flange is loaded in its through-thickness direction) or beam-to-column joints with bolted end plates (where the end plate is loaded in its through-thickness direction).

Finally, for steel grades ranging from S460 to S690 (S700) reference is made to EN 1993-1-12 (CEN, 2007a).

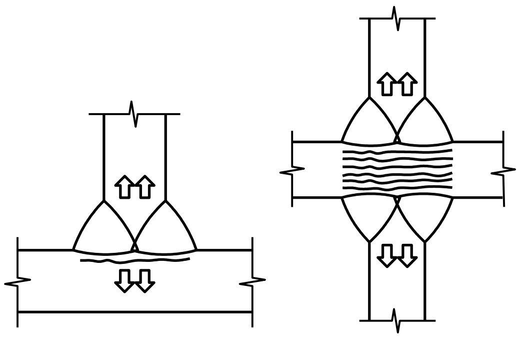


Figure 1.19 – Lamellar tearing

UK Specific Comment

Although S460 is attracting growing interest, S355 is still the dominant grarsde of steel used in the UK.

1.4 FABRICATION AND ERECTION

The design process and the fabrication and erection of a structure are not independent. In many cases, and in particular for the joints, the design procedures are directly dependent on how a structure is built and vice versa. Eurocodes, strictly speaking, are design codes, or even verification codes, i.e. they provide the designer with rules to verify the resistance, stability, deformations and ductility requirements. But no guidelines related to fabrication and erection are given. For these specific aspects, the engineer must refer to the fabrication and erection, the so-called "execution standard", EN 1090-2 (CEN, 2011). This standard gives quite a number of detailed requirements to be fulfilled.

For bolted and welded joints, the link between design rules and fabrication/execution rules are particularly strong. For instance, the design of a bolt in bearing depends on the diameter of the bolt, but also on the diameter of the bolt hole. In practice, the hole clearance, i.e. the difference of diameter between the hole and the bolt diameter, is not specified in EN 1993-1-8, but the design rules provided there are only valid if the hole is drilled in accordance with EN 1090-2 provisions. If it is not the case, then the design rule proposed in EN 1993-1-8 cannot be applied.

Even if it is not explicitly specified in EN 1993-1-8, most of the design rules are only valid if the structural elements comply with the regulations specified in the execution/fabrication/erection standard EN 1090-2. Requirements are given for imperfections, geometric tolerances, e.g. tolerances on hole diameter for bolts and pins, methods of tightening of preloaded bolts, preparation of contact surfaces in slip resistant connections, welding plan and preparation and execution of welding, etc.

In this book, not specific focus is given to the fabrication and erection aspects. To know more about that, reference should be made to commentaries and background documentation of the execution standard EN 1090-2. But for sure all the design rules provided in this book may only be applied under the condition that the requirements of EN 1090-2 are satisfied. Similarly, all connected or connecting elements (bolts, welding materials, steel for end plates, etc.) are assumed to respect the EN product norms listed on the first pages of EN 1993-1-8.

1.5 COSTS

The price of a steel structure depends significantly on the fabrication, transportation and erection costs in countries where labour costs are the dominant factor. Hence, in order to design steel structures in an economical manner, special attention should be given to the joints which are known to be expensive. However, the joint detailing must also be chosen in such a way that the actual behaviour of the joint, i.e. its structural properties (strength, stiffness and ductility), is in line with the assumptions made when the joints have been modelled for the global frame analysis. In chapter 9, some strategies are presented which allow to minimise the effects of the joints on the global cost of steel structures.

1.6 APPLICATION OF THE “STATIC APPROACH”

The design of joints may, as for any other cross section, be performed on an elastic or a plastic basis. In a pure elastic approach, the joints should be designed in such a way that the generalised Von Mises stress does not exceed the elastic strength of the constitutive materials. For that purpose, and because of the geometric complexity of the connection elements, a refined stress analysis will be required, often requiring sophisticated numerical approaches such as finite element analysis. In this process, the presence of residual stresses or of any other set of self-equilibrated stresses which could, for instance, result from lack-of-fit, is usually neglected.

Steel usually exhibits significant ductility. The designer may therefore profit from this ability to deform steel plastically and so to develop "plastic" design approaches in which local stress plastic redistributions in the joint elements are allowed.

The use of plastic design approaches in steel construction is explicitly allowed in EN 1993. For joints and connections, reference is made to clause 2.5(1) of EN 1993-1-8. Obviously, limitations to the use of such plastic approaches exist. They all relate to the possible lack of ductility of the steel material, on the one hand, and of some connections elements like bolts, welds, reinforced concrete slab in tension or concrete in compression, on the other hand. As far as steel material itself is concerned, the use of normalised steels according to European standards, e.g. EN 10025, guarantees a sufficient material ductility. For potentially non ductile joint elements, EN 1993-1-8 introduces specific requirements which will be detailed later on when describing the design methods in dedicating chapters.

Clause 2.5(1) of EN 1993-1-8 is simply paraphrasing the so-called "static" theorem of the limit analysis. The application of this theorem to cross sections, for instance in bending, is well known as it leads to the concept of bi-rectangular stress patterns and to the notion of plastic moment resistance used for Class 1/2 cross sections (as no limit of ductility linked to plate buckling phenomena prevents these sections from developing a full plastic resistance). For connections and joints, the implementation of the static theorem is probably less direct, but this does not at all prevent designers from using it.

The static theorem requires first the determination of an internal statically admissible distribution of stresses (for cross sections) or forces (for connections and joints), i.e. of a set of stresses or forces in equilibrium with the external forces acting on the cross section or on the joint/connection and resulting from the global structural analysis. The second requirement is to be plastically admissible, which means that the plastic resistance and ductility criteria have to be met in the cross section or joint/connection.

A large number of statically and plastically admissible distributions may exist. In most cases, these will not respect the "kinematically admissible" criterion; in fact, only the actual distribution has the ability to satisfy the three requirements: equilibrium, plasticity and compatibility of displacements. But in fact, this is not a problem as long as sufficient local deformation capacity (ductility) is available at the places where plasticity develops in the cross section or in the joint/connection. As long as this last condition is fulfilled, the static theorem ensures that the predicted resistance of the cross section or joint/connection will be lower than the actual one (and therefore on the safe side). The closer the assumed distribution is to the actual one, the closer the estimated resistance will be to the actual resistance.

1.6.1 Component approach

1.6.1.1 General

The characterisation of the response of the joints in terms of stiffness, resistance and ductility is a key aspect for design purposes. From this point of view, three main approaches may be followed:

– Experimental;
– Numerical;
– Analytical.

The most practical one for the designer is the analytical approach. Analytical procedures enable a prediction of the joint response based on the knowledge of the mechanical and geometric properties of the so-called "joint components".

In this section the *component method* is introduced as a general analytical procedure. It applies to any type of steel joints, whatever the geometric configuration, the type of loading (axial force and/or bending moment) and the type of member cross sections.

The method is nowadays widely recognised, and particularly in EN 1993, as a general and convenient procedure to evaluate the mechanical properties of joints subjected to various loading situations, including static and dynamic loading conditions, fire, and earthquake.

1.6.1.2 Introduction to the component method

A joint is generally considered as a whole and studied accordingly; the originality of the component method is to consider any joint as a set of individual basic components. For the particular joint shown in Figure 1.9a (joint with an extended end plate connection mainly subject to bending), the relevant components (i.e. zones of transfer of internal forces) are the following:

– Column web in compression;
– Beam flange and web in compression;
– Column web in tension;
– Column flange in bending;
– Bolts in tension;
– End plate in bending;
– Beam web in tension;
– Column web panel in shear.

Each of these basic components possesses its own strength and stiffness either in tension, compression or shear. The column web is subjected a combination of compression, tension and shear. This coexistence of several components within the same joint element can obviously lead to stress interactions that are likely to decrease the resistance of the individual basic components.

The application of the component method requires the following steps:

1. *identification* of the active components in the joint being considered;
2. *evaluation of the stiffness and/or resistance characteristics* for each individual basic component (specific characteristics – initial stiffness, design resistance, deformation capacity – or whole deformation curve);
3. *assembly* of all the constituent components and evaluation of the stiffness and/or resistance characteristics of the whole joint (specific characteristics - initial stiffness, design resistance, rotation capacity - or whole deformation curve).

The assembly procedure is the step where the mechanical properties of the whole joint are derived from those of all the individual constituent components. That requires, according to the static theorem introduced in section 1.6.1, to define how the external forces acting on the joint distribute into internal forces acting on the components in a way that satisfies equilibrium and respects the behaviour of the components.

Guidelines on how to apply the component method for the evaluation of the initial stiffness and the design moment resistance of the joints are given in EN 1993-1-8; the aspects of ductility are also addressed. The application of the component method requires a sufficient knowledge of the behaviour of the basic components. Those covered for static loading by EN 1993-1-8 are listed in Table 1.2. The combination of these components covers a wide range of joint configurations and should be largely sufficient to satisfy the needs of practitioners. Examples of such joints are given in Figure 1.20.

Table 1.2 – List of components covered in EN 1993-1-8

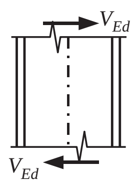
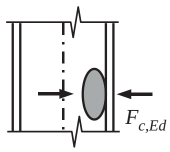
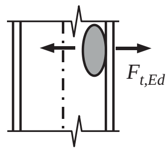
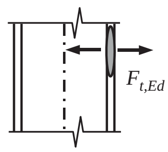
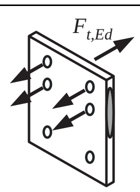
No	Component	
1	Column web panel in shear	
2	Column web in transverse compression	
3	Column web in transverse tension	
4	Column flange in bending	
5	End plate in bending	

Table 1.2 – List of components covered by Eurocode 3 Part 1-8, (continuation)

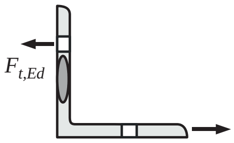
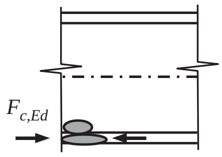
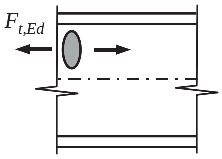
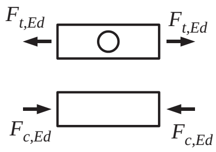
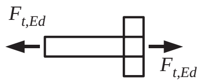
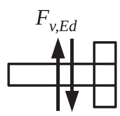
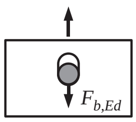
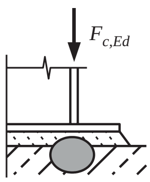
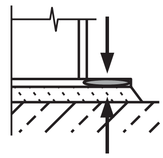
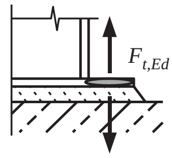
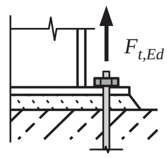
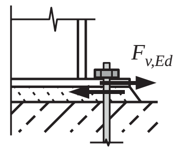
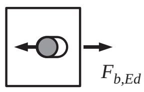
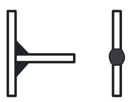
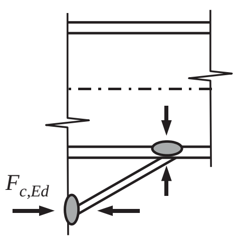
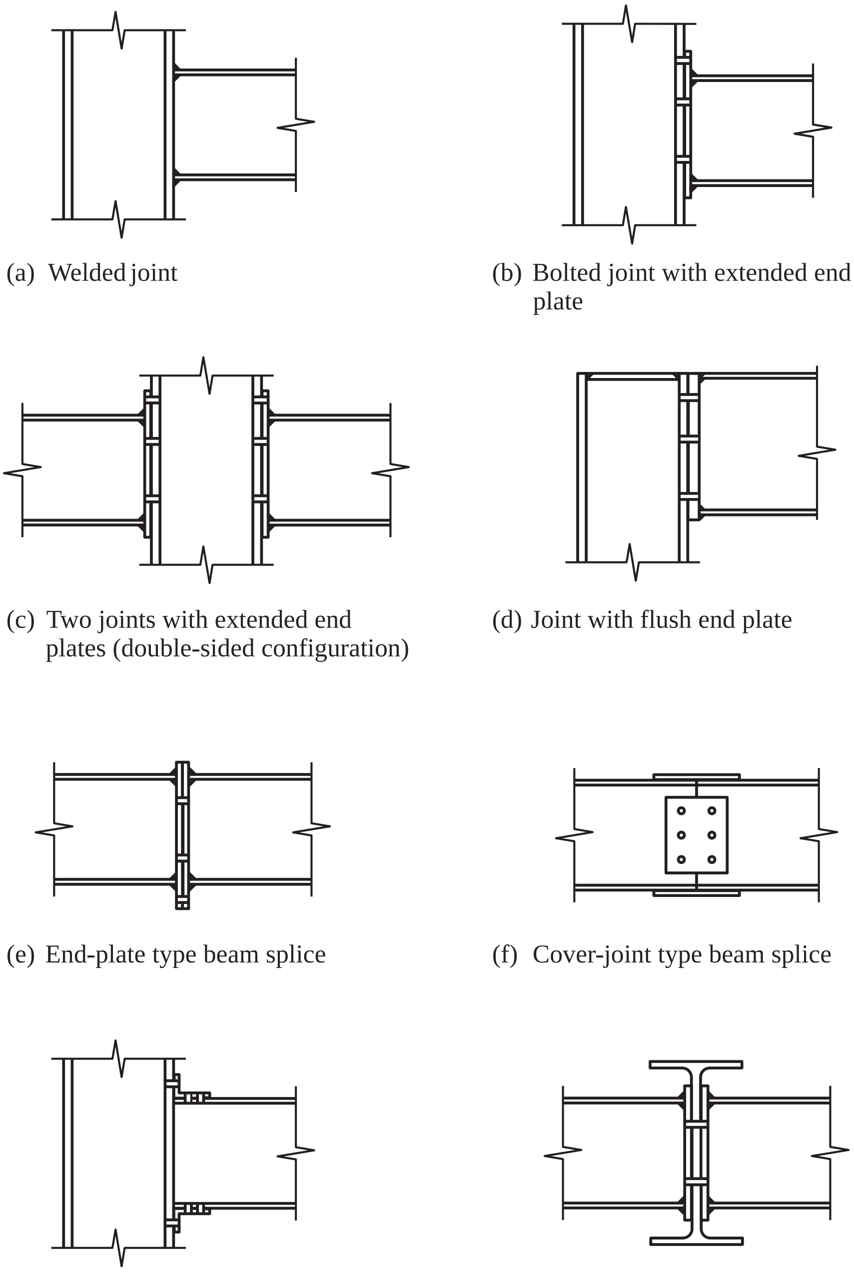
No	Component	
6	Flange cleat in bending	
7	Beam or column flange and web in compression	
8	Beam web in tension	
9	Plate in tension or compression	
10	Bolts in tension	
11	Bolts in shear	
12	Bolts in bearing (on beam flange, column flange, end plate or cleat)	
13	Concrete in compression including grout	

Table 1.2 – List of components covered by Eurocode 3 Part 1-8, (continuation)

No	Component	
14	Base plate in bending under compression	
15	Base plate in bending under tension	
16	Anchor bolts in tension	
17	Anchor bolts in shear	
18	Anchor bolts in bearing	
19	Welds	
20	Haunched beam	



(a) Welded joint

(b) Bolted joint with extended end plate

(c) Two joints with extended end plates (double-sided configuration)

(d) Joint with flush end plate

(e) End-plate type beam splice

(f) Cover-joint type beam splice

(g) Bolted joint with angle flange cleats

(h) Two beam-to-beam joints (double-sided configuration)

Figure 1.20 – Examples of steel beam-to-column joints, beam-to-beam joints and beam splices joints covered in EN 1993-1-8

UK Specific Comment

With reference to Figure 1.20 it is noted that (site) welded joints and angle flange cleats are rarely, if ever, used in the UK.

1.6.2 Hybrid joints aspects

In the present book, the design of welded and bolted connections is mainly considered. All the principles ruling the design of such joints are globally similar for hybrid joints in which the transfer of forces between two connected elements is commonly achieved by two different connectors, for instance welds and bolts.

Hybrid joints are however not widely used in practice and many experts in the field of joints are not recommending them, because of the different level of stiffness which often characterises their respective behaviours and which generally prevents the user to "add up" the contributions of the two connection systems to the resistance and so to profit from the higher expected joints resistance. For these reasons, hybrid joints will not be explicitly addressed here.

In EN 1993-1-8, few recommendations are given: "where fasteners with different stiffnesses are used to carry a shear load, the fasteners with the highest stiffness should be designed to carry the design load. As an exception, preloaded class 8.8 and 10.9 bolts in connections designed as slip-resistant at the ULS may be assumed to share load with welds, provided that the final tightening of the bolts is carried out after the welding is complete."

UK Specific Comment

Hybrid connections are rarely used in the UK for the reasons noted.

1.7 DESIGN TOOLS

EN 1993 gives new and advanced options to design efficient and economic steel structures. The design of joints plays a major role in that process. Thus the detailing of joints and the methods of considering the joints properties in the frame analysis will significantly influence the costs of a steel structure. This has been demonstrated by various investigations.

However, the exploitation of the advanced possibilities is rather time consuming for the designer if no appropriate tools for a quick and easy design are available. Different opinions have been discussed in Europe concerning the development of the Eurocodes. On one side, it is expected that the Eurocodes provide design methods which will allow safe, robust and economic solutions. Of course this requires more sophisticated approaches for the design rules. On the other side, the users of the Eurocodes are requesting simple codes for practice. But this is in conflict with the major request to make steel structures more economic. It would be unfortunate to make standards too simple as there is the loss of many possibilities to take advantage of the new and advanced options mentioned above.

The message is quite clear: there was and there is still a need for sophisticated standards which form an accepted basis to design steel structures. Based on the methods given in these standards simple design tools need to be developed and provided to practitioners. This is an optimal way to bring more economic solutions to the market with an acceptable effort needed by the designers. As it is often said: "*Not simple rules sell steel, but simple tools sell steel*."

1.7.1 Types of design tools

Beside the need for background information, the engineer requires simple design tools to be able to design joint in an efficient way. Three different types of design aids can be provided. The most appropriate type depends on various aspects.

- *Design tables* are ready-to-use tables containing standardised joint layouts including dimension details and all relevant mechanical properties like resistance, stiffness and ductility. The use of tables is certainly the quickest way to design a joint. However, any change in the layout will require further calculations and tables are no more helpful;
- *Design sheets* are sets of simple design formulae. The aim is to allow a simple and rather quick hand calculation. Due to simplifications, the results could be more conservative or the range of validity more limited. Both design tables and design sheets can be published in handbooks;

– The most flexible way is the use of *software*. Of course it takes a few minutes to enter all joint details, but there will be only few limitations in the range of validity, and any re-calculation, for example due to a change in the layout, is a matter of a few seconds.

1.7.2 Examples of design tools

In recent years, efforts have been made in various countries to develop specific design tools for joints. It is not practical to be exhaustive in the referencing of these tools. Therefore only a few characteristic examples (drafted in English) will be briefly described below to illustrate the variety of the available information sources.

The difficulties for the designer are to know about the existence of such tools, and to select the one which would be the most appropriate for his daily practice. This will require some investment in time, but this investment will be quickly repayed by an efficient and economical design of the joints, a quick fabrication in workshop and an easy erection on site.

– Design tables: as an example, the DSTV/DASt Ringbuch (Weynand and Oerder, 2013) can be mentioned here. The book is a publication of Stahlbau Verlags- und Service GmbH in Germany and is published in two languages, English and German. It covers a very wide range of moment-resisting and simple (pinned) joints in various joint configurations: beam-to-column, beam splices and beam-to-beam. Bolted connections with flush and extended end plates, with two or four bolts per row, header plates and web cleats are considered, as well as different bolt and steel grades. The book consists of design tables, which can be used in a straightforward manner to select a joint or, for instance, to check the resistance of a specified joint. They are provided for a selection of standard combinations of connected member sections and, in more details, provide the designer with the following data:

  - the detailing of the joint (geometric and mechanical properties of the sections and connection elements);
  - indications about requirements for stiffeners;
  - for simple joints: the design shear resistance of the joint;

- for moment-resisting joints: the design moment resistance under positive and negative bending moments and the shear resistance of the joint as well as the initial stiffness;
- an identification of the joint component which is governing the resistance moment;

Similar books are published in the UK as joint publications of the SCI, the British Constructional Steel Association (BCSA) and Tata Steel, for moment-resisting joints (Brown *et al*, 2013), and for simple joints (Moreno *et al*, 2011).

– Design sheets for a simple design of joints in accordance with EN 1993 was first published published by the European Commission (EC) (CRIF 1996). These sheets reference a EC-funded project, called SPRINT, in which they have been developed. Each individual set of sheet is devoted a particular joint configuration and connection type, see Figure 1.21.

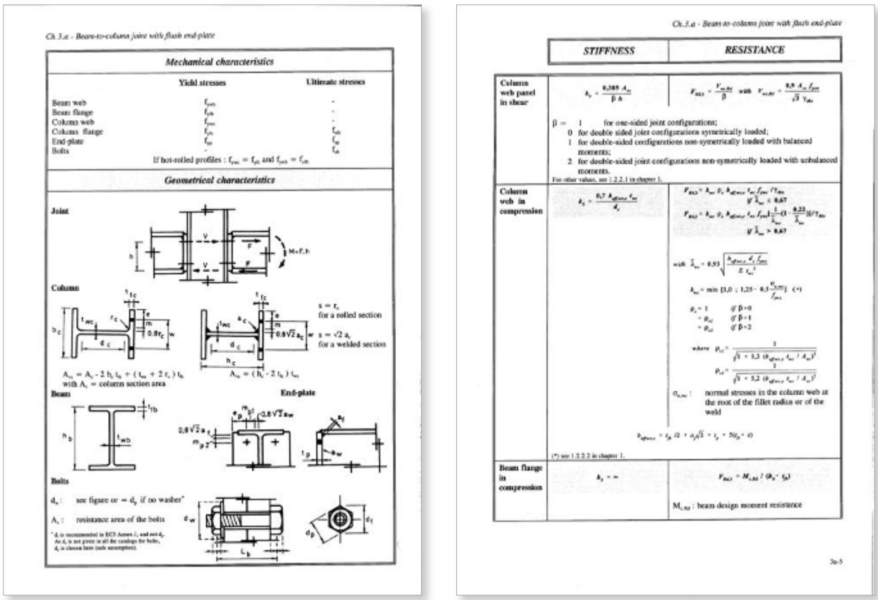


Figure 1.21 – Design sheets (CRIF 1996)

The joint design procedure included in this design tool is aimed at assisting the designer who wishes to take account of the full potential of semi-rigidity and/or partial-strength joints, without having to go through the more general but often complex approach provided by EN 1993-1-8. In reality, to derive these sheets, profit has been taken of all possible “shortcuts” allowed by the standards so as to safely but more easily perform the calculations. In each set, the first design sheet summarises all the data requested concerning the joint configuration and the connection type. In the remaining sheets, the calculation procedure first provides all the expressions for the evaluation of both stiffness and resistance for each of the joint components in a logical order, and finally shows how to derive the mechanical properties of the whole joint, i.e. the initial stiffness and/or the design moment resistance. The failure mode corresponds to the component whose resistance determines the design moment resistance of the joints. It gives an indication on the level of rotational capacity of the joint.

The shear resistance of the joint is an important value. For the sake of clarity, it is not dealt with in the design sheets but relevant information is provided just after the sheets.

In Maquoi and Chabrolin (1998), sheets are provided for beam splices and beam-to-column steel joints with end plates or flange cleats. In Anderson *et al* (1999), similar sheets are available for composite beam-to-column joints. Such sheets may be prepared for any joint configuration, connections types or loading situations. The designer may easily programme the sheet so as to develop its own set of sheets or even to establish his own design tables (i.e. corresponding to the particular need of his company).

– A number of software tools are offered on the market. As an example, the programme COP (Weynand *et al*, 2014), is mentioned here. COP is an innovative computer programme for the design of joints in steel and composite structures. The calculations are made in full accordance with the EN 1993-1-8 using the component method. The software fits perfectly with the needs of engineers and draughtsmen. All joint details are defined in clearly arranged data input masks. During input, all data will be visualised in scaled two-dimensional (2-D) or 3-D views and a data check module monitors if the requirements of EN 1993, such as bolt end distances, weld sizes, etc., are fulfilled, and, if needed, valid values are proposed. Four editions of COP are available for:

- Composite joints;
- Hollow section joints;
- Simple joints;
- Moment-resisting joints and simple joints.

The two first editions are freely available at http://cop.fw-ing.com. COP provides a full calculation note and it is available in English, German and French (output language independent of user interface language).

UK Specific Comment

There is not much use of CoP© in the UK. There are a number of other software tools on the market, some more comprehensive than others.

1.8 WORKED EXAMPLES

Worked examples represent another significant aid for designers as they start to apply new design approaches. However, in the field of joints, the variety of the joint configurations, connection types, connected elements and loading situations is such that a number of worked examples would be required to cover the whole range of European practical applications and design practices.

To be useful, a complete set of worked examples must cover the following:

- Bolted connections;
- Welded connections;
- Simple joints in buildings:
  - beam splice, beam-to-beam and beam-to-column joints;
  - column splices;
  - column bases;

– Moment-resisting joints in buildings:
  - beam splice, beam-to-beam and beam-to-column joints;
  - column splices;
  - column bases;
– Joints in lattice girders;

knowing that, in each category, several sub-categories should be considered according to the connected and the connecting elements and the loading situation.

In this book, a design example of a beam-to-column joint with an end plate connection with several bolt-rows and subjected to bending and shear is given in section 6.6.3. This case illustrates the components characterisation and assembly to derive stiffness and the resistance of the whole joint.

Several publications are available for other situations which can be found in practice. Hereunder, some are selected (non-exhaustive list) according to the two following criteria: easily available to designers and drafted in English.

– Bolted and welded connections (Veljkovic, 2015);
– Simple joints in buildings: beam splice, beam-to-beam and beam-to-column joints (Anderson *et al*, 1999; Moreno *et al*, 2011; Veljkovic, 2015);
– Simple joints in buildings: column splices (Brettle, 2009; Moreno *et al*, 2011);
– Simple joints in buildings: column bases (Brettle, 2009; Moreno *et al*, 2011; Veljkovic, 2015);
– Moment-resisting joints in buildings: beam splice, beam-to-beam and beam-to-column joints (Anderson *et al*, 1999; Brettle, 2009; Brown *et al*, 2013; Veljkovic, 2015);
– Moment-resisting joints in buildings: column splices (Brettle, 2009; Brown *et al*, 2013);
– Moment-resisting joints in buildings: column bases (Brettle, 2009; Brown *et al*, 2013; Veljkovic, 2015);
– Joints in lattice girders (Wardenier *et al*, 1991; Packer *et al*, 1992; Kurobane *et al*, 2004; Wardenier *et al*, 2008; Brettle, 2008; Packer *et al*, 2009).