Analysis and Design of Stainless Steel Bolted Connections

A thesis submitted to the Imperial College London
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By

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**ABSTRACT**

The use of stainless steel in construction is steadily growing, with applications designed to exploit its structural properties, durability, appearance and fire resistance. The mechanical behaviour of stainless steel is fundamentally different from that of carbon steel. The stress-strain curve of stainless steel is rounded without a well-defined yield stress and exhibits significant strain hardening at relatively small strains. Nevertheless, design provisions for bolted connections between stainless steel structural members in current international standards are essentially based on the rules for carbon steel with some very limited modifications. As the connections form an essential part of all structural assemblages, a comprehensive understanding of their behaviour is vital for efficient design and consequently better performance of structures. For this reason, an investigation into the behaviour of stainless steel bolted connections has been carried out so as to better understand the response of these structural components.

Suitable available test data have been reviewed and replicated using numerical models in order to study the behaviour of lap bolted connections and gusset plate connections in stainless steel under static tensile load. Strain-based criteria were defined to identify three failure modes: net section rupture, bolt shear and bearing failure. The developed FE models were successfully validated against the test results, after which they were employed to meticulously investigate the behaviour of bearing and net section rupture of lap bolted connections, as well as the net section failure of single angles connected to gusset plates.

The results demonstrated that the response of stainless steel connections has some different aspects from that of carbon steel. The findings have been used to revise the design rules for net section and bearing capacities in Eurocode 3 Part 1.4. These proposed rules take into account the particular mechanical characteristics of stainless steel and therefore offer an improvement to those currently available.
Declaration:

I confirm that this thesis is my own work and that any material from published or unpublished work from others is appropriately referenced.

Signature: …………………..
ACKNOWLEDGEMENT

This thesis is dedicated to my respectful father, mother, wife and my lovely daughter Minna.

First of all I thank our generous Allah for all his great blessings and I supplicate to him to shower mercy upon our beloved brother Sohayb and to accept him with the righteous slaves.

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NOTATION

- $\alpha$: Bearing coefficient
- $\alpha_b$: The smallest of $\alpha_d$, $f_{ub}/f_u$ or 1.0
- $\alpha_d$: Bearing coefficient in Eurocode 3 depends on the distance between bolts in the direction of loading
- $\alpha_{rt}$: Weighting factor for $Q_{rt}$
- $\alpha_\delta$: Weighting factor for $Q_\delta$
- $\alpha_1$, $\alpha_2$, $\alpha_3$: Proposed bearing coefficients
- $\beta_{b,\text{def}}$: Bearing coefficient from the FE models by adopting deformation criterion
- $\beta_{b,\text{frac}}$: Bearing coefficient from the FE models by adopting strength criterion
- $\delta$: Error term
- $\delta_i$: Observed error term for test specimen $i$
- $\Delta$: Average value of $\Delta_i$
- $\Delta_i$: Logarithm of the error term $\delta_i$
- $\varepsilon$: Strain
- $\varepsilon_{0.2}$: Total strain at the 0.2% proof stress $\sigma_{0.2}$
- $\varepsilon_{\text{frac,area}}$: True fracture plastic strain based on the reduction in cross-sectional area
- $\varepsilon_{\text{frac,elong}}$: True fracture plastic strain based on the elongation in specimen gauge
- $\varepsilon_{\text{true}}$: True plastic strain
- $\varepsilon_{\text{nom}}$: Nominal strain
- $\varepsilon_u$: Total strain at the ultimate stress $\sigma_u$
- $\phi_{\text{bear}}$: Resistance factor against bearing in ASCE (2002) and AS/NZ (2001)
- $\phi_{\text{end}}$: Resistance factor against end tear-out in ASCE (2002) and AS/NZ (2001)
\( \phi_{\text{net}} \)  Resistance factor against net section rupture in ASCE (2002) and AS/NZ (2001)

\( \phi_t \)  Resistance factor against net section rupture in AISC (2005)

\( \gamma_{M2} \)  Partial safety factor against fracture in Eurocode 3

\( \lambda \)  Reduction factor suggested by Wu and Kulak (1993)

\( \mu \)  Coefficient of friction

\( \theta \)  Angle

\( \sigma \)  Stress

\( \sigma_b \)  Nominal bearing stress

\( \sigma_{b,\text{def}} \)  Nominal bearing stress from the FE models by adopting deformation criterion

\( \sigma_{b,\text{frac}} \)  Nominal bearing stress from the FE models by adopting strength criterion

\( \sigma_{\text{frac,true}} \)  True fracture stress

\( \sigma_{\text{nom}} \)  Nominal stress

\( \sigma_{\text{true}} \)  True stress

\( \sigma_u \)  Ultimate stress

\( \sigma_{u,\text{true}} \)  True ultimate stress

\( \sigma_{0.2} \)  0.2% proof stress

\( \tau_{\max} \)  Maximum shear stress

\( \tau_u \)  Ultimate shear strength

\( b \)  Correction factor

\( b_w \)  Width of the connected plate

\( c_1, c_2, c_3 \)  Numerical constants

\( d \)  Bolt diameter

\( d_0 \)  Bolt hole diameter

\( d_1 \)  Spacing between bolts

\( e_1 \)  End distance

\( e_2 \)  Edge distance

\( f_{\text{net}} \)  Nominal stress over critical cross-section at net section failure

\( f_t \)  Nominal tension stress in ASCE (2002) and AS/NZ (2001)
NOTATION

$\mathbf{f_u}$ Material ultimate tensile strength of plates

$\mathbf{f_{ub}}$ Material ultimate tensile strength of bolts

$\mathbf{f_{ur}}$ Reduced ultimate strength obtained from tests failing by bearing

$\mathbf{f_{u,red}}$ Reduced ultimate strength proposed by the SCI/Euro Inox Design Manual (1996)

$\mathbf{f_y}$ Material yield strength

$\mathbf{g_{r(t)(X)}}$ Resistance function of the basic variables $X$ used as a design model

$\mathbf{h}$ Length of cover plate

$\mathbf{j}$ Total number of independent variables

$\mathbf{k_{d,n}}$ Design fractile factor

$\mathbf{k_{d,\infty}}$ Design fractile factor for $n = \infty$

$\mathbf{k_r}$ Reduction factor for net section capacity

$\mathbf{k_{r,Winter}}$ Reduction factor proposed by Winter (1956)

$\mathbf{k_1}$ Bearing coefficient in Eurocode 3 depends on the distance between bolts in the direction perpendicular to loading

$\mathbf{m_s}$ Strain hardening exponent

$\mathbf{n}$ Number of experiments or numerical test results

$\mathbf{n_b}$ Number of bolts at the critical cross-section

$\mathbf{n_{line}}$ Number of bolts per line in a gusset plate connection

$\mathbf{n_s}$ Strain hardening exponent

$\mathbf{p}$ Strain hardening exponent

$\mathbf{p_1}$ Spacing between bolts in the direction of loading

$\mathbf{p_2}$ Spacing between bolts in the direction perpendicular to loading direction

$\mathbf{q_1}, \mathbf{q_2}$ Numerical constants

$\mathbf{r}$ Ratio between the number of bolts at the cross section to the total number of bolts in the connection

$\mathbf{r_{area}}$ Ratio of gross cross-sectional area of the unconnected leg to net cross-sectional area of the connected leg

$\mathbf{r_d}$ Design value of the resistance

$\mathbf{r_e}$ Experimental resistance value

$\mathbf{\bar{r_e}}$ Mean values of the experimental resistance

$\mathbf{r_{ei}}$ Experimental resistance for specimen i

$\mathbf{r_n}$ Nominal value of the resistance
**NOTATION**

- \( r_t \): Theoretical resistance determined from the resistance function \( g_m(X) \)
- \( \bar{r}_t \): Mean values of the theoretical resistance
- \( r_{ti} \): Theoretical resistance determined using the measured parameter \( X \) for specimen \( i \)
- \( s_\Delta \): Variance of error term
- \( t \): Plate thickness
- \( t_c \): Thickness of the connected-leg of an angle
- \( x \): Connection eccentricity

- \( A \): Width of the connected leg of an angle
- \( A_0 \): Original cross-sectional area of tensile coupon specimen
- \( A_f \): Minimum cross-sectional area after fracture of tensile coupon specimen
- \( A_{gross} \): Gross cross-sectional area
- \( A_{g,smaller} \): Gross cross-sectional area of the smaller leg of an angle
- \( A_{g,unc} \): Gross cross-sectional area of the unconnected leg of an angle
- \( A_{net} \): Net cross-sectional area
- \( A_{net,eff} \): Effective net cross-sectional area
- \( A_{net,uneq} \): Net section area of unequal-leg angle connected with its smaller leg as recommended in EN 1993-1-4 (2006) and SCI/Euro Inox (2006)
- \( A_{n,con} \): Net cross-sectional area of the connected leg of an angle
- \( A_{red} \): Reduced net cross-sectional area
- \( B \): Width of the unconnected leg of an angle
- \( C \): Upper constant value of the bearing coefficient \( \alpha \)
- \( E \): Initial Young’s Modulus
- \( E_{0.2} \): Tangent stiffness at the 0.2% proof stress
- \( F_{b,def} \): Bearing resistance per bolt from the FE models by adopting deformation criterion
- \( F_{b,frac} \): Bearing resistance per bolt from the FE models by adopting strength criterion
- \( F_{b,3.0} \): Load corresponding to 3.0 mm hole elongation
- \( F_{net} \): Net section failure load
- \( F_{net,FE} \): Net section failure load obtained from obtained from FE model
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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Stainless steel is a general term used to describe iron alloys that contain a minimum of 10.5% chromium. In addition to chromium, stainless steel contains many other alloying elements. These include, for example, carbon, manganese, nickel, silicon, copper, molybdenum, nitrogen and phosphorus. Chromium is the most important element because it is responsible for the excellent corrosion resistance of stainless steel by creating a thin surface film when exposed to an oxidising environment. The chemical elements in different grades of stainless steel are listed in the European Standard EN 10088-2 (2005).

1.2 MATERIAL GRADES AND MECHANICAL PROPERTIES

There are various grades of stainless steel which are created through variation in chemical composition. Stainless steel grades can be classified into five basic groups, according to their metallurgical structure. These are the austenitic, ferritic, martensitic, duplex and precipitation-hardening groups. The most common grades for structural applications are the austenitic and duplex. Austenitic stainless steels typically contain 17-18% chromium and have good corrosion resistance properties. Duplex stainless steels
typically contain 22-23% chromium and exhibit high strength and wear resistance.

The mechanical behaviour of stainless steel material is fundamentally different from that of carbon steel. The most important aspect of this for structural applications is the rounded uniaxial stress-strain curve, meaning that there is no sharp yield point and that the material exhibits significant strain hardening, as can be seen in Figure 1.1. Moreover, the ductility of stainless steel is much higher than that of carbon steel, which is manifest in the larger percentage elongation at fracture, approximately 40-60%, and the higher values of tensile-to-yield strength ratios for stainless steel material.

![Figure 1.1: Indicative stress-strain curves for carbon steel and stainless steel](image)

The most desirable property of stainless steel is its corrosion resistance, enabling the material to retain its appearance and integrity for long periods of exposure with a minimum maintenance. In addition, stainless steel has other superior characteristics. For example, the performance of stainless steel at elevated temperatures is better than that of carbon steel, since stainless steel provides better retention of stiffness and strength at temperatures above 500° C than carbon steel (Gardner, 2005). As a result, structural elements generally retain their load carrying capacity for a longer time when exposed to fire. This characteristic of performance in fire coupled with the good energy absorption and high ductility of stainless steel makes it an ideal material for structures vulnerable to
explosion. A detailed explanation of the benefits offered by the material and its usage in structures has been reported by Mann (1993) and Gardner (2005).

1.3 USAGE OF STAINLESS STEEL IN CONSTRUCTION

All these advantageous properties have attracted architects and engineers to adopt stainless steel as a structural material in certain circumstances. The early use of stainless steel was largely limited to facades and roofing with occasional use in load-bearing applications. For example, the Chrysler building in New York, which was completed in 1930 and has since become one of the New York’s landmark structure, was covered with stainless steel cladding as shown in Figure 1.2(a). Another example of an early usage of stainless steel in construction is the Gateway arch in St. Louis, Missouri. Its construction started in 1963 and was completed in 1965. The cross-section of the arch, which is shown in Figure 1.2(b), is an equilateral triangle of carbon steel on the interior and stainless steel on the exterior, held together by welded steel rods.

(a) The Chrysler building – New York, 1930
( Photo: aviewoncities.com)

(b) The Gateway arch – St. Louis, Missouri, 1965 (Photo: ArtToday.com)

Figure 1.2: Examples of early usage of stainless steel in construction
Although the usage of stainless steel in structures still only represents a small fraction of that of conventional carbon steel, stainless steel is steadily growing in popularity owing to its foremost properties discussed above. Interestingly, although high initial cost is its major drawback, recent developments suggest that low nickel content stainless steel referred to as lean duplex, can provide many of the durability related benefits of the more traditional grades but at around half the cost – thereby further encouraging its adoption in appropriate circumstances (Theofanous and Gardner, 2009).

The Millennium footbridge in York, United Kingdom, which was erected in 2001, is a clear example of the recent increase in using stainless steel in large scale structures as shown in Figure 1.3(a). The Helix Bridge in Paddington train station, London, is another example of a recent stainless steel structure (see Figure 1.3(b)).

(a) The Millennium Bridge, York, UK, 2001 (Photo: Klaus Föhl)

(b) The Helix Bridge, London, 2004 (Photo: Buro Happold)

**Figure 1.3:** Recent stainless steel structures
1.4 STAINLESS STEEL CONNECTIONS

Connections are a vital component in steel construction. For stainless steel structures, since the connection zones are often the most susceptible areas to corrosion due to the fine cracks that develop during the drilling of holes for bolts, the design of connections needs careful attention to ensure a good corrosion resistance performance. The SCI/Euro Inox Design Manual for structural stainless steel (2006) gives many recommendations regarding connections. For instance, it is recommended to avoid connections that include carbon steel or any other metallic material, in order to prevent galvanic corrosion, otherwise precautions should be made by insulating the carbon steel and stainless steel using non-metallic washers, gaskets or bushes. Furthermore, the document advises avoiding the use of carbon steel bolts with stainless steel elements at all.

Research on the behaviour of stainless steel members, covering local and member buckling (Ashraf et al., 2008) has shown that explicit recognition of stainless steel’s stress-strain behaviour leads to significant improvements in load carrying capacity. Moreover, concepts such as cross-sectional classification and effective width of slender plate elements have been shown to be inappropriate (Ashraf et al., 2006), leading to the development of more suitable treatments; some of these e.g. the continuous strength method for determining cross-sectional strength, have subsequently been shown to be advantageous when dealing with carbon steel (Gardner, 2008). This noticeable progress in understanding the performance of stainless steel members, which has led to significant improvements in their design rules, has not been accompanied by comparable work on stainless steel joints – research conducted on stainless steel connections being somewhat limited. This shortcoming has considerably overshadowed the understanding of the behaviour of stainless steel connections and consequently the efficiency of the existing design provisions for these structural components.

1.5 OUTLINE OF THE THESIS

An introduction to stainless steel, including its various grades and characteristics has been presented in this chapter. A summary of the subjects discussed in each chapter in this thesis is also presented. In Chapter 2, a review of previous studies of both carbon
steel and stainless steel connections relevant to the current research project has been carried out.

Chapter 3 presents the development and validation of sophisticated 3-D numerical models for bearing-type bolted connections with two types of stainless steel materials: austenitic and ferritic. A material model that describes the nonlinearity of stainless steel is employed. All the essential features of these types of connection have been successfully modelled. The numerical models have been validated against the existing test results.

An investigation into the behaviour of the net section rupture of lap bolted connections is conducted in Chapter 4. The validated finite element models were utilised to generate further results to study the influence of the key parameters on the net section capacity. These results are used to modify the existing design equation.

In Chapter 5, the bearing behaviour of lap bolted connections is investigated on the basis of the results of parametric studies. Similarities and differences between the bearing response of carbon steel and stainless connections are investigated. Simple design provisions are devised by adopting a more rational bearing failure criterion.

The net section failure of stainless steel single angles under pure tension force is studied in Chapter 6. Firstly, numerical models are used to replicate existing test data. After validating these models, the net section resistance of these structural elements is investigated, and finally design provisions are presented. Chapter 7 summarises the key findings and identifies topics in need for further study.
CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter reviews the most relevant studies for bolted connections between carbon steel elements as well as stainless steel elements. This has enabled similarities and differences between the behaviour of bolted connections made of these types of steel to be identified. A brief overview of the previous experimental and numerical studies pertinent to the subjects covered in this thesis has been presented. Further, more detailed, examination of particular aspects of these studies is made where appropriate in the following chapters.

2.2 TYPES OF BOLTED LAP CONNECTIONS

According to the load transfer mechanism, bolted lap connections are classified into two main types: slip-resistant connections and bearing connections.

2.2.1 Slip-resistant connections

The load in this type of connection is initially entirely transferred by the frictional forces between the connected parts at their contact surfaces. Once a slip occurs, the behaviour
CHAPTER 2 – LITERATURE REVIEW

is as for bearing-type connections. These frictional forces are developed as a result of the pressure at the interfaces that arises from the bolt tightening. Hence, in this type of connection, the bolt is stressed neither in shear nor in bearing until the slip resistance is exceeded. This kind of connection is suitable in situations where slip has undesirable effects on the connection, for example in structures subjected to reversal of loads, where fatigue effects take place. The slip load of the joint depends on the coefficient of friction between the connected parts and the bolt preload. Figure 2.1(a) illustrates the load transfer mechanism in this type of connections.

2.2.2 Bearing-type connections

In this type of connection, the load is mainly transferred by means of shear over the bolt cross-section and bearing between the bolt shank and the inner surface of the bolt hole. The bolts need only to be tightened to a snug-tight condition, where a small amount of bolt preload is required so that the connected plates are in firm contact. As a result, initially a small portion of the applied load is resisted by frictional forces, and when the load exceeds these forces, a major slip occurs and the bolt hole comes into bearing against the bolt shank, thereafter the load is carried by shear and bearing. In most situations, the structures designed with this kind of connection are tolerant to such slips. The current study focuses on bearing-type connections between stainless steel plates. The internal forces developed in this type of connection are shown in Figure 2.1(b).

For these types of connection three fundamental modes of failure are possible, as shown in Figure 2.2, depending on the relative geometrical and material strengths of the components, as follows:

1. Bolts fail in shear.
2. Plates fail in tension at net section.
3. Bolts or plates fail in bearing.

It should be noted that the failure of bolts in bearing is only possible if very high strength plates and low strength bolts are used, which is very unlikely with practical arrangements.
In present study, two failure modes of bearing-type connections between stainless steel are considered: tension failure at the net section and plate bearing failure.

Figure 2.1: Load transfer mechanism in bolted lap joints

Figure 2.2: Schematic representation of failure modes of bearing-type connections
2.3 PREVIOUS STUDIES ON BOLTED LAP CONNECTIONS

2.3.1 Carbon steel connections

2.3.1.1 Experimental studies

The bearing behaviour of connections between cold-formed steel has been investigated in a number of studies. Winter (1956) conducted a large number of tests on bolted connections in light-gauge cold-formed carbon steel. A wide range of variables was investigated. Two forms of bearing failure were observed. The first was described as a longitudinal shearing of the sheets along two parallel planes whose distance equals the bolt diameter – this form is usually referred to as end tear-out. This mode occurred for connections with relatively small end distances. The second bearing failure was described as a tearing along two distinctly inclined planes with considerable piling-up of material in front of the bolt – this form is called bearing failure. This type of failure took place in connections with large end distances. By adopting the maximum load attained during testing as the bearing failure, a bearing design equation was proposed. A single equation that relates the connection bearing resistance to the sheet material yield strength $f_y$ and the end distance $e_1$ was proposed. Winter’s (1956) equation was modified by Dhalla et al. (1971). The same two bearing failure forms were observed by Dhalla et al. (1971). In their equation, the bearing resistance was related to the material ultimate tensile strength $f_u$ instead of the material yield strength $f_y$. Recently, following an experimental investigation, Rogers and Hancock (1998, 1999) proposed bearing design provisions. A bearing coefficient method that depends on the ratio of bolt diameter to sheet thickness $d/t$ was proposed. To develop their equation, the maximum loads corresponding to end tear-out and bearing failures, typical of those observed by Winter (1956), were used to define bearing resistance regardless of the associated amount of deformation. Similar equations were proposed by Wallace et al. (2001) by adopting the approach of Rogers and Hancock (1998, 1999).

A series of experimental investigations of lap bolted connections were carried out at Imperial College London between 1976 and 1980. In these studies, the bearing behaviour of different connected steel grades and grade 8.8 bolts was examined and design equations for bearing were recommended.
Owens et al. (1976a) also defined a ‘bursting’ bearing failure mode in which fracture initiates transversely at the outer edge of the plate in front of the bolt. The influence of the pitch on the bearing resistance of a multi-bolt connection was also investigated by Owens (1980). A significant increase in the bearing strength was observed when the pitch increased from 2.5 times the bolt diameter to 3.0 times the bolt diameter. This was attributed to the fact that the adverse interaction between neighbouring bolts is reduced.

The bearing capacity of bolted connections between high strength steel plates was studied experimentally by Puthli and Fleisher (2001). The bearing resistance of the connections, defined as the ultimate load, was compared to that given in EN 1993-1-8 (2005). It was concluded that the design provisions for bearing failure in Eurocode 3 for hot-rolled carbon steel could be extended to cover high strength steel connections.

Defining bearing failure by limiting hole deformation was initially suggested by Perry (1981) for hot-rolled steel connections. Perry (1981) investigated the bearing behaviour of test specimens by examining the load-deformation relationship. It was observed that beyond 6.35 mm (0.25 inch) deformation, the load-deflection curves were virtually flat and the connections had almost reached their ultimate capacities. Hence, it was suggested that at 6.35 mm deformation the bearing failure is said to have occurred. This bearing definition was adopted in the development of the bearing design equation in the AISC Specification (2005). LaBoube et al. (1995) derived a bearing design equation for connections between thin gauge sheets that limit the bolt hole elongation to 6.35 mm. Kim (1996) also defined bearing failure by adopting this deformation criterion when hot-rolled carbon steel connections were tested. The bearing strengths obtained by Kim (1996) showed a wide scatter when normalized by $f_y$ but there was much less scatter when normalized by $f_u$. In previous studies at Imperial College London (Owens et al.; 1976a and 1980), a deformation of 1.0 mm at serviceability state was considered to be acceptable.

Many researchers have conducted experimental studies to investigate the net section resistance of cold-formed carbon steel connections. Winter (1956) concluded that for connections between wide sheets, which failed in the net section, the ductility of the cold-formed steel was insufficient to overcome the deleterious effects of stress concentration. For these connections, the nominal stress over the net section at failure
was found to be less than the material ultimate tensile strength. A factor that is dependent on the sheet width to reduce the material tensile strength was applied to the net section resistance. This reduction factor was confirmed by the tests conducted by Dhalla et al. (1971).

Later, Popowich (1969) suggested a modification to Winter’s (1956) reduction factor. Popowich (1969) recognised that there are two sources of stress concentration near the bolt hole. In addition to the well-known source that is produced by the existence of the bolt hole, which was observed by Winter (1956), Popowich (1969) suggested that a stress concentration is also generated by the fact that the bolt force is applied to the plate locally at the hole. Popowich’s (1969) factor takes into account the number of bolts in a line in the direction of the applied load in addition to the width of the connected sheet. His recommendation forms the basis of the design provisions in EN 1993-1-3 (2006), the AISI Specification (1996) and the AS/NZS Specification (2001). A detailed investigation into the usage of the reduction factor in the net section resistance in ENV 1993-1-3 (1996) and the AISI Specification (1996) was carried out by Rogers and Hancock (1998, 1999). Bolted connections between low-ductility sheets with a maximum ultimate-to-yield ratio of 1.13 and sheet thicknesses less than 1.0 mm were tested. Moreover, all the previous tests used to propose the reduction factor were reviewed. Rogers and Hancock (1998, 1999) suggested that the net section failure in many of these tests was misidentified. It was concluded that the provisions for the net section failure in these design standards are conservative due to the presence of the reduction factor. A design equation for net section rupture that does not contain a reduction factor was proposed. Their design proposal for net section resistances has been included in the most recent version of the AISI Specification.

2.3.1.2 Numerical studies

Up until about 1980, before the advancement in the computational capability of computers and the availability of sophisticated finite element software, numerical simulations of structural elements was somewhat complicated, particularly, for bolted connections because of the difficulties in modelling the interfaces between the different components. Krishnamurthy and Graddy (1976) were pioneers in the 3-D modelling of bolted connections. The material was assumed to be elastic, but the analysis was still
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computationally expensive because the contact conditions were approximated by attaching and releasing appropriate nodes at each loading step. Nowadays, most programs contain algorithms that use contact elements at interfaces. These algorithms are able to detect when two surfaces are in contact or not, thereby contact constraints are applied or removed accordingly.

Many researchers have used 3-D models to simulate lap shear connections by introducing contact elements at the interfaces (Fan et al., 1997; Chung and Ip, 2000; Ju et al., 2004). Fan et al. (1997) explained that three-dimensional solid (brick) elements with eight nodes and one integration point perform better than the three-dimensional shell elements. This was attributed to the limitation of the shell elements in the computation of the large inelastic strains that occur in the vicinity of the holes. Moreover, due to the effect of the bolt preloading, in the area underneath the bolt nut and the bolt head the sheet was subjected to transverse stresses that are not defined in the shell elements. Similarly, Chung and Ip (2000) and Ju et al. (2004) employed three-dimensional solid elements with eight nodes for all connection components. They found that this type of element was suitable for the modelling of bolted connections including large inelastic strains.

Bolt preload has been incorporated into FE models using different techniques. In some studies (Fan et al., 1997; Chung and Ip, 2000; Ju et al., 2004) the bolt preload was simulated in the first step. In this case, the researchers modelled the bolt as being shorter than its actual length by a predefined amount, as a result, the bolt head or the bolt nut initially penetrated the side of the plate. During the first step, the plate pushes the penetrated part away due to the presence of contact elements and consequently, a tension force was developed in the bolt. In the following step, the external loading was applied.

Friction forces that developed between the connected parts depend on the coefficient of friction of the surfaces. These friction forces play an important role in slip-resistant connections. A value of the classical Coulomb friction coefficient from 0.1 to 0.25 has been used in most studies. For example, Fan et al. (1997) used 0.13 while Chung and Ip (2000) and Ju et al. (2004) used 0.2.
2.3.2 Stainless steel connections

2.3.2.1 Experimental studies

A limited number of studies have been carried out to investigate the behaviour of stainless steel bolted connections. The first experimental work was conducted a third of a century ago by Errera et al. (1974). The main objective of their study was to assess the validity of the design equations for cold-formed carbon steel bolted connections, which had been proposed by Winter (1956), for stainless steel connections. Four fundamental types of failure were observed – end tear-out, bearing, net section and bolt shear failures – identical to those previously reported by Winter (1956) for cold-formed carbon steel. It was concluded that Winter’s (1956) design equations for bearing resistance were valid, without modifications for cold-formed stainless steel connections. Van Der Merwe (1987) compared his test results with the design equations given in the AISI Specification (1986) for cold-formed carbon steel. It was concluded that these equations predict tear-out capacity satisfactorily, but overestimate the bearing capacity of stainless steel connections.

The Steel Construction Institute (1991) conducted an experimental study of stainless steel bolted connections of grades 1.4306, 1.4404 and 1.4462 to develop design equations. Some 31 bolted connection specimens in single shear were tested. On the basis of the results of these tests, the design equations for the bearing capacity of carbon steel given in EN 1993-1-8 (2005) were adopted with a slight modification.

For net section behaviour, Errera et al. (1974) concluded that the design equation for the net section resistance which was developed by Winter (1956) for cold-formed carbon steel is suitable for stainless steel connections. Van Der Merwe (1987) showed that the design equations given in the AISI Specification (1986) for cold-formed carbon steel predicted the net section rupture of stainless steel connections accurately. On the basis of the results of the tests conducted by the Steel Construction Institute (1991), the net section design equation in ENV 1993-1-3 (1996) was adopted (SCI/Euro Inox, 1996) with a modification – a constant factor 0.9, which is also present in EN 1993-1-8 (2005), was introduced.
The use of stainless steel for main structural components in buildings and bridges – which requires larger thicknesses of material has encouraged researchers to focus on connections between thicker hot-rolled plates. As a part of a programme sponsored by the European Coal and Steel Community (ECSC) to promote the use of stainless steel in construction, an extensive experimental programme on different types of bolted connections was performed by Ryan (1999a). Lap connections made from three types of stainless steel – austenitic grade 1.4306, ferritic grade 1.4016 and duplex grade 1.4462 formed the basis for revising the design provisions in ENV 1993-1-4 (1996). Although Ryan’s (1999a) results suggested removal of the 0.9 reduction factor in the net section design formula for hot-rolled stainless steel, he recommended maintaining the factor \( k_r \) until such time that the number of tests provided full justification. However, it was concluded that the bearing resistance in ENV 1993-1-4 (1996) is safe. Because these set of tests constitute the basis of the FE validation in the current research project, a detailed description of the specimen geometry, material properties and failure loads is postponed until Chapter 3.

Kuwamura and Isozaki (2001a, 2001b) performed laboratory tests on connections between cold-formed austenitic stainless steel grade 1.4301. A total of 90 connections with plate thicknesses of 1.5 or 3.0 mm were tested. These tests were then used to validate numerical models (Kim and Kuwamura, 2007); subsequently parametric studies were performed by Kim et al. (2008).

<table>
<thead>
<tr>
<th>Reference</th>
<th>Type of connected plates</th>
<th>No. of specimens</th>
<th>Specimens description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Errera et al. (1974)</td>
<td>Cold-formed</td>
<td>25</td>
<td>Single and double shear</td>
</tr>
<tr>
<td>Van Der Merwe (1987)</td>
<td>Cold-formed</td>
<td>66</td>
<td>Single and double shear</td>
</tr>
<tr>
<td>The Steel Construction Institute (1991)</td>
<td>Cold-formed</td>
<td>31</td>
<td>Single shear</td>
</tr>
<tr>
<td>Ryan (1999a)</td>
<td>Hot-rolled</td>
<td>36</td>
<td>Double shear</td>
</tr>
<tr>
<td>Kuwamura and Isozaki (2001a, 2001b)</td>
<td>Cold-formed</td>
<td>90</td>
<td>Single shear</td>
</tr>
</tbody>
</table>
2.3.2.2 Numerical studies

To investigate the behaviour of stainless steel connections, a number of numerical studies have been conducted. Kim and Kuwamura (2007) investigated bolted connections between cold-formed stainless steel sheets. They employed the ABAQUS package to model previously tested specimens (Kuwamura and Isozaki, 2001a and 2001b). Only the thinnest plate of the single shear connection was modelled and the bolts were simulated as rigid. The developed FE models were then used by Kim et al. (2008) to perform a parametric study to investigate the influence of out-of-plane deformation of the connected sheet (curling) on the connection’s capacity. A reduction of 4% to 25% in ultimate capacity of the models in which curling was allowed when compared to those restrained against curling was observed. Design equations were proposed for net section and bearing failures for thin gauge stainless steel sheets susceptible to curling.

Bouchair et al. (2008) employed the finite element package ABAQUS to initially replicate experiments on stainless steel lap connections. The Ramberg-Osgood material model was adopted (Ramberg and Osgood, 1943). The elongation of the connection was obtained by combining the responses of each component after modelling each part of the connection individually. A comparison between the load-deformation curves obtained from the tests and FE models showed good agreement. However, the ultimate capacities of the connections were not obtained. They recommended that the reduction factor \( k_r \) in the net section design equation and the influence of the end and edge distances on bearing behaviour require further investigation. Kiymaz (2009) investigated the bearing behaviour of austenitic grade 1.4301 and duplex grade 1.4462 stainless steels. The Ramberg-Osgood material model was again adopted, together with the minimum values of yield and tensile strength from SCI/Euro Inox (2006). By adopting a 3.0 mm deformation failure criterion, it was concluded that the SCI/Euro Inox (2006) design rules were conservative, but the rules given in ASCE (2002) are unsafe.

2.4 PREVIOUS STUDIES ON ANGLE TO GUSSET PLATE CONNECTIONS

Angles connected to gusset plates may fail in several modes. Since the net section rupture of the connected angles in gusset plate connections is investigated in the current
research, a review of previous studies concerning this mode of failure is presented in this section.

2.4.1 Carbon steel connections

2.4.1.1 Experimental studies

Investigations into the net section failure of single and double angle members connected to gusset plates under tensile loads date back to as early as the beginning of the last century (McKibben, 1906, 1907). For almost all studies, the efficiency of the net section in resisting the tensile forces, which is measured as the ratio of nominal stress (mean) over the critical net section to the ultimate tensile strength, was less than 100%. Furthermore, the net section efficiency for single and double angles was more or less the same. For instance, the mean net section efficiency obtained from McKibben’s (1906, 1907) experiments on riveted angles was about 80% for both single and double angles and that obtained by Nelson (1953) varied between 64% and 84%. Young (1935) reviewed previous test results conducted on riveted single and double angles. The net section efficiency for single angles ranged between 70% and 80% and that for double angles between 77% and 81%. However, Davis and Boomsliter (1934) concluded (after testing riveted single and double angles) that the net section efficiency for single angles is less than that for double angles. After conducting an experimental investigation, Wu and Kulak (1993) concluded that the net section efficiency of single and double angles could be treated as the same.

The effects of member length and the in-plane restraint of the gusset plate in addition to other parameters were investigated by Nelson (1935). It was concluded that no significant difference was found by doubling the member length or by changing the connection between the gusset plate and the testing machine from fixed to pinned. The same conclusion was also reached by Wu and Kulak (1993).

Many researchers proposed empirical formulae to determine the net section efficiency for these types of connection. Based on previous test results, Young (1935) proposed that the net section efficiency $U$ for single angles be as follows:
CHAPTER 2 – LITERATURE REVIEW

\[ U = 1.0 - 0.18 \frac{B}{A} \]  \hspace{1cm} (2.1)

where \( A \) is the width of the connected leg and \( B \) is the width of the unconnected leg.

By studying the results of 18 single angle specimens, Nelson (1953) concluded that the net section efficiency is a function of the number of bolts per line as well as the relative areas of the unconnected and connected legs. The following empirical equations were suggested:

\[ U = \frac{1}{1 + \frac{r_{\text{area}}}{n_{\text{line}}}} \]  \hspace{1cm} (2.2)

\[ r_{\text{area}} = \frac{A_{\text{g,unc}}}{A_{\text{n,con}}} \]  \hspace{1cm} (2.3)

where \( n_{\text{line}} \) is the number of bolts per line, \( A_{\text{g,unc}} \) is the gross cross-sectional area of the unconnected leg and \( A_{\text{n,con}} \) is the net cross-sectional area of the connected leg.

Munse and Chesson (1963) and Chesson and Munse (1963) studied the net section behaviour of riveted and bolted tension members commonly used in trusses. They examined a large number of specimens including their own test results in addition to those conducted by others. From these test results, they found that the net section efficiency is dependent on many parameters, as explained below. The effective net section area \( A_{\text{net,eff}} \) according to their formula is given by:

\[ A_{\text{net,eff}} = K_1 K_2 K_3 K_4 A_{\text{net}} \]  \hspace{1cm} (2.4)

where \( A_{\text{net}} \) is the net cross-sectional area of the member at the critical section, \( K_1 \) is a ductility factor, \( K_2 \) is a factor for the method of hole forming (0.85 for punched holes and 1.0 for drilled holes), \( K_3 \) is a geometric factor and \( K_4 \) is the shear lag factor.
where \( A_{\text{gross}} \) is the gross cross-sectional area of the member, \( R \) is the percentage reduction in the area of a standard test coupon (51 mm gauge length), \( x \) is the eccentricity of the load measured from the face of the gusset plate to the centre of gravity of the cross-section of the member and \( L \) is the connection length taken as the distance between the first and the last bolts. The most important observation by Chesson and Munse (1963) concerned the effects of the eccentricity and the length of the connection on the net section efficiency, which forms the basis of the design provisions in the AISC Specification (2005).

The influence of the connection eccentricity and connection length was also studied by Bartels (2000) by conducting a series of tests on tee sections connected to gusset plates through their webs. Conclusions similar to those reached by Chesson and Munse (1963) were made. It was found that the net section efficiency increases with an increase in connection length, and increases with a decrease in connection eccentricity. The net section efficiency was assumed to be linearly related to both eccentricity and connection length. Equations 2.8 and 2.9 that predict the net section efficiency in terms of eccentricity and connection length were proposed following linear regression analyses.

\[
U = 0.50 - 0.20x + 0.050L \leq 0.90 \quad (2.8)
\]

\[
U = 0.50 - 0.20x + 0.053L \leq 0.95 \quad (2.9)
\]

where \( x \) and \( L \) are in inches.
Wu and Kulak (1993) concluded that net section efficiencies with three or fewer fasteners per line are significantly lower than members with four or more fasteners per line. They employed their tests to develop and validate FE models.

2.4.1.2 Numerical studies

A number of numerical studies have been conducted to investigate the behaviour of tension members connected unsymmetrically to gusset plates; most of these were concentrated on block shear failure. In most of these investigations, simple FE models were developed. For instance, Wu and Kulak (1993), Epstein and Chamarajanagar (1996) and Epstein and McGinnis (2000) modelled the members only, but the bolts were replaced by applying appropriate boundary conditions on half of the circumference of the bolt hole where bearing between bolts and plates occur. Hence, no contact behaviour was incorporated.

Different failure criteria have been suggested. Epstein and Chamarajanagar (1996) and Epstein and McGinnis (2000) observed that during testing, fracture initiated at the outer edge of the connected part closest to the leading bolt hole and then propagated to the whole section. This observation was used in defining the ultimate load in the FE models. Epstein and Chamarajanagar (1996) assumed that failure took place when the strain at this point reached five times the yield strain of the material, while Epstein and McGinnis (2000) adopted three times the yield strain of the material as a threshold. In other studies, the failure load was defined as the peak load reached during the loading history (Barth et al., 2002; Topkaya, 2004). Wu and Kulak (1993) simply adopted the load corresponding to the last converged step.

Epstein and McGinnis (2000) studied the block shear failure of tee sections connected by the flange using FE models, which had been validated against previously tested specimens. The location where fracture initiated in the tests was the same location as in the study by Epstein and Chamarajanagar (1996). Failure was considered to have occurred when the strain at this point reached three times the yield strain of the material. From the FE results, it was observed that a compressive zone formed in the web, suggesting that considerable moments are induced due to the eccentricity of the tensile loading. In both studies, to avoid including contact definition between interfaces, only
the members were modelled, and the bolts were replaced by applying appropriate boundary conditions on half of the circumference of the bolt hole.

Barth et al. (2002) developed FE models to examine the effects of connection eccentricity and connection length on the ultimate capacity of bolted tee tension members connected by the web. The eight node hexahedral elements in ABAQUS were used to model members and gusset plates while bolts were assumed to be rigid and a surface-to-surface contact used to fully transfer the load from the gusset plate to the web. A trilinear model was used to represent the nonlinear material stress-strain behaviour. The load corresponding to the peak point of the load-deformation curve was taken as the failure load. At failure, substantial necking occurred at the net section of the leading bolt hole. Results of the finite element analyses were compared with experimental results, code predictions and analytical predictions of the section capacity. The failure capacities predicted by the FE models were in good agreement with the experimentally observed failure capacities of the tee sections under tensile loading.

Wu and Kulak (1993) concluded from the FE model that for specimens with four or six bolts, either there was no compression zone in the unconnected leg at all or this zone was not at the critical net section and therefore the net section efficiency for these connections was considerably larger. It was also observed that for connections with four bolts or more, the average stress at failure at the critical section of the connected leg approaches the ultimate strength of the material, while the average stress of the unconnected leg is approximately equal to the yield strength of the material. For connections with less than four bolts, the average stress of the unconnected leg was found to be 50% of the material yield strength while the average stress of the connected leg was also close to the ultimate material strength. This conclusion was employed to determine net section efficiency. They suggested that the ultimate net section capacity \( N_{n,Wu} \) is given by the following formula:

\[
N_{u,Wu} = f_u A_{n,con} + \lambda f_y A_{g,unc}
\]  \hspace{1cm} (2.10)

where \( A_{n,con} \) is net cross-sectional area of the connected leg, \( A_{g,unc} \) is the gross cross-sectional area of the unconnected leg and \( \lambda \) is a reduction factor; for connections with
four bolts or more $\lambda = 1.0$ and for connections with two or three bolts $\lambda = 0.5$. The efficiency $U$ can be obtained by rearranging Equation 2.10, as follows:

$$U = \frac{A_{n,\text{con}} + \lambda \left( \frac{f_y}{f_u} \right) A_{g,\text{unc}}}{A_{\text{net}}}$$

(2.11)

where $A_{\text{net}}$ is the net cross-sectional area of the member at the critical section.

The FE models developed by Topkaya (2004) were able to capture necking at the critical net section at the lead bolt; such necking was observed during tests in many studies. Parametric studies were employed to develop a block shear design equation.

### 2.4.2 Stainless steel connections

#### 2.4.2.1 Experimental studies

Ryan (1999b) conducted an experimental investigation to assess the design rules for net section rupture of stainless steel gusset plate connections given in ENV 1993-1-4 (1996). A total of 12 specimens were tested. The specimens were composed of a member with a single angle or a tee section member connected to gusset plates at the ends of the member and a tensile load were applied to the plates. It was concluded that the rules in ENV 1993-1-4 (1996) for the net section tensile capacity of angles were unconservative for relatively long connections. For short connections, the failure modes were other than net section rupture, hence, the validity of these rules could not be confirmed. The $\beta$ factor rules given in ENV 1993-1-1 (1992) were found to be suitable for stainless steel connections. The tests are described in detail in Chapter 6, where they are employed to validate the numerical models.

### 2.5 CURRENT DESIGN RULES FOR STAINLESS STEEL CONNECTIONS

Despite the fundamental differences in the mechanical behaviour of stainless steel and carbon steel discussed in Chapter 1, design provisions for bolted connections between stainless steel structural members in current international standards are essentially based

The American and Australian Standards provide design rules for bolted connections between thin cold-formed stainless steel but do not have rules for thick hot-rolled stainless steel. The ASCE (2002) and AS/NZS (2001) Specifications provide design provisions for net section capacity exactly that are the same as those in the AISI Specification (1996) for cold-formed carbon steel bolted connections, and marginally modified the bearing equations.

2.6 CONCLUDING REMARKS

The research on structural stainless steel has increased recently as a natural result of the recent growth in the usage of the material in construction. Most of these studies, which have been focused on the behaviour of beams and columns, have shown that a significant improvement in the load carrying capacity of these members was achieved by recognising the true nonlinear stress-strain behaviour of stainless steel. Yet, as the review in this chapter has demonstrated, research on stainless steel joints is still limited and lags behind. This limited studies of stainless steel connections means that a full understanding of the actual response of these important structural components has yet to be attained. Consequently, international standards in their design guidance, simply assume an analogy with carbon steel. The scope of the present research is primarily to investigate the underlying behaviour of stainless steel connections to enable the development of appropriate and simple design equations, which are, where possible, consistent with those for carbon steel.
CHAPTER 3

NUMERICAL MODELLING

3.1 INTRODUCTION

Although experimental testing plays a major role in developing an understanding of the behaviour of structures, results from finite element simulations can provide further insight, allowing, for example, the detailed analysis of stresses and strains. Therefore, numerical simulations can be used to generate supplementary information that is not readily available from experiments. In addition, parametric studies using validated finite element models can be carried out to provide a basis for enhancing the efficiency of the design provisions. Sophisticated finite element models can now handle various sources of nonlinearity: large deformations, plasticity and contact compatibility. A combination of experiments and numerical simulations is now commonplace in structural engineering research. The most efficient approach is to conduct selected experiments and then to replicate the results of these tests using finite element (FE) models. Once these models are validated, further results can be generated by changing the different geometrical and mechanical parameters. In this chapter, FE models of stainless steel connections have been developed and validated against existing test results.
3.2 MODELLING PARAMETERS

3.2.1 Material modelling

Incorporating a precise description of material behaviour is pivotal to the development of accurate FE models. Inappropriate representation of material behaviour will adversely affect the results obtained from the FE models.

The stress-strain behaviour of stainless steel is different from that of carbon steel. While carbon steel has a well-defined yield point followed by a plateau and some strain hardening at high strain, stainless steel exhibits a rounded stress-strain curve and considerable strain hardening at relatively small strains. Since no pronounced yield stress exists for stainless steel, an equivalent yield stress based on the stress at 0.2% plastic strain is typically adopted, as shown in Figure 3.1. Representing stainless steel behaviour in FE models using the conventional elastic-perfectly-plastic material model, which is frequently used for carbon steel, leads to inaccurate results. Therefore, exact material modelling is important to exploit the special mechanical features of this material.

![Stress-strain curves](Figure 3.1: Representative stress-strain curves for carbon steel and stainless steel)
3.2.1.1 Material model adopted in the present study

To describe the nonlinear behaviour of stainless steel material, the compound Ramberg-Osgood stress-strain expressions (Ramberg and Osgood, 1943) that were developed by Mirambell and Real (2000) and Rasmussen (2003) – given by Equation 3.1 – were incorporated into the FE models. This material model has the ability to describe the stress-strain curve accurately up to the ultimate stress and, therefore, was found to be suitable (Rasmussen, 2003) for structural components that undergo significant straining before reaching their ultimate capacity, as is the case for lap connections subjected to tensile loads, which are considered in the present study.

\[
\varepsilon = \begin{cases} 
\frac{\sigma}{E} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^{n_s} & \text{for } \sigma \leq \sigma_{0.2} \\
\left( \frac{\sigma - \sigma_{0.2}}{E_{0.2}} \right) + \varepsilon_0 \left( \frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^{m_s} + \varepsilon_{0.2} & \text{for } \sigma_{0.2} < \sigma \leq \sigma_u 
\end{cases}
\]  

where \( \varepsilon \) is the nominal strain, \( \sigma \) is the corresponding nominal stress, \( \sigma_{0.2} \) is the 0.2% proof stress, \( \varepsilon_{0.2} \) is the total strain at the 0.2% proof stress, \( \varepsilon_0 \) is the strain at the ultimate stress \( \sigma_u \), \( E \) is the initial Young’s modulus, \( E_{0.2} \) is the tangent stiffness at the 0.2% proof stress and \( n_s \) and \( m_s \) are the strain hardening exponents. These parameters can be obtained from Equations 3.2 to 3.5 developed by Rasmussen (2003).

\[
E_{0.2} = \frac{E}{1 + 0.002n_s \left( \frac{\sigma_{0.2}}{E} \right)}
\]  

\[
n_s = 5 + \left[ \frac{\sigma_{0.2} - \sigma_u \left( 0.2 + 185 \frac{\sigma_{0.2}}{E} \right)}{0.0375 \sigma_{0.2}} \right]
\]  

\[
m_s = 1 + 3.5 \frac{\sigma_{0.2}}{\sigma_u}
\]
\[ \varepsilon_u = 1 - \frac{\sigma_{0.2}}{\sigma_u} \]  \hfill (3.5)

Since FE analysis for bolted connections is expected to involve large inelastic strains, nominal stresses \( \sigma_{\text{nom}} \) and nominal strains \( \varepsilon_{\text{nom}} \) were converted into the corresponding true stresses \( \sigma_{\text{true}} \) and true plastic strains \( \varepsilon_{\text{true}}^{\text{pl}} \) respectively. These account for the change of dimensions under load and were incorporated into the models using the following relationships:

\[ \sigma_{\text{true}} = \sigma_{\text{nom}} (1 + \varepsilon_{\text{nom}}) \]  \hfill (3.6)

\[ \varepsilon_{\text{true}}^{\text{pl}} = \ln(1 + \varepsilon_{\text{nom}}) - \frac{\sigma_{\text{true}}}{E} \]  \hfill (3.7)

The true stress in a stress-strain curve continues to rise beyond the nominal ultimate stress until the occurrence of fracture. In order to model the full material response until fracture, so as to capture the localized stresses and strains, the true stress-strain curve was extended beyond the true ultimate stress up to fracture at 100% strain (see Section 3.2.6.1) using Equation 3.8 (Dowling, 1999).

\[ \sigma_{\text{true}} = K (\varepsilon_{\text{true}}^{\text{pl}})^p \quad \text{for} \quad \sigma_{\text{u,true}} < \sigma_{\text{true}} \leq \sigma_{\text{frac,true}} \]  \hfill (3.8)

where \( \sigma_{\text{u,true}} \) is the true ultimate stress, \( \sigma_{\text{frac,true}} \) is the true fracture stress, \( K \) is the strength constant and \( p \) is the strain hardening exponent. \( K \) and \( p \) were obtained by substituting the true stresses and the corresponding true plastic strains of two points obtained from Equation 3.1 into Equation 3.8. A summary of the values of the constants \( n_s, m_s, K \) and \( p \) for the material employed in this study is given in Table 3.2.

All models were developed using the FE package ABAQUS (2007) in which material behaviour may be represented by a multi-linear stress-strain curve in terms of true stress and true plastic strain. Plasticity was treated by means of the Von Mises yield criterion with isotropic hardening, whereby the yield surface expands uniformly in all directions.
such that the yield stress effectively increases in all stress directions as plastic strains develop.

### 3.2.2 Element type

Solid elements are generally used to model thick-walled structural components. The three-dimensional solid element with full integration – C3D8 in ABAQUS (2007) – contains eight nodes with three translational displacements for each node and utilises eight integration points. This element is suitable for complex nonlinear analyses involving contact, plasticity and large deformations (ABAQUS, 2007), and has proved to be suitable when simulating lap bolted connections in previous similar investigations (Chung and Ip, 2000; Ju et al., 2004) and is hence employed for the models generated herein.

The size of the FE model mesh is one of the crucial parameters that may have a significant influence on the results. Generally, finer meshes can predict the behaviour more accurately than coarser ones. On the other hand, the computation cost increases with the increase of mesh fineness. An optimum mesh density that makes a compromise between the accuracy and the solution cost was determined through a convergence study.

### 3.2.3 Contact

Contact between all components in the connections that are expected to interact with each other was defined using a surface-to-surface contact command in ABAQUS/Standard. In this command, two surfaces that may interact with each other are paired; one surface is assigned as a master and the other as a slave. The master-slave algorithm in ABAQUS/Standard recognises the surfaces that are in contact and applies constraints (pressure) to slave nodes in order to prevent them penetrating the master surface. The frictional effects between contact surfaces were also included by incorporating the classical isotropic Coulomb friction model into the contact definition. In this model, two contacting surfaces can transmit shear stresses up to a certain limit $\tau_{\text{max}}$ before sliding relative to one another takes place. This maximum shear stress is
proportional to the contact pressure \( \tau_{\text{max}} = \mu \times \text{normal pressure} \) where the constant \( \mu \) is the coefficient of friction. The frictional stress acting on the slave node is compared to the maximum shear stress and hence, the condition of the node is defined as either sticking or sliding. The sensitivity of the behaviour of the connection to variations in the friction coefficient value was assessed (see Section 3.3.3). It was found that the friction between surfaces affects the slippage load. A constant friction coefficient \( \mu \) equal to 0.2 throughout the analysis, which was used previously in a study by Chung and Ip (2000), was employed.

The surfaces that are anticipated to interact with each other in the current models include the bolt shank to bolt holes, the bolt head or nut to plates and the internal plates to external plates.

### 3.2.4 Bolt preload

The bolt preload which usually arises from bolt tightening has been simulated using the BOLT LOAD option in ABAQUS/Standard. To apply a bolt load using this command a cross-section through the bolt shank and the axis of the bolt shank perpendicular to the cross-section are defined. The total bolt preload value that is acting on the cross-section and coinciding with the bolt shank axis is then specified. Bolts were located centrally into the holes with a uniform clearance of 1.0 mm. Applied loads were initially carried by friction until the occurrence of slippage, after which direct bearing was the primary means of load transfer.

### 3.2.5 Method of analysis

A nonlinear structural problem is one in which the stiffness of the structure changes as it deforms. There are three common sources of nonlinearity in structural models, material and geometric nonlinearities being the most frequently encountered; a third source is the boundary nonlinearity associated with a change in the boundary conditions during the analysis. A clear example of this kind of nonlinear behaviour is a model that contains contact between different parts, as is the case for bolted connections. In the current study, material, geometrical and contact nonlinearities were incorporated into the
models using the PLASTIC, NLGEOM and CONTACT PAIR commands respectively in ABAQUS. A general static analysis with displacement control was employed in the present study. ABAQUS/Standard uses the Newton-Raphson method to obtain solutions for nonlinear problems. In a nonlinear analysis, the solution is found by applying the loads or displacements in increments until the final solution is reached. Many iterations may be required to establish a solution at a given load increment. The approximate solution is the summation of all of these increments.

Models containing contact definitions are computationally expensive owing to the algorithm that is used to solve these problems. At the beginning of each increment, an inspection of the state of all contact interactions is performed to determine whether slave nodes are open or closed. If the node is closed a further check is performed to identify whether the node is sticking or sliding. Constraints are applied by the algorithm to each closed node and are released from each open node. This process is repeated until no change is found in the status of all slave nodes. Then, an equilibrium iteration is performed, and if the residuals are not within the prescribed tolerance, the increment is neglected and a smaller increment is applied and the procedure is repeated. This process often makes the convergence of such models extremely difficult and in many cases the program terminates before the entire solution can be found.

3.2.6 Failure criteria for FE models

To verify the ability of the FE models to replicate the experimental results, including the full deformation characteristics and failure modes, precise material modelling and failure criteria are required. In numerical investigations of structural members such as beams and columns, the ultimate capacity is, simply, the peak load from the load-deformation curve. In the case of bolted connections however, this curve usually exhibits no clear peak; moreover the failure mode must be identified to relate the load carrying capacity to the appropriate mode. For this reason, employing suitable failure criteria is vital when studying bolted connections numerically. Three failure modes were considered in this study – net section fracture, bolt shear failure and bearing failure. The definition of the net section failure in this study is based on a fracture strain criterion –
this requires the accurate determination of localized fracture strains in ductile materials, which is discussed below.

### 3.2.6.1 Fracture strain determination

The true plastic strain at fracture from a tension test coupon based on elongation $\varepsilon_{\text{frac,elong}}$ is given by the following relationship:

$$
\varepsilon_{\text{frac,elong}} = \ln\left(\frac{L_f}{L_0}\right) \times 100 \%
$$

(3.9)

where $L_0$ is the original gauge length and $L_f$ is the final length after fracture. Fracture strains derived using Equation 3.9 are average values over the chosen gauge length, and will vary with gauge length. This is due to the fact that, for ductile materials, once necking occurs, deformations become localized, resulting in very high strains in a small region until the occurrence of rupture as shown in Figure 3.2. Dowling (1999) explained that for a ductile material, Equation 3.9 significantly underestimates strains beyond necking and therefore is invalid in this region. More accurate values of the true localized fracture strain, which is considered to be more suitable for the prediction of fracture in numerical simulations, can be calculated on the basis of the reduction in cross-sectional area at fracture, as follows:

$$
\varepsilon_{\text{frac,area}} = \ln\left(\frac{A_0}{A_f}\right) \times 100 \%
$$

(3.10)

where $A_0$ is the original cross-sectional area and $A_f$ is the minimum cross-sectional area after fracture.

Many experimental studies have shown that the localized fracture strain obtained using Equation 3.10 is often much higher than the fracture strain determined using Equation 3.9. An experimental investigation by Khoo et al. (2000) showed that the localized fracture strain based on area reduction for structural carbon steel ranges between about 80% and 120%, with an average value of 100%. Table 3.1 compares the fracture strains
\( \varepsilon_{\text{frac,elong}} \) and \( \varepsilon_{\text{frac,area}} \) obtained from test coupons from three studies (Dowling, 1999; Huns et al., 2002; Nip et al., 2010). Despite marked differences in material properties, the localized fracture strain based on area reduction may be seen to be fairly consistent, ranging between 93% and 117%.

Huns et al. (2002) conducted a numerical investigation of gusset plate connections. In validating their FE models a fracture strain of 100% was adopted in the absence of measured values. This value of the localized fracture strain represents a lower bound to their test results and an average of the values reported by Khoo et al. (2000).

\[ \text{Figure 3.2: Deformation of a ductile material in a tensile test: (a) original specimen, (b) before necking and (c) after fracture.} \]
### Table 3.1: Coupon test results from existing studies

<table>
<thead>
<tr>
<th>Reference</th>
<th>Steel</th>
<th>$f_y$ or $\sigma_{0.2}$ (N/mm²)</th>
<th>$f_u$ (N/mm²)</th>
<th>$f_u/f_y$</th>
<th>$\varepsilon_{\text{frac,elong}}$ (%)</th>
<th>$\varepsilon_{\text{frac,area}}$ (%)</th>
<th>$\varepsilon_{\text{frac,area}} / \varepsilon_{\text{frac,elong}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Huns et al. (2002)</td>
<td>Hot-rolled carbon steel</td>
<td>336</td>
<td>450</td>
<td>1.34</td>
<td>30</td>
<td>113</td>
<td>3.77</td>
</tr>
<tr>
<td>Dowling (1999)</td>
<td>Hot-rolled carbon steel</td>
<td>260</td>
<td>441</td>
<td>1.70</td>
<td>31</td>
<td>94</td>
<td>3.03</td>
</tr>
<tr>
<td>Nip et al. (2010)</td>
<td>Hot-rolled carbon steel</td>
<td>461</td>
<td>543</td>
<td>1.18</td>
<td>36</td>
<td>117</td>
<td>3.25</td>
</tr>
<tr>
<td>Nip et al. (2010)</td>
<td>Cold-formed stainless steel</td>
<td>498</td>
<td>755</td>
<td>1.52</td>
<td>60</td>
<td>93</td>
<td>1.55</td>
</tr>
</tbody>
</table>
In the experimental programme on stainless steel connections reported by Ryan (1999a), the results of which are used in the present study for FE validation, fracture strain of the material based on area reduction was not measured. However, since a number of studies have shown that the localized fracture strain for structural steel and stainless steel based on area reduction are fairly consistent at around 100% (see Table 3.1). In the FE models, fracture of the material was therefore assumed to take place when the equivalent plastic strain reached this value.

In some previous numerical studies (Epstein and Chamarajanagar, 1996; Epstein and McGinnis, 2000; Huns et al., 2002; Kim and Kuwamura, 2007) connection failure was assumed to occur when either the stresses or strains at critical points (where stresses are concentrated) reach threshold values. However, there is no consensus amongst researchers about these limiting values. For example, in the numerical studies of block shear failure in hot-rolled carbon steel gusset plate connections that were conducted by Epstein and Chamarajanagar (1996), it was assumed that fracture initiates when the plastic strain at the critical location reaches five times the yield strain, while Epstein and McGinnis (2000) adopted three times the yield strain. Huns et al. (2002), in an investigation of gusset plate connections, concluded that the rupture strain proposed by Epstein and Chamarajanagar (1996) – five times the yield strain – is much smaller than the rupture strain observed for structural hot-rolled steel; instead a fracture strain of 100% was adopted. Given that properties will vary between different grades and forms of material, Kim and Kuwamura (2007) suggested limiting either the maximum stress or strain at critical locations to the corresponding maximum true stress or strain obtained from coupon tests. In this study, as noted above, a fracture strain of 100% will be used for all models.

3.2.6.2 Net section failure

The existence of holes in a plate causes a discontinuity in geometry and a disruption of the stress trajectories. As a result, stress concentrations occur in the region of the hole. As the applied load increases, the magnitude of the maximum stress increases until fracture initiates at the edge of the bolt hole which is closest to the applied load where the maximum stress concentration develops; the crack then propagates transversely.
across the net section resulting in net section failure, as illustrated in Figure 3.3. The form of this failure is greatly affected by the ductility of the material. If the material is ductile, local yielding at the hole edge as a result of the stress concentration allows for stress redistribution across the net section, resulting in necking and finally a ductile failure. In contrast, if the material possesses little ductility, the stress concentration in the vicinity of the hole cannot be redistributed leading to immediate cracking, resulting in a brittle failure. Net section fracture is critical in connections with relatively narrow plate widths. To model net section rupture in the FE models, reference was made to the aforementioned point at the edge of the bolt hole (see Figure 3.3); when the maximum equivalent plastic strain at this point reaches the true localized fracture strain of 100%, it was assumed that a crack initiates, and the ultimate load is said to be reached.

**Figure 3.3: Illustration of net section fracture**

### 3.2.6.3 Bolt shear failure

In bearing-type lap connections the load is transferred between the connected plates by means of shear across the bolt cross-section. When the shear load in the bolt exceeds its capacity, a brittle failure occurs called ‘bolt shear’ (Kulak et al., 1987). Owens and Cheal (1989) concluded that the shear resistance of a single bolt with the shank in the shear plane is about 80% of its ultimate tensile resistance, while this value is only 63%
when the shear plane passes through the threads. In addition, they stated that in the latter case the deformation capacity is significantly reduced.

Ju et al. (2004) investigated bolt shear failure for lap bolted connections similar to those in this study using numerical models. It was proposed that a bolt fails in shear when the critical cross-section of the bolt becomes fully plastic. In the current study, this criterion has also been adopted. Hence, when the equivalent plastic strain over the full critical cross-section reaches the true plastic strain corresponding to $\sigma_{0.2}$, the cross-section of the bolt is fully plastic, and is deemed to have failed.

**3.2.6.4 Bearing failure**

The bearing resistance of bolted connections may be defined either on the basis of strength or deformation criteria. For the former, the bearing resistance is taken as the maximum load measured in tests regardless of the associated extension. In contrast, adopting a deformation criterion, the ultimate capacity of a connection is defined as the load corresponding to a specified extension. For carbon steel, for which the load-deformation behaviour flattens once significant bearing deformations have occurred, the exact choice of deformation limit has relatively little effect on the corresponding load. However, stainless steel connections, owing to the hardening characteristics of the material, generally exhibit a rising relationship, with no clear flattening. Thus, the load level corresponding to bearing failure will depend on the deformation limit selected. Figure 5.3 in Chapter 5 demonstrates the difference in the deformation response for carbon steel and stainless steel bolted connections. In the experimental study performed by Ryan (1999a), three specimens were reported to have failed solely by bearing. Since these connections were not loaded until the occurrence of rupture (as shown in the photographs), the reported maximum loads for these connections do not correspond to an actual fracture. Clearly, applying arbitrary deformation limits will give inconsistent results. For this reason, the validation of the FE models for these connections will be on the basis of the load-deformation curves only. The bearing behaviour in stainless steel connections has been investigated by Salih et al. (2009a) and Nethercot et al. (2009), and is discussed in this thesis in Chapter 5.
3.3 VALIDATION OF FE MODELS

3.3.1 Introduction

In this study, the finite element software ABAQUS 6.7.1 (2007) has been used to develop numerical models for the austenitic grade 1.4306 and ferritic grade 1.4016 stainless steel lap bolted connections that were tested by Ryan (1999a). Table 3.2 summarises the mechanical properties of the materials. The details of these specimens are presented in Figure 3.4 and Table 3.3.

3.3.2 Boundary conditions and loading

The symmetry of the tested specimens about two axes allows the division of the connection into four identical parts. In order to reduce the size of the model and, consequently, the computational cost, only one quarter of the connection was modelled by applying appropriate boundary conditions. Load was applied by means of displacement-control which enables the post-ultimate behaviour in the nonlinear analyses to be captured. A uniform translational displacement was applied at the end of the inner plate. Figure 3.5 illustrates the boundary conditions and loading applied to the FE models.
### Table 3.2: Material properties of specimens tested by Ryan (1999a)

<table>
<thead>
<tr>
<th>Steel</th>
<th>Element</th>
<th>$f_y$ (σ₀.₂) (N/mm$^2$)</th>
<th>$f_u$ (N/mm$^2$)</th>
<th>Elongation at fracture (%)</th>
<th>$f_u/f_y$</th>
<th>$n_s$</th>
<th>$m_s$</th>
<th>$K$ (N/mm$^2$)</th>
<th>$p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic grade 1.4306</td>
<td>Internal plates</td>
<td>288</td>
<td>581</td>
<td>62.0</td>
<td>2.02</td>
<td>6.58</td>
<td>2.73</td>
<td>1548</td>
<td>0.64</td>
</tr>
<tr>
<td>Austenitic grade 1.4306</td>
<td>External plates</td>
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<td>577</td>
<td>63.0</td>
<td>2.13</td>
<td>6.10</td>
<td>2.64</td>
<td>1585</td>
<td>0.68</td>
</tr>
<tr>
<td>Ferritic grade 1.4016</td>
<td>Internal plates</td>
<td>262</td>
<td>522</td>
<td>51.0</td>
<td>1.99</td>
<td>8.16</td>
<td>2.76</td>
<td>1390</td>
<td>0.64</td>
</tr>
<tr>
<td>Ferritic grade 1.4016</td>
<td>External plates</td>
<td>350</td>
<td>487</td>
<td>26.0</td>
<td>1.39</td>
<td>12.2</td>
<td>3.52</td>
<td>1008</td>
<td>0.34</td>
</tr>
<tr>
<td>Austenitic</td>
<td>M12 bolt</td>
<td>692</td>
<td>863</td>
<td>-</td>
<td>1.25</td>
<td>3.73</td>
<td>3.81</td>
<td>1554</td>
<td>0.24</td>
</tr>
<tr>
<td>Austenitic</td>
<td>M16 bolt</td>
<td>748</td>
<td>955</td>
<td>-</td>
<td>1.28</td>
<td>1.30</td>
<td>3.74</td>
<td>1767</td>
<td>0.25</td>
</tr>
<tr>
<td>Austenitic</td>
<td>M20 bolt</td>
<td>686</td>
<td>852</td>
<td>-</td>
<td>1.24</td>
<td>4.03</td>
<td>3.82</td>
<td>1530</td>
<td>0.23</td>
</tr>
</tbody>
</table>
Figure 3.4: Configuration of specimens tested by Ryan (1999a) and investigated numerically in the present study
Figure 3.4 (continued): Configuration of specimens tested by Ryan (1999a) and investigated numerically in the present study
Table 3.3: Geometry of experimental specimens tested by Ryan (1999a)

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Type of connection</th>
<th>Plate material</th>
<th>Bolts diameter (mm)</th>
<th>Plate dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>t</td>
<td>e₁</td>
</tr>
<tr>
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<td>12</td>
<td>10</td>
</tr>
<tr>
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<td>Type I</td>
<td>Austenitic</td>
<td>16</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>Type I</td>
<td>Austenitic</td>
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<td>10</td>
</tr>
<tr>
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<td>Austenitic</td>
<td>12</td>
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<tr>
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<td>Austenitic</td>
<td>16</td>
<td>10</td>
</tr>
<tr>
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<td>Type II</td>
<td>Austenitic</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>7</td>
<td>Type III</td>
<td>Austenitic</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>8</td>
<td>Type III</td>
<td>Austenitic</td>
<td>16</td>
<td>10</td>
</tr>
<tr>
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</tr>
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</tr>
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<td>Type IV</td>
<td>Austenitic</td>
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<td>10</td>
</tr>
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<td>Type IV</td>
<td>Austenitic</td>
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<td>10</td>
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</tbody>
</table>
### Table 3.3 (continued): Geometry of experimental specimens tested by Ryan (1999a)

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Type of connection</th>
<th>Plate material</th>
<th>Bolts diameter (mm)</th>
<th>Plate dimensions (mm)</th>
</tr>
</thead>
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<tr>
<td></td>
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<td>e₁</td>
</tr>
<tr>
<td>13</td>
<td>Type I</td>
<td>Ferritic</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>14</td>
<td>Type I</td>
<td>Ferritic</td>
<td>16</td>
<td>8</td>
</tr>
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<td>Ferritic</td>
<td>20</td>
<td>8</td>
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<tr>
<td>22</td>
<td>Type IV</td>
<td>Ferritic</td>
<td>12</td>
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<td>Ferritic</td>
<td>16</td>
<td>8</td>
</tr>
<tr>
<td>24</td>
<td>Type IV</td>
<td>Ferritic</td>
<td>20</td>
<td>8</td>
</tr>
</tbody>
</table>
Figure 3.5: FE models for the four connection types shown in Figure 3.4
3.3.3 Effect of bolt preload and coefficient of friction

In slip-resistant connections, design standards prescribe a minimum bolt preload, whereas in bearing-type connections bolts need only to be tightened to a snug-tight condition, such that the connected plates are in firm contact. In order to examine the effect of the bolt preload on the behaviour of bearing connections, three values of bolt preload were studied for specimen No. 6 in Table 3.3; these are 40 kN, 80 kN and 120 kN representing approximately 15%, 30% and 45% of the bolt ultimate tensile capacity. The coefficient of friction $\mu$ was fixed at 0.2. From Figure 3.6 it can be seen that the bolt preload has a major influence on the slip resistance. For a specific coefficient of friction, the slip resistance increases approximately linearly with bolt preload, but once the connected plates and the bolt shank have come into contact, the bolt preload does not affect the behaviour of the connection.

The influence of the coefficient of friction between the contact surfaces on the behaviour of bearing connections was also investigated. The same specimen (No. 6) was considered. Three values of friction coefficient were examined: 0.1, 0.2 and 0.3 with a bolt preload equal to 80 kN (30% of the bolt ultimate tensile capacity). Figure 3.7 indicates that the friction between components has a dominant role on the behaviour before the major slip occurs. However, the friction has insignificant effects on the connection’s stiffness once the slip into bearing has taken place. Since neither the bolt preload nor the coefficient of friction has significant influence on the overall strength of bearing-type connections, constant values of $\mu = 0.2$ and a preload of 30% of the bolt ultimate tensile capacity were used throughout the study.
Figure 3.6: Bearing behaviour for different bolt preloads

Figure 3.7: Bearing behaviour for different friction coefficients
3.3.4 Comparison between test and FE results

The FE models were validated against the experimental results of Ryan (1999a) in two steps. In the first step, the overall deformation behaviour was validated by comparing the experimental and numerical load-deformation curves. It should be noted that slip was prevented in the laboratory tests and therefore in all numerical load-deformation curves a slip of 4.0 mm was subtracted from the deformation values. It is clear from Figures 3.8 to 3.11 that the predicted load-deformation curves are in good agreement with the tests. The discrepancy in initial stiffness is attributed to the slack in the experimental setup upon first loading. The effect of this slackening on the initial stiffness is evident in the high stiffness during subsequent unloading-reloading stages. When unloading and reloading, the drop of load after reaching a peak in Figures 3.8, 3.10 and 3.11 occurs due to necking of the critical net section; the mode of failure for those models was net section rupture. However, no peak is evident in Figure 3.9 due to the absence of such necking. Note that the extension shown is the total deformation between the ends of the connection, as illustrated in Figure 3.13. In the second step, the ultimate capacities obtained from the FE models using the proposed failure criteria set out in Section 3.2.6 were compared with those obtained in the tests. The comparison in Table 3.4 shows that the developed models are able to predict the observed ultimate capacities very accurately, with a mean FE/test ratio of 0.99 and a standard deviation of 0.03. It is shown in Table 3.4 that the scatter of the results is very low with only one test outside 5%, and 5 tests outside 2%. The bearing failure reported during testing, as discussed in Section 3.2.6.4, is rather subjective and does not represent a ‘true’ failure and thus it was excluded for the purposes of validation. Comparisons between the deformed specimens for two typical tests and the corresponding FE models are shown in Figures 3.12 and 3.13. It can be concluded that the developed numerical models are able to replicate the experimental behaviour and can, therefore, be employed to increase the pool of structural performance data by means of parametric studies, which are described in the following two chapters: Chapters 4 is concerned chiefly with net section failure while Chapter 5 deals principally with bearing failure.
**Figure 3.8:** Load-deformation curves from tests and numerical models
(Type I austenitic steel connection)

**Figure 3.9:** Load-deformation curves from tests and numerical models
(Type II ferritic steel connection)
Figure 3.10: Load-deformation curves from tests and numerical models
(Type III ferritic steel connection)

Figure 3.11: Load-deformation curves from tests and numerical models
(Type IV austenitic steel connection)
Figure 3.12: Comparison between deformed test specimen and numerical model (net section failure – Type I connections)

Figure 3.13: Comparison between deformed test specimen and numerical model (bearing failure – based on excessive deformation – Type II connection)
### Table 3.4: Comparison between test and FE results (see Table 3.3)

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Type of connection</th>
<th>Plates material</th>
<th>Test Ultimate load (kN)</th>
<th>Test Failure mode</th>
<th>FE Ultimate load (kN)</th>
<th>FE Failure mode</th>
<th>FE/Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Type I</td>
<td>Austenitic</td>
<td>173.9</td>
<td>N/SH</td>
<td>171.6</td>
<td>N</td>
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<td>2</td>
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<td>N</td>
<td>224.8</td>
<td>N</td>
<td>0.96</td>
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<td>3</td>
<td>Type I</td>
<td>Austenitic</td>
<td>297.1</td>
<td>N</td>
<td>289.1</td>
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</tr>
<tr>
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<td>SH</td>
<td>1.02</td>
</tr>
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<td>328.6</td>
<td>SH</td>
<td>0.96</td>
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<td>6</td>
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<td>Austenitic</td>
<td>444.5</td>
<td>B*</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
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<td>378.7</td>
<td>N</td>
<td>385.6</td>
<td>N</td>
<td>1.02</td>
</tr>
</tbody>
</table>

**Mean** 0.99  
**S.D.** 0.03

B = Bearing failure - SH = Bolt shear failure - N = Net section failure  
* No full rupture occurred during testing
3.4 CONCLUDING REMARKS

In this chapter, FE models for lap bolted connections in stainless steel have been developed using the ABAQUS package. Numerical analyses including geometrical, material and boundary nonlinearities have been performed. The essential features of the bolted connections have been incorporated including hole clearance, friction and tightening arising from the bolt preload. Using results from previous laboratory tests, together with appropriate failure criteria, model validation was performed in two stages. Firstly, the load-deformation curves obtained from the tests were compared with those extracted from the FE models; excellent agreement was obtained. Secondly, the ultimate capacities were compared; a mean ratio of ultimate FE capacity to ultimate test capacity of 0.99 and a standard deviation of 0.03 was obtained. Furthermore, the effect of the bolt preload and the friction coefficient on lap bolted connections were investigated to ensure that the models are a good representation of the experimental specimens and can be used to thoroughly study the behaviour of bolted connections in stainless steel by means of parametric studies.
CHAPTER 4

NET SECTION FAILURE

4.1 INTRODUCTION

In this chapter, the validated FE models developed in Chapter 3 have been extended to study the behaviour of net section failure of stainless steel connections by means of parametric studies. Connections between both thick plates and thin plates were investigated. The outcomes of the parametric studies were then used to propose design equations.

4.2 CURRENT DESIGN RULES

4.2.1 Eurocode 3 and the SCI/Euro Inox Design Manual


\[ N_{u,ENV3} = \frac{0.9{k_r A_{net} f_u}}{\gamma_{M2}} \]  (4.1)
where $k_r$ is a reduction factor defined by Equation 4.2, $A_{\text{net}}$ is the net cross-sectional area at the critical section, $f_u$ is the material ultimate tensile strength and $\gamma_{M2}$ is a partial safety factor.

$$k_r = 1 + 3r\left(\frac{d_0}{u} - 0.3\right) \leq 1.0$$

(4.2)

in which $d_0$ is the bolt hole diameter, $r$ is the ratio between the number of bolts at the cross-section considered to the total number of bolts in the connection and $u = 2e_2$ but $\leq p_2$, where $e_2$ is the edge distance and $p_2$ is the spacing between bolts perpendicular to the loading (see Figure 4.4). The design provisions adopted in these standards was proposed by SCI/Euro Inox (1996) based on an experimental study. Nine specimens were reported to have failed by net section rupture. Only in three of them, the ratio of the nominal (mean) stress over the critical net section at failure to material tensile strength was less than 0.9. Hence, it was decided to include the $k_r$ factor in addition to the constant factor 0.9 which was already used for hot-rolled carbon steel connections.

Ryan’s (1999a) experimental results plotted in Figures 4.1 to 4.3, show that the ratio of the nominal stress over the net section at ultimate load $f_{\text{net}}$ to the material ultimate tensile strength $f_u$ is greater than 0.9 in all cases, and greater than 1.0 in all bar two cases. On the basis of these results it was recommended (Ryan, 1999a) that the 0.9 reduction factor in the net section resistance of stainless steel is not needed and can be removed. However, it was decided that the test results were insufficient to assess the need for the $k_r$ factor, and it was recommended that further investigation into net section behaviour is necessary.

The recent versions of EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (2006) have adopted the design rules for cold-formed carbon steel connections in EN 1993-1-3 (2006) with no distinction being made between thick and thin plated connections, i.e. the 0.9 factor was removed, and resistance to net section fracture $N_{u, EC3}$ is given by Equation 4.3:

$$N_{u, EC3} = \frac{k_r A_{\text{net}} f_u}{\gamma_{M2}}$$

(4.3)
Figure 4.1: Ryan’s (1999a) test results for two bolts in one row ($r = \frac{1}{2}$)

Figure 4.2: Ryan’s (1999a) test results for four bolts in two rows ($r = \frac{1}{2}$)
4.2.2 American and Australia/New Zealand Standards

The ASCE (2002) and AS/NZS (2001) standards provide design rules for stainless steel bolted connections composed of thin cold-formed sheets, which are similar to those recommended for carbon steel in the AISI Specifications (1996). The net section resistance is given by Equation 4.4, where symbols have been harmonised with those used in Eurocode 3.

\[
\phi_{\text{net}} N_u = A_{\text{net}} f_t
\]  
\[N_u,\text{ASCE} = \phi_{\text{net}} A_{\text{net}} f_t \]  

(4.4)

where \( \phi_{\text{net}} \) is the resistance factor of 0.7 and \( f_t \) is the nominal tension stress as follows:

for single shear connections

\[
f_t = \left( 1.0 - r + 2.5 r \frac{d}{p_2} \right) f_u \leq f_u
\]  

(4.5)

Figure 4.3: Ryan’s (1999a) test results for three bolts \((r = 2/3)\)
and for a double shear connection
\[ f_i = \left( 1.0 - 0.9r + 3r \frac{d}{p_2} \right) f_u \leq f_u \] (4.6)

in which \( d \) is the nominal bolt diameter, and in the case of single-row bolted connections, \( p_2 = 2e_2 \).

The design expressions of Equations 4.2 and 4.6 are a modified version of Winter’s (1956) equation, which was derived from experiments on bolted connections between cold-formed carbon steel sheets. For specimens failing in net section fracture, Winter concluded that the ductility of this steel, which was less than that of hot-rolled structural steel due to plastic deformation during forming, was not sufficient to overcome the stress concentration effects and redistribute the stresses across the net section. As a result, it was concluded that the nominal stress across the net section at failure could not reach the ultimate tensile strength of the material. For bolted connections failing by net section fracture, Winter plotted the ratio of the nominal stress across the net section at ultimate load \( f_{net} \) to the material ultimate tensile strength \( f_u \) against the \( d/p_2 \) ratio, and derived the following empirical equation:

\[ f_{net} = k_{r,Winter} f_u \] (4.7)

where \( k_{r,Winter} \) is the reduction factor given by Equation 4.8.

\[ k_{r,Winter} = \left( 0.1 + 3 \frac{d}{p_2} \right) \leq 1.0 \] (4.8)

Popowich (1969) also studied cold-formed carbon steel connections and suggested that the reduction factor recommended by Winter (1956) accounts adequately for the stress concentration caused by the presence of the hole in the plate but does not distinguish between connections with different numbers of bolts in one row. Popowich (1969) concluded that, in addition to the basic stress concentration which arises from the existence of a hole in the plate, the application of the bolt load locally at the bolt hole (similar to a point load) is another source of stress concentration and should be
considered in the stress reduction factor. Hence, a modification to Winter’s formula was proposed, redefining the reduction factor as:

\[
k_r = 1 - 0.9r + 3r \frac{d}{p_2} \leq 1.0
\]  

(4.9)

The minimum values of the key dimensions, as illustrated in Figure 4.4, covered by the rules in current design codes are compared in Table 4.1.

![Figure 4.4: Key dimensions in bolted lap connections](image)

<table>
<thead>
<tr>
<th>Reference</th>
<th>End distance (e_1)</th>
<th>Edge distance (e_2)</th>
<th>Spacing (p_1)</th>
<th>Spacing (p_2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCI/Euro Inox (1996)</td>
<td>1.5 d</td>
<td>1.5 d</td>
<td>2.5 d</td>
<td>3.0 d</td>
</tr>
<tr>
<td>ASCE (2002) and AS/NZS (2001)</td>
<td>1.5 d</td>
<td>1.5 d</td>
<td>3.0 d</td>
<td>3.0 d</td>
</tr>
<tr>
<td>EN 1993-1-4 (2006) and</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI/Euro Inox (2006)</td>
<td>1.2 (d_0)</td>
<td>1.2 (d_0)</td>
<td>2.2 (d_0)</td>
<td>2.4 (d_0)</td>
</tr>
</tbody>
</table>

Table 4.1: Minimum values of key dimensions in stainless steel lap bolted connections in design standards
4.3 PARAMETRIC STUDIES

4.3.1 General

The number of available test results for stainless steel bolted connections that failed in the net section is insufficient to fully investigate the underlying structural behaviour and to study the effects of the key parameters featured in the design equations, e.g. the edge distance $e_2$ and the ratio $r$, which, as shown in Equation 4.2, depends on the number and the arrangement of the bolts. In order to investigate these parameters, the data set has been extended by generating further results using the validated FE models. The same two types of stainless steel considered in Chapter 3 were investigated – the austenitic grade 1.4306 and the ferritic grade 1.4016, though these grades may be considered to be representative of the wider austenitic and ferritic families, respectively, of stainless steel. The material properties of the stainless steels employed in the parametric studies are shown in Table 4.2. The bolts material was considered elastic. EN 1993-1-3 (2006) provides design expressions for bolted connections between thin plates with thicknesses of 3.0 mm or less. This definition of thickness range was adopted in the parametric study; 8.0 mm and 10.0 mm thick plates were investigated to represent thick plate connections, while 1.0 mm and 2.0 mm thick plates were considered to represent thin plate connections. The geometry of the FE models was chosen in order to investigate net section failure for a wide range of $e_2/d_0$ ratios. The failure criterion suggested in Chapter 3 was employed in the parametric studies to determine the net section capacity.
Table 4.2: Material properties of stainless steel for the parametric studies

<table>
<thead>
<tr>
<th>Steel</th>
<th>$f_y (\sigma_{0.2})$ (N/mm²)</th>
<th>$f_u$ (N/mm²)</th>
<th>Elongation at fracture (%)</th>
<th>$f_u/f_y$</th>
<th>$n_s$</th>
<th>$m_s$</th>
<th>$K$ (N/mm²)</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic grade 1.4306</td>
<td>288</td>
<td>581</td>
<td>62.0</td>
<td>2.02</td>
<td>6.58</td>
<td>2.73</td>
<td>1548</td>
<td>0.64</td>
</tr>
<tr>
<td>Ferritic grade 1.4016</td>
<td>262</td>
<td>522</td>
<td>51.0</td>
<td>1.99</td>
<td>8.16</td>
<td>2.76</td>
<td>1390</td>
<td>0.64</td>
</tr>
</tbody>
</table>
### 4.3.2 Connections composed of thick plates

To examine the effect of the key parameters (the edge distance $e_2$ and the ratio $r$) on the net section capacity of lap connections between thick plates, six configurations shown in Figures 4.5 and 4.6 were adopted with plate thicknesses of 8.0 mm and 10.0 mm. These configurations correspond to four values of $r$ (1/2, 1/3, 1/4 and 2/5). The edge distance ratio $e_2/d_0$ varied between 1.2 and 4.0 and the bolts used were 20.0 mm diameter.

Figures 4.7 to 4.18 show the results of the parametric studies for the thick plate connections. It is clear that for stainless steel bolted connections composed of thick plates failing in net section rupture, the nominal stress $f_{net}$ at the critical net section at failure, given by Equation 4.10, is consistently greater than or equal to the material ultimate tensile strength (Salih et al. (2009b)).

\[
 f_{net} = \frac{F_{net,FE}}{A_{net}} \quad (4.10)
\]

where $F_{net,FE}$ is the net section failure load obtained from the parametric studies.

Note that the ratio of $f_{net}/f_u$ is generally greater than unity in Figures 4.7 to 4.18 and 4.21 to 4.32; this has also been observed in other similar experimental studies, and it is believed to be attributed to restraint from the material immediately adjacent to the net cross-section.
Figure 4.5: Single-row bolted connection configuration employed in the double shear parametric studies
Figure 4.6: Two-row bolted connection configuration employed in the double shear parametric studies
CHAPTER 4 – NET SECTION FAILURE

Figure 4.7: FE model results for austenitic bolted connections between thick plates (two bolts in one row – $r = 1/2$)

EN 1993-1-4 (2006) and SCI/Euro Inox (2006) ($\gamma_{M2} = 1.0$)

Figure 4.8: FE model results for ferritic bolted connections between thick plates (two bolts in one row – $r = 1/2$)

EN 1993-1-4 (2006) and SCI/Euro Inox (2006) ($\gamma_{M2} = 1.0$)
CHAPTER 4 – NET SECTION FAILURE

**Figure 4.9**: FE model results for austenitic bolted connections between thick plates (three bolts in one row – \( r = 1/3 \))

**Figure 4.10**: FE model results for ferritic bolted connections between thick plates (three bolts in one row – \( r = 1/3 \))
Figure 4.11: FE model results for austenitic bolted connections between thick plates (four bolts in one row – \( r = 1/4 \))

Figure 4.12: FE model results for ferritic bolted connections between thick plates (four bolts in one row – \( r = 1/4 \))
CHAPTER 4 – NET SECTION FAILURE

Figure 4.13: FE model results for austenitic bolted connections between thick plates (six bolts in two rows – r = 1/3)

Figure 4.14: FE model results for ferritic bolted connections between thick plates (six bolts in two rows – r = 1/3)
Figure 4.15: FE model results for austenitic bolted connections between thick plates (eight bolts in two rows – $r = 1/4$)

Figure 4.16: FE model results for ferritic bolted connections between thick plates (eight bolts in two rows – $r = 1/4$)
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Figure 4.17: FE model results for austenitic bolted connections between thick plates (five bolts – \( r = 2/5 \))

EN 1993-1-4 (2006) and SCI/Euro Inox (2006) \( (\gamma_{M2} = 1.0) \)

\[ \Delta \text{ Thickness } = 8.0 \text{ mm} \]
\[ \square \text{ Thickness } = 10.0 \text{ mm} \]

Figure 4.18: FE model results for ferritic bolted connections between thick plates (five bolts – \( r = 2/5 \))

EN 1993-1-4 (2006) and SCI/Euro Inox (2006) \( (\gamma_{M2} = 1.0) \)

\[ \Delta \text{ Thickness } = 8.0 \text{ mm} \]
\[ \square \text{ Thickness } = 10.0 \text{ mm} \]
Figure 4.19 shows a typical deformed FE model from the parametric studies. A substantial strain concentration at the edge of the bolt hole as well as the significant necking at the critical net section can be clearly observed.

![High strains due to stress concentration](image)

**Figure 4.19: Typical FE model failing in net rupture from the parametric studies**

### 4.3.3 Connections composed of thin plates

While the provisions for net section capacity in EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (2006) do not distinguish between single and double shear connections, ASCE (2002) and AS/NZS (2001) recommend two design equations: one for single shear connections and one for double shear connections. Therefore, the behaviour of single and double shear connections between thin sheets has been studied and compared. The same parameters, which were investigated for thick plate connections, were considered: the edge distance \( e_2 \), the ratio \( r \) and the plate thickness \( t \). Three configurations were adopted corresponding to three values of \( r \) (1/2, 1/3 and 1/4) with an edge distance ratio \( e_2/d_0 \) varying from 1.2 to 3.4 and bolts of 20.0 mm diameter. The configurations shown in Figure 4.5 were adopted for the double shear connections, while the configurations in Figure 4.20 were used for the single shear connections.

The results of the parametric studies for double shear connections between thin sheets are shown in Figures 4.21 to 4.26. It can be seen that the nominal stress at the critical net section at failure, given by Equation 4.10, is greater than or equal to the material ultimate tensile strength. For single shear connections, as shown in Figures 4.27 to 4.32 the ratio of nominal stress to ultimate tensile strength across the critical net section at failure for some values of edge distance \( e_2/d_0 \) ratios is slightly less than 1.0 – the smallest value of \( f_{net}/f_u \) is 0.96.
Figure 4.20: Single-row bolted connection configuration employed in the single shear parametric studies
Figure 4.21: FE model results for austenitic double shear connections of thin plates (two bolts in one row – $r = 1/2$)

Figure 4.22: FE model results for ferritic double shear connections of thin plates (two bolts in one row – $r = 1/2$)
Figure 4.23: FE model results for austenitic double shear connections of thin plates (three bolts in one row – $r = 1/3$)

Figure 4.24: FE model results for ferritic double shear connections of thin plates (three bolts in one row – $r = 1/3$)
Figure 4.25: FE model results for austenitic double shear connections of thin plates (four bolts in one row – $r = 1/4$)

Figure 4.26: FE model results for ferritic double shear connections of thin plates (four bolts in one row – $r = 1/4$)
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Figure 4.27: FE model results for austenitic single shear connections of thin plates (two bolts in one row – $r = 1/2$)

Figure 4.28: FE model results for ferritic single shear connections of thin plates (two bolts in one row – $r = 1/2$)
**Figure 4.29:** FE model results for austenitic single shear connections of thin plates (three bolts in one row – $r = 1/3$)

**Figure 4.30:** FE model results for ferritic single shear connections of thin plates (three bolts in one row – $r = 1/3$)
Figure 4.31: FE model results for austenitic single shear connections of thin plates (four bolts in one row – \( r = 1/4 \))

Figure 4.32: FE model results for ferritic single shear connections of thin plates (four bolts in one row – \( r = 1/4 \))
In order to investigate the applicability of the proposals of Winter (1956) and Popowich (1969) to hot-rolled and cold-formed stainless steel bolted connections, a study of the strain distribution at the critical net section was conducted. The strain distributions obtained from the FE models for connections with the same edge distance \(e_2\) but different \(r\) ratios were compared. Figures 4.33 and 4.34 show that the ratio \(r\) does not have a significant effect on the strain distribution at the net section at failure in stainless steel connections. A comparison of the strain distribution at the net section at failure for connections with the same \(r\) ratio but different edge distances \(e_2\) is shown in Figures 4.35 and 4.36. Although the non-uniformity of strain distribution increases with an increase of the edge distance, the ductility of stainless steel is sufficient for the stresses across the net section to be redistributed, thereby eliminating the deleterious effect of stress concentrations.

4.4 PROPOSED DESIGN RULES

4.4.1 General

The results of the parametric studies discussed in the previous sections demonstrate that for stainless steel bolted connections composed of either thick or thin plates failing by net section rupture, the nominal stress at the critical net section at failure is generally greater than the material ultimate tensile strength, though for some models the nominal stress is slightly less than the material ultimate tensile strength. This suggests that the ultimate material tensile strength can be used without including a reduction factor, and hence the ultimate net section capacity \(N_{u,\text{prop}}\) can be given by Equation 4.11. This conclusion is consistent with the findings of Rogers and Hancock (1998, 1999), who observed that the provisions for the net section failure in ENV 1993-1-3 (1996) and the AISI Specifications (1996), which include a reduction factor, are conservative. They proposed a design equation for net section rupture that does not contain a reduction factor (Equation 4.11). Their proposal has been adopted in the recent revisions of the AISI Specification.

\[
N_{u,\text{prop}} = \frac{A_{\text{net}} f_u}{\gamma_{M2}} \tag{4.11}
\]
**CHAPTER 4 – NET SECTION FAILURE**

**Figure 4.33:** Strain distribution at failure along net section for austenitic bolted connections with different values of $r$

**Figure 4.34:** Strain distribution at failure along net section for ferritic bolted connections with different values of $r$
CHAPTER 4 – NET SECTION FAILURE

Figure 4.35: Strain distribution at failure along net section for single-row austenitic bolted connections with four bolts

Figure 4.36: Strain distribution at failure along net section for single-row ferritic bolted connections with four bolts
4.4.2 Net section design equation proposed by Kim et al. (2008)

Kim et al. (2008) suggested that net section capacity for connections susceptible to curling depends on the arrangement of the bolts. The proposed net section capacity $N_{Kim}$ is given by the following equation:

$$N_{Kim} = b_e t f_u$$

(4.12)

where $b_e$ is the effective width of the connected plate, which depends on the number and arrangement of the bolts, and is given as by Equations 4.13 to 4.16. The four cases are illustrated in Figure 4.37.

for case a : $b_e = 2d \leq b_w - d_0$  \hspace{1cm} (4.13)

for case b : $b_e = 4d \leq b_w - d_0$  \hspace{1cm} (4.14)

for case c : $b_e = 2d + p_2 - d_0 \leq b_w - 2d_0$  \hspace{1cm} (4.15)

for case d : $b_e = 4d + p_2 - d_0 \leq b_w - 2d_0$  \hspace{1cm} (4.16)

where $b_w$ is the plate width.

The effective width $b_e$ of the connected plates according to Kim et al. (2008) – Equations 4.13 to 4.16 – for the connections between thin sheets employed in the parametric studies performed herein in Section 4.3.3, is always equal the actual plate width, and therefore, Kim et al.’s (2008) design expression (Equation 4.12) returns to the proposed design expression of Equation 4.11.
4.4.3 Statistical validation

In order to verify the net section fracture prediction of Equation 4.11 and to determine an appropriate partial safety factor $\gamma_{M2}$ which is applied to fracture resistance in EN 1993-1-1 (2005), statistical analyses in accordance with Annex D of EN 1990 (2002) were performed.

In applying the method of Annex D of EN 1990 (2002), many previous investigations (Rebelo et al., 2009; Chan and Gardner, 2009) have utilised both test and FE data, as opposed to the results of experiments only. This same approach has been adopted in the present study. However, the inherent scatter associated with nominally repeated tests is absent in repeated FE simulations. This could be accounted for by increasing the coefficient of variation of the design model $V_\delta$ in step 4 below. This issue requires further research.

Figure 4.37: Definition of the effective width in net section failure defined by Kim et al. (2008)
CHAPTER 4 – NET SECTION FAILURE

Step 1: Developing a design model

A theoretical design model that is a function of a number of relevant independent variables \( X \) is firstly established. The net section design model \( r_t \), which is given by Equation 4.11, can be written in terms of the basic independent variables as follows:

\[
r_t = g_n(X) = A_{net} f_u = t(b_w - n_b d_0) f_u
\]

where \( t \) is the thickness of the plate, \( b_w \) is the width of the plate, \( d_0 \) is the bolt hole diameter, \( n_b \) is the number of bolts at the critical cross-section and \( \gamma_{M2} \) is the partial safety factor.

Step 2: Comparison between the experimental and theoretical net section capacities

The theoretical values of net section resistance for all tests and FE model results are obtained by substituting the actual measured properties into the resistance function in Equation 4.17. These values were compared with the corresponding experimental ultimate net section capacity \( r_e \) to investigate the deviation from the line \( r_e = r_t \). A comparison for test data is plotted in Figure 4.38, while a comparison with both test and FE data is shown in Figure 4.39.

Step 3: Estimation of the mean value of factor \( b \)

The probabilistic model of the resistance, \( r \) is given as follows:

\[
r = br_t \delta
\]

where \( b \) is the slope of the least squares regression line that can be obtained using Equation 4.19, and \( \delta \) is an error term which gives information on the scatter of the points from the mean value of the strength function.
where \( r_{ei} \) and \( r_{ti} \) are the experimental and theoretical net section resistance for the \( i^{th} \) specimen and \( n \) is total number of specimens. The values of \( b \) in the current study are shown in Table 4.3. Values of \( b \) being greater than unity indicate that, on average, the proposed design equation underpredicts the test or FE models.

### Step 4: Estimation of the coefficient of variation of the errors of the design model

An estimated value of the coefficient of variation \( V_\delta \) of the design model error is given in a log-normal distribution as follows:

\[
V_\delta = \sqrt{\exp(s^2_\Delta) - 1} \quad (4.20)
\]

\[
s^2_\Delta = \frac{1}{(n-1)} \sum_{i=1}^{n} (\Delta_i - \overline{\Delta})^2 \quad (4.21)
\]

\[
\overline{\Delta} = \frac{1}{n} \sum_{i=1}^{n} \Delta_i \quad (4.22)
\]

\[
\Delta_i = \ln(\delta_i) \quad (4.23)
\]

\[
\delta_i = \frac{r_{ei}}{br_{ti}} \quad (4.24)
\]

where \( s^2_\Delta \) is the variance of the error terms, \( \overline{\Delta} \) is the average value of the error terms, \( \Delta_i \) is the \( i^{th} \) error in the log-normal distribution and \( \delta_i \) is the error term for the \( i^{th} \) specimen.
Step 5: Analysis of compatibility

The degree of scatter of the test data about the regression line \( r_e = b r_i \) is measured using the coefficient of determination \( R^2 \). This coefficient measures the quality of the approximation – the quality of the regression increases as \( R^2 \) approaches 1.0.

\[
R^2 = \frac{ss_{te}}{ss_{tt} ss_{ee}} \leq 1.0 \quad (4.25)
\]

where

\[
ss_{tt} = \sum_{i=1}^{n} (r_{ti} - \bar{r}_t)^2 \quad (4.26)
\]

\[
ss_{ee} = \sum_{i=1}^{n} (r_{ei} - \bar{r}_e)^2 \quad (4.27)
\]

\[
ss_{te} = \sum_{i=1}^{n} (r_{ti} - \bar{r}_t)(r_{ei} - \bar{r}_e) \quad (4.28)
\]

where \( \bar{r}_t \) and \( \bar{r}_e \) are the mean values of the theoretical and experimental net section resistance, respectively, for all specimens.

The coefficients of determination \( R^2 \) for both sets are shown in Table 4.3. These values indicate that the proposed design equation accurately describes the net section failure response.

Step 6: Determination of the coefficient of variation of the basic variables

The uncertainty of the independent variables in the net section design model, including the steel ultimate tensile strength and the geometrical properties should also be taken into account. These variables are assumed to be uncorrelated log-normally distributed (EN 1990, 2002). The variability \( V_{Xi} \) of each independent variable was determined on the basis of prior knowledge as follows:
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Coefficient of variation for plate thickness $V_t = 0.05$  
(Može et al., 2007)

Coefficient of variation for plate width $V_b = 0.005$  
(Može et al., 2007)

Coefficient of variation for bolt hole diameter $V_{d_h} = 0.005$  
(Može et al., 2007)

Coefficient of variation for tensile strength $V_{f_t} = 0.027$  
(Groth and Johanson, 1990)

**Step 7: Determination of the design value of the resistance**

The design value of the resistance $r_d$ function should be obtained from either Equation 4.29 or 4.30, depending on the number of available test or FE results.

$$r_d = b g_{n} (X_m) \exp (-k_{d,n} \alpha_{n} Q_{n} - k_{d,n} \alpha_{Q_{n}} - 0.5 Q^2) \quad \text{for} \quad n < 100$$ (4.29)

$$r_d = b g_{n} (X_m) \exp (-k_{d,\infty} Q - 0.5 Q^2) \quad \text{for} \quad n \geq 100$$ (4.30)

where

$$Q_{n} = \sigma_{ln(r)} = \sqrt{\ln(V_{n}^2 + 1)}$$ (4.31)

$$Q_{\delta} = \sigma_{ln(\delta)} = \sqrt{\ln(V_{\delta}^2 + 1)}$$ (4.32)

$$Q = \sigma_{ln(r)} = \sqrt{\ln(V_{r}^2 + 1)}$$ (4.33)

$$\alpha_{n} = \frac{Q_{n}}{Q}$$ (4.34)

$$\alpha_{\delta} = \frac{Q_{\delta}}{Q}$$ (4.35)

where $X_m$ is the mean values of the basic variables, $k_{d,n}$ is the design fractile factor, $k_{d,\infty}$ is the value of $k_{d,n}$ for $n = \infty$ , $\alpha_{n}$ is the weighting factor for $Q_{n}$, $\alpha_{\delta}$ is the weighting factor for $Q_{\delta}$.
The prediction of the overall error $V_e^2$ for small values of $V_0^2$ and $V_{Xi}^2$ can be approximated using the following equations (EN 1990, 2002):

$$V_e^2 = V_0^2 + V_{pi}^2 \quad (4.36)$$

$$V_{pi}^2 = \sum_{i=1}^{j} V_{Xi}^2 \quad (4.37)$$

where $j$ is the total number of independent variables.

**Step 8: Determination of the partial safety factor**

The safety factor $\gamma_{M2}$ for the proposed design equation is the ratio between the nominal resistance $r_n$ and the design resistance $r_d$:

$$\gamma_{M2} = \frac{r_n}{r_d} \quad (4.38)$$

The nominal resistance $r_n$ is obtained by substituting the nominal values of all the basic variables $X_n$ into the theoretical resistance function as follows:

$$r_n = g_n(X_n) \quad (4.39)$$

The ratio of mean to nominal ultimate tensile strengths (i.e. the material over-strength) has been taken as 1.20 (Groth and Johansson; 1990), while the ratios of mean to nominal values of the geometrical parameters ($t$, $b_w$ and $d_o$) were assumed to be 1.0.
**Figure 4.38:** Comparison between theoretical and experimental resistances

- $r_e = 1.06 \times r_t$
- $R^2 = 0.989$
- $r_e = r_t$

**Figure 4.39:** Comparison between theoretical and experimental and numerical resistances

- $r_e = 1.06 \times r_t$
- $R^2 = 0.999$
- $r_e = r_t$
CHAPTER 4 – NET SECTION FAILURE

Table 4.3: Summary of statistical analysis results

<table>
<thead>
<tr>
<th>Data set</th>
<th>Number of specimens</th>
<th>k_{d,n}</th>
<th>b</th>
<th>R^2</th>
<th>V_δ</th>
<th>V_r</th>
<th>γ_{M2}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test results</td>
<td>17</td>
<td>3.79</td>
<td>1.06</td>
<td>0.989</td>
<td>0.0403</td>
<td>0.0699</td>
<td>1.002</td>
</tr>
<tr>
<td>Test + FE results</td>
<td>345</td>
<td>3.12</td>
<td>1.06</td>
<td>0.999</td>
<td>0.0288</td>
<td>0.0639</td>
<td>0.959</td>
</tr>
</tbody>
</table>

The maximum required partial factor, given in the final column of Table 4.3, for the proposed nominal resistance of the net section is 1.002, hence, it can be safely taken as 1.1, which is less than the recommended value of 1.25 in Eurocode 3 but equal to the value recommended for hot-rolled carbon steel in the UK National Annex to EN 1993-1-1 (2005). Therefore, the ultimate design capacity for net section failure of hot-rolled and cold-formed stainless steel bolted connections can be taken as:

\[
N_{u,\text{prop}} = \frac{A_{\text{net}} f_u}{\gamma_{M2}}
\]  

(4.40)

where \(\gamma_{M2} = 1.1\). From Figure 4.40 it can be seen that the design provisions for net section failure in Eurocode 3 and the SCI/Euro Inox Design Manual are overly conservative, and the proposed expression provides greater efficiency. Note that, to ensure a ductile failure of a tension member, the ultimate resistance of the net cross-section (Equation 4.40) should be greater than the yield resistance (gross area \(\times\) yield strength) of the gross section.
4.5 CONCLUDING REMARKS

The validated FE models were used to perform parametric studies to investigate the net section fracture behaviour of stainless steel connections under static shear load. The parametric studies, together with the test data, showed that the reduction factor $k_r$ employed in Eurocode 3 Part 1.4 and the SCI/EuroInox Design Manual is unnecessary. A design equation for the net section capacity of stainless steel connections, applicable to both thick and thin material, was proposed, and a suitable partial safety factor of 1.1 was derived following reliability analyses in accordance with EN 1990.

Figure 4.40: Comparison between the proposed design equations and those recommended in EN 1993-1-4 (2006) and the SCI/EuroInox (2006) for connections with four bolts in a single row
CHAPTER 5

BEARING FAILURE

5.1 INTRODUCTION

In this chapter, a thorough investigation of the bearing behaviour of bolted lap connections in stainless steel was performed by means of parametric studies. A comparison between the response of carbon steel and stainless steel connections shows that the bearing behaviour is different. A strain-based failure criterion is proposed to define ultimate bearing capacity. The results of the parametric studies were employed to propose bearing design equations for stainless steel connections for both thick and thin material.

5.2 BEARING FAILURE OF BOLTED LAP CONNECTIONS

After the load applied to a bolted lap connection overcomes the friction forces, a major slip takes place in the connection and consequently the bolt shank and the side of the bolt hole come into contact. Bearing stresses are developed at the contact surfaces. Initially, these stresses are concentrated at the tip of the bolt hole where contact initiates. Gradually, due to the yielding of the plate material the bolt shank embeds into the plate and the contact area increases. The actual bearing stress distribution is not known, yet in design, a simple uniform stress distribution $\sigma_b$ is assumed, which is given as follows:
\[ \sigma_b = \frac{P_{\text{bolt}}}{td} \]  

(5.1)

where \( P_{\text{bolt}} \) is the load transmitted by the bolt, \( t \) is the plate thickness and \( d \) is the bolt diameter.

Even though the bolt is subjected to the same magnitude of bearing stresses as the plate, because the tensile strength of bolt material is generally much greater than that of plate material, bearing is most critical in the plate material (Owens and Cheal, 1989). This is the most common situation for typical connections in steel. Owens and Cheal (1989) concluded that bolt bearing failure is only possible for connections with extremely high strength plates. As this situation is not common in practice, bearing failure of the plate normally governs bearing design and, therefore, in the current study only the bearing of the connected plate is considered.

The form of failure in bearing depends on geometrical factors; these are the end distance, the edge distance, the bolt diameter and the thickness of the connected ply. Either a tear-out of the bolt through the end of the plate occurs due to insufficient end distance as shown in Figure 5.1(a) or excessive deformations are developed in the plate material before fracture occurs at the edge of bolt hole as indicated in Figure 5.1(b). Winter (1956) observed that end tear-out was characterised by horizontal shearing of the sheets along two parallel planes separated by a distance equal to the bolt diameter, while for bearing failure, tearing along two distinctly inclined planes was observed with considerable pilling-up of material in front of the bolt.

A third form of bearing failure known as ‘bursting’ failure was observed by Owens et al. (1976a). It was found that for plates with small widths, the mean stress over the net section is approximately equal to the yield stress. As a result the resistance to transverse deformation (splay), as illustrated in Figure 5.1(c), is greatly reduced enabling fracture to initiate transversely at the outer edge of the plate in front of the bolt (see Figure 5.1(c)). Resistance to this form of failure can be less than the other two forms of bearing failure.
The bearing resistance of shear bolted connections has been determined in previous studies either on the basis of a strength or a deformation criterion. These two criteria are discussed in this section.

5.3.1 Strength criterion

For this criterion, the bearing capacity of a bolted connection is taken as the maximum load attained in the test regardless of the associated deformation. Many researchers (Winter, 1956; Errera et al., 1974; Dhalla et al., 1971; Rogers and Hancock, 1998; Puthli and Fleischer, 2001; Brown et al., 2007) who conducted experimental studies adopted this criterion to develop bearing design equations even though large deformations were
observed at the ultimate load. For instance, Rogers and Hancock (1998) developed a bearing design equation for cold-formed carbon steel bolted connections by adopting the maximum loads from their tests, even though in many specimens a level of deformation as large as 15 mm was reached. In the above-mentioned studies and in Kim’s (1996) experimental work, the researchers observed that bearing failure was initiated by fracture at the edge of the elongated bolt hole at two symmetrical locations oriented at approximately \( \theta = 45^\circ \) and \( 135^\circ \) (see Figures 5.1(b) and 5.6), which indicates the occurrence of peak strains at these locations as will be discussed later in this chapter.

### 5.3.2 Deformation criterion

The bearing resistance of a connection according to this second criterion is taken as the applied load measured at a pre-specified acceptable deformation depending on the usage of the connections. This limit does not correspond to the maximum load attained in the test and hence, no rupture takes place in the material. The determination of bearing failure of bolted shear connections by limiting deformations was proposed by many researchers. However, there is no consensus about whether to limit the permanent or the total elongation, nor on the value to adopt as a suitable deformation limit. Perry (1981) investigated carbon steel bolted connections, and recommended that the failure load be the load corresponding to a deformation of 6.35 mm since, beyond this level, the load-deflection curves of typical connections become virtually flat. Perry’s (1981) definition has been adopted in developing design guidance for carbon steel connections in the AISC (2005) Specification. The SCI/Euro Inox (1996, 2006) design provisions for stainless steel connections were developed on the basis of a 3.0 mm deformation limit defining the ultimate conditions. By imposing this limit, it was suggested that the deformation at the service loads would be of the order of 1.0 mm. ENV 1993-1-4 (1996) and EN 1993-1-4 (2006) adopted the same design provisions.

Ryan (1999a) proposed a deformation criterion for defining the bearing capacity of stainless steel connections. It was suggested that ultimate load be defined as the load corresponding to 5.0 mm permanent deformation and that the service load be defined as the load corresponding to 1.75 mm permanent deformation. It was concluded that the proposal in ENV 1993-1-4 (1996) for using a reduced ultimate tensile steel strength in
the determination of bearing resistance is acceptable for austenitic and ferritic stainless steels, but requires further investigation for duplex stainless steel.

In order to investigate the difference in the deformation behaviour of carbon steel and stainless steel bolted connections, an FE model was developed with the material behaviour of austenitic stainless steel and the geometry of a carbon steel connection tested by Kim (1996), which is shown in Figure 5.2. Figure 5.3 shows that for the carbon steel connection, the load-deflection response attained a relatively flat state once significant bearing deformations had developed, and therefore, the exact deformation limit selected has relatively little effect on the ‘failure load’. However, for the stainless steel connection, owing to its rounded stress-strain relationship and the pronounced strain hardening with increased deformation, a rising relationship, without significant flattening off was obtained. Thus, selection of different deformation limits will significantly affect the load corresponding to bearing failure. For instance, in this example, the increase of the load carrying capacity of the stainless steel connection based on bearing fracture compared to that at 6.35 mm deformation is 26 %, while this value for carbon steel connection is just 9%.

**Figure 5.2:** Carbon steel connection tested by Kim (1996) and modelled in the current study by employing stainless steel material properties
5.4 PARAMETRIC STUDIES

5.4.1 General

In order to investigate the bearing behaviour of stainless steel bolted connections so as to modify the current bearing design guidance, the validated FE models have been employed to conduct parametric studies. The results of these studies will be used in Section 5.6 to propose bearing design equations for connections between either thick or thin plates. The two types of stainless steel considered in Chapter 4 were investigated – the austenitic grade 1.4306 and the ferritic grade 1.4016. The material properties of the stainless steels that were employed in the parametric studies are given in Table 4.2 in Chapter 4. The material of the bolts was considered elastic. Bolt threads were not explicitly modelled since they were found to have no significant effect on either net section or bearing failure, and caused undue numerical convergence difficulties. Instead, a uniform diameter of bolt shank was assumed. The investigated parameters include the end distance $e_1$, the edge distance $e_2$ and the thickness of the plate $t$, with a constant bolt

Figure 5.3: Bearing behaviour of carbon steel and stainless steel bolted connections with identical geometry
diameter of 20 mm. It should be noted that values of edge distance ratios \( e_2/d_0 \) less than 1.5 were not investigated, because for this configuration net section fracture was found to occur prior to bearing failure, and consequently the bursting form of bearing failure identified by Owens et al. (1976a) was not observed.

Two additional phenomena are associated with the bearing behaviour of thin sheet connections as compared to thick plate connections: curling and pulling into line. Curling is the out-of-plane deformation of the connected sheet in front of the end bolt. This deformation, which occurs in both sheets in single shear connections and in the outer sheets of double shear connections, is effectively buckling of the plate when it is subjected to compressive stresses (Kim et al., 2008). The part of the plate in front of the end bolt can be regarded as a strut that is fixed at one end by means of the bolt head or nut and free at the other end (see Figure 5.5). Pulling into line only takes place in single shear connections to enable the applied tensile loads, which are initially acting at an eccentricity to one another and inducing bending of the plate, to act along the same line, as demonstrated in Figure 5.5(b).

Eurocode 3 Part 1.3 (2006) provides design expressions for bolted connections between thin plates with thicknesses of 3.0 mm or less. This 3.0 mm limit has also been adopted in the present study to demark the transition between ‘thick’ and ‘thin’ material; hence, 8.0 mm and 10.0 mm thick plates were investigated to represent thick plate connections, while plates with thicknesses 1.0 mm and 2.0 mm were used to investigate thin plate connections.

### 5.4.2 Connections composed of thick plates

In this group, lap connections with bolts in double shear with plate thicknesses of 8.0 mm and 10.0 mm were investigated. The arrangement of the FE models is shown in Figure 5.4(a). A wide range of end distance ratios \( e_1/d_0 \) – varied from 0.8 to 4.0 was investigated with four values of the edge distance ratio \( e_2/d_0 \) (1.5, 2.0, 3.0 and 4.0).
CHAPTER 5 – BEARING FAILURE

(a): Double shear connections – inner plate critical

(b): Double shear connections – outer plates critical

(c): Single shear connections

Figure 5.4: Configuration of FE models used in the parametric studies
A typical distribution of plastic strain in the plate in front of the bolt obtained from the parametric study is shown in Figure 5.6(a). It shows that the strains are very high at two symmetrical locations at about $\theta = 45^\circ$ and $135^\circ$. This strain distribution agrees with the observations in the experimental studies discussed in Section 5.3.1 and confirms that bearing fracture occurs at these locations. This conclusion will be adopted to determine the bearing capacity when using the strength criterion: when the peak plastic strain in the plate material in front of the bolt reaches the localized fracture strain of the material (Salih et al., submitted), fracture occurs and the maximum load is said to have been reached.

Figure 5.7 shows the stiffness at three stages of loading of a stainless steel connection which failed by bearing, where deformation is measured as the elongation between points A and B, as shown in Figure 5.4. The stiffness at the point of bearing fracture is almost equal to the stiffness at 6.35 mm deformation and is about 50% of the stiffness at 3.0 mm deformation. For a connection failing by net section fracture as shown in Figure 5.9, the stiffness at fracture is about 12% of that at 3.0 mm deformation and 25% of that at 6.35 mm deformation. It is clear that for connections failing by bearing the loss of stiffness at bearing fracture is relatively modest when compared to the loss of stiffness at net section fracture in connections failing by net section rupture. Thus, it may be concluded that defining bearing failure on the basis of deformation limits (3.0 mm or 6.35 mm), underestimates the true bearing resistance of stainless steel connections. For consistency with net section failure behaviour, in which the ultimate load is taken as the load causing fracture, the load at bearing fracture can be considered as the bearing ultimate load.

5.4.3 Connections composed of thin sheets

The bearing behaviour of bolted connections composed of thin sheets was thoroughly examined by considering three scenarios. The first scenario addresses the case of double shear connections in which the inner sheet is critical. In this case, due to the restraint provided by the outer sheets, curling will be prevented. The second scenario again considers double shear connections, but those in which the outer sheets are critical and
curling is expected. The third scenario covers bolted connections in single shear where both curling and pulling into line take place.

5.4.3.1 Inner sheets in double shear connections

A set of FE models for the double shear configurations shown in Figure 5.4(a) has been investigated with plate thicknesses of 1.0 mm and 2.0 mm. The end distance ratios $e_1/d_0$ varied from 0.8 to 4.0 with three values of the edge distance ratio $e_2/d_0$ (1.5, 2.0 and 3.0). The bearing response in this situation is found to be similar to that for thick plate connections. The stiffness for a connection with thin plates ($t = 2$ mm) at different load levels shown in Figure 5.8 exhibits a rising load-deformation behaviour. Figure 5.6(b) shows that the distribution of strains in front of the bolt has the same peak values at about $\theta = 45^\circ$ and $135^\circ$. The overall response of the inner sheets in double shear connections may therefore be said to be insensitive to the material thickness.

5.4.3.2 Single shear connections and outer sheets in double shear connections

In order to investigate the effects of curling and pulling into line on the bearing behaviour, the load-deformation curves from three cases from the FE models of austenitic connections were compared. The thickness of the investigated sheet is 2.0 mm and the edge distance $e_1/d_0$ ratio is 4.0. These models can be described as follows:

- Model I: double shear connection as shown in Figure 5.4(b) where out-of-plane deformation (curling) of the outer sheets is prevented by applying appropriate boundary conditions.
- Model II: double shear connection as shown in Figure 5.4(b) where out-of-plane deformation (curling) of the outer sheets is permitted.
- Model III: single shear connection as shown in Figure 5.4(c) without restraining the out-of-plane deformation (curling).

Figure 5.10 shows a comparison between the results of the above-mentioned connections, while Figure 5.5 compares the deformed shapes for Models II and III. It can be concluded that while curling, which occurs in both single and double shear
connections, significantly affects the load carrying capacity of the connections, pulling into line, which takes place in single shear connections only, does not affect the bearing behaviour. Thus, Models II and III are effectively equivalent. This behaviour has been previously observed. For instance, Kuwamura and Isozaki (2001a, 2001b) studied the tensile behaviour of lap connections between thin sheets experimentally and then Kim and Kuwamura (2007) employed numerical models to simulate these tests. The test setup consisted of single bolted connections between thin sheets (1.5 mm or 3.0 mm) which were considered to be the test specimens, and thick plates (6.0 mm) in which no significant deformation occurred. When these tests were replicated using FE models (Kim and Kuwamura, 2007), only the thinner sheet and bolts were modelled. This indicates that only the curling phenomenon was thought to affect the behaviour of bolted connections between thin sheets. Moreover, the AISI Specification (1996) provides bearing design equations for bolted connections between thin carbon steel sheets for two situations; the first covering the inner sheet in double shear connections, and the second covering single shear connections and the outer sheets in double shear connections.

The FE models of the single shear connections between thin sheets were frequently unstable preventing full solutions from being achieved. However, it was shown earlier that the behaviour of the outer sheet in double shear connections can also represent the behaviour in single shear; the arrangement in Figure 5.4(b) was therefore adopted to conduct parametric studies to represent both connection types. The end distance ratio varied from 0.8 to 4.0 with three values of the edge distance ratio $e_2/d_0$ (1.5, 2.0 and 3.0). Despite the occurrence of curling, the strain distribution in the plate in front of the bolt as shown in Figure 5.6(c) remains consistent with that previously observed with peak strains arising at two symmetrical points at approximately $\theta = 45^\circ$ and $135^\circ$. Hence, the originally proposed failure criterion remains valid – bearing fracture occurs when the peak strain reaches the material true fracture strain of 100%.
Figure 5.5: Deformed shape of connections susceptible to curling and pulling into line
Figure 5.6: Plastic strain distribution in the plate in front of the bolt

(a) Thick plate connection (t = 8 mm)

(b) Thin plate connection (inner sheet in double shear connection, t = 2 mm)

(c) Thin plate connection (outer sheet in double shear connection, t = 2 mm)
Figure 5.7: Stiffness of stainless steel connections between thick plates failing by bearing at different load levels

(a) Connection failing by bearing \((e_1/d_0 = 3.0)\)

(b) Connection failing by end tear-out \((e_1/d_0 = 1.2)\)
Figure 5.8: Stiffness of stainless steel connections between thin sheets failing by bearing at different load levels
Figure 5.9: Stiffness of stainless steel connections failing by net section rupture at different load levels

Figure 5.10: Comparison between thin sheet connections in single and double shear
5.4.4 Failure modes and connection geometry

In order to distinguish between end tear-out failure and bearing failure from the FE models, a failure criterion that depends on the deformation of the connection has been devised. In the FE models five reference points A, B, C, D, E and F were assigned to the plate as shown in Figure 5.11. The overall deformation of a connection is often considered as the elongation of the plate parallel to the direction of the load between points C and E. Four components of deformation contribute to the plate elongation to a different degree depending on the precise arrangement. The first is the relative horizontal displacement between points A and B. Since this deformation shows approximately the protrusion of the bolt from the plate material, it will be called ‘Bolt protrusion’. The second component is the shortening of the plate material in front of the bolt, which is measured by the relative horizontal displacement between points B and C. Because this part measures the amount of the bolt embedding into the plate material, it will be called ‘Bolt embedding’. The third component of deformation is the elongation in the net section that occurs as a result of the high stresses over the net section. This elongation is approximately twice the relative horizontal displacement between points D and F. The fourth component that contributes to the overall elongation is the elongation in the gross section and this is measured as a relative displacement between points D and E. The elongation in the gross section of the plate is very small relative to other sources of deformation as shown in Figures 5.12, 5.13 and 5.14. Therefore, in many previous studies (Perry, 1981; Kim, 1996) plate deformation is regarded as the hole elongation.

By examining the relative contributions of these components of deformation, three failure modes can be identified. When the ‘Bolt protrusion’ constitutes the majority of the deformation, as shown in Figure 5.12, the connection has failed by end tear-out. This mode takes place in connections with small end distances as can be seen in Figure 5.15(a). When the deformation of the plate is mainly due to the ‘Bolt embedding’ as shown in Figure 5.13, the failure mode is bearing (see Figure 5.15(b)). In connections with relatively small edge distance $e_2$, the deformation is essentially due to net section elongation as shown in Figure 5.14 – net section fracture is the mode of failure in this case (see Figure 5.16). Note that this criterion is used solely to distinguish between end-
tear out and bearing failure, while the failure load is determined by employing the strain-based criterion which has been discussed previously.

Figure 5.11: Reference points assigned to plates in the FE models
Figure 5.12: Components of hole elongation for connection failing by end tear-out
(see Figure 5.15(a))
Figure 5.13: Components of hole elongation for connection failing by bearing
(see Figure 5.15(b))
**Figure 5.14:** Components of hole elongation for connection failing in the net section (see Figure 5.16)
5.4.5 Comparison between FE models and Kulak et al.'s (1987) model

The criterion described in Section 5.4.4 has been employed to identify end-tear out and bearing failures as shown in Figures 5.17 to 5.20. In this section, the FE model results are compared with a limit that was proposed by Kulak et al. (1987).

Kulak et al. (1987) developed an analytical model to estimate the end distance required to prevent end tear-out. In their model, end tear-out was assumed to occur due to shearing of the plate along two horizontal lines, shown as dashed lines in Figure 5.1(a). Therefore, the model assumed that the end tear-out was resisted by shear stresses acting on these planes. The load on the bolt $P_{\text{bolt}}$ was given by:
CHAPTER 5 – BEARING FAILURE

\[ P_{\text{bolt}} = t d \sigma_b \]  \hspace{1cm} (5.2)

where \( t \) is the plate thickness, \( d \) is the nominal diameter of the bolt and \( \sigma_b \) is the nominal bearing stress.

The shear resistance \( P_{\text{shear}} \) of the plate along the failure planes (see Figure 5.1(a)) was given by:

\[ P_{\text{shear}} = 2 t \left[ e_1 - \frac{d_u}{2} \right] \tau_u \]  \hspace{1cm} (5.3)

where \( \tau_u \) represents the shear strength which for most steels is about 70% of the tensile strength. Hence, Equation 5.3 became

\[ P_{\text{shear}} = 2 t \left[ e_1 - \frac{d_u}{2} \right] (0.7 f_u) \]  \hspace{1cm} (5.4)

By equating Equations 5.2 and 5.4, a lower bound of the \( e_1/d_0 \) ratio that will prevent the end tear-out mode was found to be:

\[ \frac{e_1}{d_0} \geq 0.5 + 0.645 \left( \frac{\sigma_b}{f_u} \right) \]  \hspace{1cm} (5.5)

Equation 5.5 is plotted in Figures 5.17 to 5.20 (\( \sigma_{b,\text{frac}} \) is defined in Section 5.6.1) together with the typical results from the parametric studies. Similar results were obtained when Kulak et al. (1987) plotted Equation 5.5 against test results for carbon steel connections. It can be seen that the strain-based fracture criterion employed to define the ultimate bearing capacity coupled with the deformation criterion that distinguishes between end tear-out and bearing failure are able to accurately describe the occurrence and form of bearing failure.
Figure 5.17: End tear-out and bearing failures ($e_2/d_0 = 1.5$)

Figure 5.18: End tear-out and bearing failures ($e_2/d_0 = 2.0$)
\[ \frac{e_1}{d_0} = 0.5 + 0.645 \left( \frac{\sigma_b}{f_u} \right) \]

Figure 5.19: End tear-out and bearing failures \((e_2/d_0 = 3.0)\)

\[ \frac{e_1}{d_0} = 0.5 + 0.645 \left( \frac{\sigma_b}{f_u} \right) \]

Figure 5.20: End tear-out and bearing failures \((e_2/d_0 = 4.0)\)
CHAPTER 5 – BEARING FAILURE

5.5 CURRENT DESIGN PROVISIONS

5.5.1 General

All existing carbon steel and stainless steel design standards consider end tear-out and bearing failure as one limit state by providing a design equation that relates end tear-out capacity to the end distance, and then setting an upper limit for this equation. The general form of bearing capacity design expressions is given by:

\[ N_b = \alpha t df_u \leq C t df_u \]  \hspace{1cm} (5.6)

where \( \alpha \) is the bearing coefficient, which is linearly related to the end distance \( e_1 \) and \( C \) is the upper constant value of the coefficient \( \alpha \) for end distances \( e_1 \) equal to or greater than a limiting value.

5.5.2 Eurocode 3 and the SCI/Euro Inox Design Manual

The design bearing resistance of carbon steel connections in Eurocode 3 Part 1.8 (2005) is given by:

\[ N_{b,EC3} = \frac{k_1 \alpha_b t df_u}{\gamma_{M2}} \]  \hspace{1cm} (5.7)

where \( \alpha_b \) is the smallest of \( \alpha_d, \frac{f_{ub}}{f_u} \) (where \( f_u \) and \( f_{ub} \) are the ultimate tensile strengths of the plate and bolt material respectively) or 1.0, \( t \) is the plate thickness, \( d \) is the nominal bolt diameter and \( \gamma_{M2} \) is a partial safety factor with a recommended value of 1.25. In the direction of load transfer, \( \alpha_d = \frac{e_1}{3d_0} \) for end bolts and \( \left( \frac{p_1}{3d_0} - \frac{1}{4} \right) \) for inner bolts, where \( d_0 \) is the bolt hole diameter, \( e_1 \) is the end distance and \( p_1 \) is the spacing between bolts in the direction of loading. In the direction perpendicular to load transfer, \( k_1 \) is the smaller of \( \left( \frac{2.8e_2}{d_0} - 1.7 \right) \) or 2.5 for edge bolts and \( \left( \frac{1.4p_2}{d_0} - 1.7 \right) \) or 2.5 for inner bolts where \( e_2 \) is the edge distance. For single lap connections with only one row of bolts, Eurocode 3 Part 1.8 (2005) recommends to provide washers under both the head and the nut. In addition, the design bearing resistance for each bolt should be limited to:
ENV 1993-1-4 (1996), EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (1996, 2006) adopt Equations 5.7 and 5.8 for stainless steel connections with a slight modification: a reduced ultimate strength of the plate material $f_{u,\text{red}}$ obtained from Equation 5.10 is used in Equations 5.7 and 5.8 in place of $f_u$. This modification was proposed by the SCI/Euro Inox (1996) to limit bearing deformations at the ultimate and service loads to acceptable levels, while maintaining the format of the resistance equation and the bearing coefficients for carbon steel. These standards recommend these provisions for both thick and thin plated connections.

To maintain the same factors used for carbon steel connections, SCI/Euro Inox (1996) derived bearing design resistance according to the following steps:

1. For connections failing by bearing, the resistance of the connection was defined by adopting a deformation criterion: the load associated with 3.0 mm hole elongation $F_{b,3.0}$.
2. A material strength $f_{ur}$ was obtained for these specimens by adopting the bearing coefficients in EN 1993-1.8 (2005) as follows:

$$f_{ur} = \frac{F_{b,3.0}}{k, \alpha_b \ t \ d} \tag{5.9}$$

3. These values were plotted against a proposed reduced ultimate material strength given by Equation 5.10 to give a best-fit regression line as shown in Figure 5.21.

$$f_{u,\text{red}} = 0.5 f_y + 0.6 f_u \leq f_u \tag{5.10}$$

4. SCI/Euro Inox recommended that the reduced ultimate strength of the plate material $f_{u,\text{red}}$ obtained from Equation 5.10 was used in Equations 5.7 and 5.8 in place of $f_u$. 

$$N_{b,\text{EC3}} = \frac{1.5 \ t \ d f_u}{\gamma M_2} \tag{5.8}$$
The result obtained by SCI/Euro Inox (1996) by following these steps is shown in Figure 5.21. It demonstrates that the correlation between the calculated material tensile strength $f_{ur}$ and the proposed reduced ultimate tensile strength $f_{u,red}$ is fairly poor with a coefficient of determination $R^2$ of 0.69, which suggests that the proposed reduced ultimate strength $f_{u,red}$ does not correctly represent the required material strength (i.e. the outcome of Equation 5.9). An alternative design approach is proposed in Section 5.6.

![Figure 5.21: Relationship between $f_{ur}$ and a proposed reduced ultimate strength $f_{u,red}$ for specimens tested by the SCI/Euro Inox (1996)](image)

**Figure 5.21:** Relationship between $f_{ur}$ and a proposed reduced ultimate strength $f_{u,red}$ for specimens tested by the SCI/Euro Inox (1996)

### 5.5.3 American and Australia/New Zealand Standards

The ASCE (2002) and AS/NZS (2001) Standards provide design rules for bolted connections composed of thin cold-formed stainless steel sheets but do not have rules for thicker material. These standards adopt the design provisions in the AISI Specification (1996) for cold-formed carbon steel bolted connections with a minor modification which is that the upper limit of the bearing capacity (given in Equations 5.11 and 5.12) is marginally reduced. The bearing resistance of cold-formed stainless
steel connections, which have been arranged in the format of Eurocode 3 for comparison purposes, is given by Equations 5.11 and 5.12.

for single shear connections:

\[ N_{b,\text{ASCE}} = \phi_{\text{end}} \left( \frac{e_i}{d} \right) t d f_u \leq \phi_{\text{bear}} (2.0 t d f_u) \]  \hspace{1cm} (5.11)

for double shear connections:

\[ N_{b,\text{ASCE}} = \phi_{\text{end}} \left( \frac{e_i}{d} \right) t d f_u \leq \phi_{\text{bear}} (2.75 t d f_u) \]  \hspace{1cm} (5.12)

where \( \phi_{\text{end}} \) and \( \phi_{\text{bear}} \) are the resistance factors against end tear-out and bearing failures of 0.7 and 0.65 respectively.

5.6 PROPOSED DESIGN RULES

In this section, the results obtained from the parametric studies are used to propose bearing design equations for hot-rolled and cold-formed stainless steel bolted connections. In order to exploit the high ductility and strain hardening characteristics of stainless steel, the concept in the AISC Specification (2005) for bearing design will be adopted. Two bearing design equations will be proposed according to the requirement (or not) for a deformation limit under service loads. The first bearing design equation is for bolted connections where the deformation under service loads is not a design consideration. This equation will be developed by adopting the strength criterion to define the ultimate bearing capacity, as controlled by fracture. The second equation is for connections where the deformation under service loads is a design consideration, and therefore the equation will be developed by considering the deformation criterion to define the service load and consequently the corresponding ultimate bearing capacity. It should be noted that two essential features were considered when suggesting the design equations. Firstly, the format of the equation is to be similar to that of Equation 5.6 and secondly, the ultimate tensile strength \( f_u \) is to be used instead of a combination between the yield strength \( f_y \) and ultimate strength \( f_u \) which is adopted in EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (2006).
5.6.1 Bearing capacity when deformation under service loads is not a design consideration

For this first scenario, the ultimate bearing capacity is taken as the load that corresponds to bearing fracture $F_{b,\text{frac}}$ as discussed in Section 5.4. The corresponding bearing coefficients $\beta_{b,\text{frac}}$ defined by Equation 5.13, obtained from the parametric studies are plotted against the edge distance ratio $e_1/d_0$ in Figures 5.22 to 5.25, 5.30 to 5.32 and 5.36 to 5.38 – $\sigma_{b,\text{frac}}$ is the nominal bearing stress at fracture. A suitable lower bound design equation for the bearing coefficient is then proposed in a format similar to Equation 5.6.

$$\beta_{b,\text{frac}} = \frac{F_{b,\text{frac}}}{tdf_u} = \frac{\sigma_{b,\text{frac}}}{f_u} \quad (5.13)$$

5.6.2 Bearing capacity when deformation under service loads is a design consideration

In this second scenario, the ultimate bearing capacity of the bolted connection is defined such that the deformation at the serviceability limit state is kept within an acceptable limit. Examining the load-deformation curves for all FE models, it was found that at 1.0 mm deformation the connections remain essentially elastic. Therefore, a service load corresponding to 1.0 mm deformation was adopted, and the corresponding ultimate bearing capacity $F_{b,\text{def}}$ (here controlled by deformation) was then obtained by assuming an average ratio of ultimate to service load of 1.45. This ratio has been obtained by adopting the partial safety factors in Eurocode 3 for dead and live loads coupled with the assumption that for steel structures the live loads are approximately double the dead loads (Ryan, 1999a). The bearing coefficients $\beta_{b,\text{def}}$ from Equation 5.14 obtained from the parametric studies are plotted against the edge distance ratio $e_1/d_0$ in Figures 5.26 to 5.29, 5.33 to 5.35 and 5.39 to 5.41 – $\sigma_{b,\text{def}}$ is the corresponding nominal bearing stress and a lower bound to this coefficient ($\beta_{b,\text{def}}$) is then proposed similar to the Equation 5.6 format.

$$\beta_{b,\text{def}} = \frac{F_{b,\text{def}}}{tdf_u} = \frac{\sigma_{b,\text{def}}}{f_u} \quad (5.14)$$
For this situation, the proposed design equations will ensure that the deformation at serviceability will be acceptable; there is, therefore, no need to conduct a separate serviceability check.

5.6.3 Connections composed of thick plates

5.6.3.1 Deformation under service loads is not a design consideration

Figures 5.22 to 5.25 illustrate the nominal bearing stress at fracture $\beta_{b,\text{frac}}$ for thick plate connections obtained from the parametric studies. It can be seen that the bearing stress factor is greater than $\alpha_1$ as defined by Equation 5.15.

$$
\alpha_1 = \begin{cases} 
2.5 \left( \frac{e_i}{3d_0} \right) \leq 2.5 & \text{for } e_i/d_0 > 1.5 \\
2.5 \left( \frac{e_i}{3d_0} \right) \leq 2.0 & \text{for } e_i/d_0 \leq 1.5 
\end{cases} 
$$

(5.15)

Thus, the proposed bearing design equation for connections where deformation is not a consideration is given by Equation 5.16.

$$
N_{b,\text{frac},\text{prop}} = \frac{\alpha_1 t d f_u}{\gamma_{M2}}
$$

(5.16)

5.6.3.2 Deformation under service loads is a design consideration

The factor $\alpha_2$ given by Equation 5.17 may be seen to provide a lower bound to the finite element data plotted in Figures 5.26 to 5.29. Thus, Equation 5.18 gives the ultimate bearing capacity when deformation is considered in design.

$$
\alpha_2 = 1.25 \left( \frac{e_i}{2d_0} \right) \leq 1.25
$$

(5.17)

$$
N_{b,\text{def},\text{prop}} = \frac{\alpha_2 t d f_u}{\gamma_{M2}}
$$

(5.18)
**Figure 5.22:** Bearing coefficient for thick plates from parametric studies by adopting strength criterion ($e_2/d_0 = 1.5$)

**Figure 5.23:** Bearing coefficient for thick plates from parametric studies by adopting strength criterion ($e_2/d_0 = 2.0$)
Figure 5.24: Bearing coefficient for thick plates from parametric studies by adopting strength criterion ($e_2/d_0 = 3.0$)

Figure 5.25: Bearing coefficient for thick plates from parametric studies by adopting strength criterion ($e_2/d_0 = 4.0$)
Figure 5.26: Bearing coefficient for thick plates from parametric studies by adopting deformation criterion ($e_2/d_0 = 1.5$)

Figure 5.27: Bearing coefficient for thick plates from parametric studies by adopting deformation criterion ($e_2/d_0 = 2.0$)
Figure 5.28: Bearing coefficient for thick plates from parametric studies by adopting deformation criterion ($e_2/d_0 = 3.0$)

Figure 5.29: Bearing coefficient for thick plates from parametric studies by adopting deformation criterion ($e_2/d_0 = 4.0$)
5.6.4 Inner sheets in double shear connections

5.6.4.1 Deformation under service loads is not a design consideration

It can be seen from Figures 5.30 to 5.32 that the bearing stress factor $\beta_{b,\text{frac}}$ (based on fracture) obtained from the FE models is greater than $\alpha_1$. Therefore, the ultimate capacity for this type of connection, when deformation is not a design consideration, may be given by Equation 5.16.

5.6.4.2 Deformation under service loads is a design consideration

The factor $\alpha_2$ defined by Equation 5.17 may be seen to provide a lower bound to the bearing stress $\beta_{b,\text{def}}$ (based on limiting deformation) plotted in Figures 5.33 to 5.35. The ultimate bearing capacity for this category is therefore given by Equation 5.18.

![Figure 5.30: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting strength criterion ($e_2/d_0 = 1.5$)](image_url)

Figure 5.30: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting strength criterion ($e_2/d_0 = 1.5$)
Figure 5.31: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting strength criterion ($e_2/d_0 = 2.0$)

Figure 5.32: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting strength criterion ($e_2/d_0 = 3.0$)
Figure 5.33: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting deformation criterion \( (e_2/d_0 = 1.5) \)

Figure 5.34: Bearing coefficient for thin sheets where curling does not occur from parametric studies by adopting deformation criterion \( (e_2/d_0 = 2.0) \)
5.6.5 Single shear connections and outer sheets in double shear connections

5.6.5.1 Deformation under service loads is not a design consideration

It can be seen in Figures 5.36 to 5.38 that the bearing stress factor at fracture $\beta_{b,\text{frac}}$ for this situation is greater than $\alpha_3$ as defined by Equation 5.19.

$$\alpha_3 = 1.6 \left( \frac{e_i}{2d_0} \right) \leq 1.6$$

(5.19)

Thus, the proposed bearing design equation is given by Equation 5.20.

$$N_{b,\text{frac,prop,c}} = \frac{\alpha_3 t d_f}{\gamma_{M2}}$$

(5.20)
5.6.5.2 Deformation under service loads is a design consideration

The factor $\alpha_2$ given by Equation 5.17 is proposed to provide a lower bound as shown in Figures 5.39 to 5.41 to the results of the parametric studies for thin plate connections where curling occurs. The ultimate bearing capacity is therefore given by Equation 5.18.

5.6.6 Comparison with design equation proposed by Kim et al. (2008)

Kim et al. (2008) proposed a design equation for the bearing failure of bolted connections between thin sheets where curling may occur. The analytical model in Section 5.4.5 was adopted and the shear strength of the plate material was assumed to be $f_u/\sqrt{3}$. In addition, the edge distance $e_1$ used in the determination of bearing capacity is limited to the smaller of $13t$ or $p_2$. The plate bearing capacity per bolt $N_{b,Kim}$ is given by the following relationship:

$$N_{b,Kim} = \frac{2e_{Kim}t f_u}{\sqrt{3}} \quad (5.21)$$

where $e_{Kim}$ is the minimum of $e_1$, $13t$ or $p_2$.

The results from the parametric studies in Section 5.4.3.2 are compared to the proposal by Kim et al. (2008) given in Equation 5.21. Figures 5.36 to 5.38 show that while Equation 5.21 underestimates the bearing resistance of 1.0 mm thick sheets, it predicts the capacity for connections between 2.0 mm thick sheets reasonably well.
Figure 5.36: Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting strength criterion \((e_2/d_0 = 1.5)\)

Figure 5.37: Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting strength criterion \((e_2/d_0 = 2.0)\)
**Figure 5.38:** Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting strength criterion \( \left( \frac{e_2}{d_0} = 3.0 \right) \)

**Figure 5.39:** Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting deformation criterion \( \left( \frac{e_2}{d_0} = 1.5 \right) \)
Figure 5.40: Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting deformation criterion \((e_2/d_0 = 2.0)\)

Figure 5.41: Bearing coefficient for thin sheets where curling does occur from parametric studies by adopting deformation criterion \((e_2/d_0 = 3.0)\)
5.6.7 Reliability analysis

Statistical analyses according to the procedure given in Annex D of EN 1990 (2002) have been carried out to determine a suitable partial safety factor $\gamma_{M2}$ for the proposed design equations. The analyses have been conducted following the steps presented in Chapter 4. The data sets obtained from the parametric studies were employed in these analyses. The strength functions for bearing resistance can be written in terms of the independent variables as follows:

Equation 5.16 becomes
$$r_i = g_n(X) = \alpha_1 t d f_u$$  \hspace{1cm} (5.22)

Equation 5.18 becomes
$$r_i = g_n(X) = \alpha_2 t d f_u$$  \hspace{1cm} (5.23)

Equation 5.20 becomes
$$r_i = g_n(X) = \alpha_3 t d f_u$$  \hspace{1cm} (5.24)

Comparisons between bearing resistances obtained from the FE models and Equations 5.22, 5.23 and 5.24 are presented in Figures 5.42, 5.43 and 5.44 respectively. A summary of the statistical analyses is listed in Table 5.1. The maximum required partial safety factor $\gamma_{M2}$, given in Table 5.1 is 1.066. Thus, a value of 1.1 is adopted for Equations 5.16, 5.18 and 5.20.

**Table 5.1:** Summary of statistical analysis results for bearing design equations

<table>
<thead>
<tr>
<th>Equation</th>
<th>Number of tests</th>
<th>$k_{d,n}$</th>
<th>$b$</th>
<th>$R^2$</th>
<th>$V_\delta$</th>
<th>$V_r$</th>
<th>$\gamma_M$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation 5.16</td>
<td>191</td>
<td>3.14</td>
<td>1.2216</td>
<td>0.962</td>
<td>0.1085</td>
<td>0.1225</td>
<td>0.995</td>
</tr>
<tr>
<td>Equation 5.18</td>
<td>251</td>
<td>3.13</td>
<td>1.2767</td>
<td>0.989</td>
<td>0.0674</td>
<td>0.0882</td>
<td>0.901</td>
</tr>
<tr>
<td>Equation 5.20</td>
<td>60</td>
<td>3.26</td>
<td>1.2206</td>
<td>0.929</td>
<td>0.1236</td>
<td>0.1360</td>
<td>1.066</td>
</tr>
</tbody>
</table>
Figure 5.42: *Comparison between numerical and theoretical resistances by adopting Equation 5.22*

Figure 5.43: *Comparison between numerical and theoretical resistances by adopting Equation 5.23*
5.7 CONCLUDING REMARKS

The bearing behaviour of stainless steel connections has been investigated herein by means of parametric studies using the previously validated FE models. The fundamental difference in the response of stainless steel and carbon steel connections is that, while the load-deformation curve for carbon steel connections flattens off after the initiation and spreading of yielding, for stainless steel connections this curve continues to rise significantly owing to strain hardening. For this reason, the limiting deformations used to define the bearing capacity of carbon steel connections were found to be unsuitable for stainless steel connections. Different failure definitions have therefore been devised for stainless steel connections, and bearing design equations for both thick and thin connections that cover two cases – one restricting and one ignoring serviceability deformations – have been proposed. These equations define the bearing capacity in terms of the material ultimate strength $f_u$ instead of the so-called reduced ultimate strength $f_{u,red}$, and therefore, are consistent with the provisions for carbon steel connections.
CHAPTER 6

GUSSET PLATE CONNECTIONS

6.1 INTRODUCTION

Angles are frequently employed as structural components acting in tension. Clear examples are roof trusses, transmission towers and bracing systems in multi-storey buildings. In these structures, either single or double angles can be used. In this chapter, net section rupture of stainless steel single angles connected by one leg to the gusset plate with a single row of bolts is investigated. The behaviour of previously tested specimens is initially replicated by employing FE models. These models are then used to perform parametric studies. Net section rupture design provisions are proposed based on the results of the parametric studies.

6.2 SHEAR LAG

For practicality, angles are usually connected to other structural members by means of intermediate structural elements, known as gusset plates, by bolting one leg only to the gusset plate. As a result, the distribution of stresses over the critical net section is nonuniform – the unconnected part being less stressed. This phenomenon is called shear lag. The effect of shear lag is that the efficiency of the net section in resisting tensile force is less than should the entire cross-section is connected. The net section efficiency
U (also referred to as the shear lag coefficient) is defined as the ratio of nominal (mean) stress over the net section to the material ultimate tensile strength as follows:

\[
U = \frac{\frac{F_{\text{net}}}{A_{\text{net}}}}{f_u} = \frac{F_{\text{net}}}{A_{\text{net}} f_u}
\]  

(6.1)

where \( F_{\text{net}} \) is the actual ultimate tensile carrying capacity of the section, \( A_{\text{net}} \) is the net section area at the critical section and \( f_u \) is the material ultimate tensile strength. Figure 6.1 illustrates a typical gusset plate connection and the stresses acting over the net section.

**Figure 6.1: Gusset plate connection**
6.3 CURRENT DESIGN PROVISIONS

Most design codes employ a simplified approach for the determination of the net section capacity of angles. The angle is considered as an equivalent concentrically loaded member, i.e. the connection eccentricity, shear lag and moment effects are ignored; instead, the net section area is reduced to an effective net section area by introducing reduction factors.

6.3.1 Carbon steel members

EN 1993-1-8 (2005) provides design rules for carbon steel single angles in tension connected by a single row in the direction of loading. The net section capacity of angles connected by a single row of bolts is given by the following equations:

\[
N_{u,EN3} = \frac{2(e_2 - 0.5d_n) \gamma f_u}{\gamma M_2} \quad (6.2)
\]

\[
N_{u,EN3} = \frac{\beta A_{net} \gamma f_u}{\gamma M_2} \quad (6.3)
\]

where \( \gamma M_2 \) is a partial safety factor with a value of 1.25, \( \beta \) is a reduction factor dependent on the pitch \( p_1 \) and number of bolts, given in Table 6.1, and \( A_{net} \) is the net area of the angle in the case of equal angles and unequal angles connected by their longer leg. For an unequal-leg angle connected by its smaller leg, \( A_{net} \) should be taken equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg. For intermediate values of \( p_1 \), EN 1993-1-8 (2005) allows \( \beta \) to be determined by linear interpolation.

<table>
<thead>
<tr>
<th></th>
<th>( \beta ) for ( p_1 \leq 2.5 \ d_0 )</th>
<th>( \beta ) for ( p_1 \geq 5.0 \ d_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two bolts</td>
<td>0.4</td>
<td>0.7</td>
</tr>
<tr>
<td>Three or more bolts</td>
<td>0.5</td>
<td>0.7</td>
</tr>
</tbody>
</table>
The ultimate net section resistance $N_{u,AISC}$ of a single angle connected to a gusset plate by one leg and subjected to pure tensile load is given in the AISC (2005) Specification by the following equation:

$$N_{u,AISC} = \phi_t U A_{net} f_u$$

where $\phi_t$ is the resistance factor of 0.75 and $U$ is the shear lag factor given in Table D3.1 of the AISC 360-05 (2005) and the values for bolted angles are reproduced herein in Table 6.2, in which $x$ is the connection eccentricity and $L$ is the connection length measured between the extreme bolts as shown in Figure 6.1. AISC (2005) recommends that the connection should be proportioned such that the minimum values given in Table 6.2 are satisfied; alternatively, a lower value of $U$ is permitted if the effects of moments arising from eccentricity of the applied tension load are taken into consideration by...
designing these members for combined tension and moments. It should be noted that AISC (2005) recommends using a minimum of two bolts.

**Table 6.2: Shear lag factor U for single angles**

<table>
<thead>
<tr>
<th></th>
<th>U</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two or three bolts</td>
<td>1- (x/L) ≥ 0.6</td>
</tr>
<tr>
<td>Four bolts or more</td>
<td>1- (x/L) ≥ 0.8</td>
</tr>
</tbody>
</table>

### 6.3.2 Stainless steel members

ENV 1993-1-4 (1996) provides design rules for net section capacity of stainless steel angles connected by one leg and subjected to tensile force as given in Equations 6.5 and 6.6. The non-uniformity of stress distribution over the critical cross section due to the eccentricity is accounted for in this Standard by assuming that the load is applied concentrically and that only half of the cross-sectional area of the unconnected leg contributes to resist the load.

\[
N_{u,ENV3} = \frac{0.9 A_{red} f_u}{\gamma_M^2} \tag{6.5}
\]

\[
A_{red} = A_{n,con} + 0.5 A_{g,unc} \tag{6.6}
\]

where \(A_{n,con}\) is the net cross-sectional area of the connected leg at the critical section, \(A_{g,unc}\) is the gross cross-sectional area of the unconnected leg but taken as not more than twice the gross cross-sectional area of the connected leg and \(\gamma_M^2\) is a partial safety factor with a value of 1.25.

The SCI/Euro Inox Design Manual (1996) adopted Equation 6.5 for net section design capacity, but with a slight difference – the reduced net section area \(A_{red}\) is given by:

\[
A_{red} = A_{n,con} + 0.5 A_{g,smaller} \tag{6.7}
\]
CHAPTER 6 – GUSSET PLATE CONNECTIONS

where \( A_{g,smaller} \) is the gross area of the smaller leg.


6.4 NUMERICAL MODELLING IN ABAQUS USING EXPLICIT SOLVER

Extreme discontinuous problems that contain a large number of contact interfaces, in addition to material and geometrical nonlinearities, are very difficult to be simulated using static finite element formulations. The FE models of these types of problem tend to terminate before a complete solution is found after performing an excessive number of iterations. On the other hand, the dynamic solution, with its simplicity, is an efficient method to model such cases.

ABAQUS (2006) provides two solvers to simulate nonlinear problems: ABAQUS/Standard and ABAQUS/Explicit. ABAQUS/Standard has been discussed in Chapter 3. ABAQUS/Explicit was developed essentially to simulate dynamic response. It uses a central difference rule to integrate the equations of motion explicitly through time. The kinematic conditions at one increment are used to determine the kinematic conditions at the next increment. At the beginning of the increment, the program solves the following dynamic equilibrium:

\[
[M][a] = [P] - [I]
\]

(6.8)

where \( M \) is the nodal mass matrix, \( a \) is the nodal acceleration matrix, \( P \) is the nodal external applied forces matrix and \( I \) is the nodal internal forces matrix. The accelerations are integrated through time using the central difference rule to calculate the change in velocity assuming that the acceleration is constant. This change in velocity is added to the velocity from the middle of the previous increment to determine the velocities at the middle of the current increment. The velocities are integrated through time and added to the displacements at the beginning of the increment to determine the displacements at the end of the increment. For the method to produce accurate results, the time
increments must be very small so that the accelerations are almost constant during an increment. Therefore, analysis using this solver requires a large number of increments. However, each increment is computationally inexpensive because there are no simultaneous equations to be solved. The foremost feature of the explicit solver is the absence of a global tangent stiffness matrix, which is required with the implicit (ABAQUS/Standard) method. Moreover, iterations and tolerances are not required because the state of the model is advanced explicitly.

6.4.1 Contact definition in ABAQUS/Explicit

Contact interfaces in ABAQUS/Explicit can be modelled using two different algorithms: the general contact algorithm and the contact pair algorithm. For general contact, ABAQUS/Explicit enforces contact constraints using a penalty contact formulation in which forces that are a function of the penetration distance are applied to the slave nodes to oppose the penetration, while equal and opposite forces act on the master surface at the penetration point.

In the contact pair algorithm, in each increment of the analysis, ABAQUS/Explicit first advances the kinematic state of the model into a predicted configuration without considering the contact conditions. ABAQUS/Explicit then determines which slave nodes in the predicted configuration penetrate the master surfaces. The depth of penetration of each slave node, the mass associated with it, and the time increment are used to calculate the resisting force required to oppose penetration. Then the resisting forces of all the slave nodes are distributed to the nodes on the master surface. The mass of each contacting slave node is also distributed to the master surface nodes and added to their mass to determine the total inertial mass of the contacting interfaces. ABAQUS/Explicit uses these distributed forces and masses to calculate an acceleration correction for the master surface nodes. Acceleration corrections for the slave nodes are then determined using the predicted penetration for each node, the time increment, and the acceleration corrections for the master surface nodes. ABAQUS/Explicit uses these acceleration corrections to obtain a corrected configuration in which the contact constraints are enforced.
One advantage of the explicit procedure over the implicit procedure is the greater ease with which it resolves complicated contact problems, especially for very large models. Extremely discontinuous contact conditions are readily formulated in the explicit method and can be enforced on a node by node basis without iteration which is used in ABAQUS/Standard to model contact interfaces as discussed in Chapter 3.

### 6.4.2 Modelling static problems using ABAQUS/Explicit

The explicit method in ABAQUS was originally developed for solving problems involving high-speed dynamic responses. If the duration of the event is short, the solution can be obtained efficiently. Nonetheless, the advantages of the explicit solver over the implicit one can also be utilized to analyse complicated static models where difficulty in convergence arises because of contact or material complexities resulting in a large number of iterations and, occasionally, premature analysis termination. These types of problems can be readily analysed using Explicit.

Applying the explicit dynamic procedure to quasi-static problems requires some special considerations. Since a static response is a long-term solution, analysing the static simulation in its real (physical) time is impractical, as it requires a very large number of small increments. On the other hand, if the loading rate is increased too much then the inertial effects will become dominant. Therefore, to model static problems efficiently using ABAQUS/Explicit, the load should be applied in the shortest possible period in which inertial forces remains insignificant. In selecting loading rates for quasi-static analyses, ABAQUS recommends that step time be at least 10 times slower than that corresponding to the fundamental frequency. In addition, it is recommended that the application of loading should be as smooth as possible to ensure small changes in acceleration from one increment to the next, such that the changes in both velocity and displacement are also smooth. ABAQUS has a simple built-in smooth step amplitude curve that creates a smooth loading amplitude as shown in Figure 6.3. When a smooth amplitude curve is defined, the first and the second data pairs are connected by a curve such that its first and second derivatives are smooth. The values of these derivatives are zero at each of these points and therefore the motion will be smooth.
As discussed previously, in quasi-static simulations, the inertial forces are negligible because the velocity of the material in the model is very small, consequently, the kinetic energy is also small and most of the external work done by the applied forces is converted into strain energy. To ensure that the response is quasi-static, the kinetic energy should not exceed a small fraction of the total energy. For static models, ABAQUS recommends that the ratio between the kinetic energy to internal energy should be less than 10%.

The ABAQUS/Explicit solver has been proved previously to be capable of modelling static problems. ABAQUS (2007) compared the results obtained from explicit dynamic and static solvers. The deformed shapes and stress contours showed that the results were similar. Yu et al. (2008) explained that explicit dynamic analysis is efficient when modelling bolted steel connections. Yu et al. (2008) used this solver to study the behaviour of T-stub connections under static loading. Good agreement between the numerical and test results was found.

6.4.3 Verification of results obtained from ABAQUS/Explicit

In order to demonstrate the capability of ABAQUS/Explicit in modelling static problems, two specimens from Chapter 3 (No. 2 and 16 in Table 3.3), which were replicated using ABAQUS/Standard, have been modelled by employing
ABAQUS/Explicit. All the measures recommended by ABAQUS (2007) to model static problems using the explicit solver were taken. The smooth amplitude shown in Figure 6.3(b) was incorporated to apply the displacement at the end of the central plates. The kinetic energy was less than 5% of the internal energy throughout the analyses and the response is therefore considered to be essentially static. Furthermore, the load-deformation curves and the strain distributions obtained from ABAQUS/Explicit and ABAQUS/Standard for these specimens are compared in Figures 6.4 and 6.5. It can be concluded that ABAQUS/Explicit is able to simulate static problems successfully provided that a certain procedure is followed.

Due to the complexity of the stainless steel gusset plate connections considered in the current chapter – arising mainly from the large number of contact interfaces – the static analysis was found to be extremely difficult, but Figures 6.4 and 6.5 show that Standard and Explicit results to be very similar. Thus, the distinct advantages of ABAQUS/Explicit in solving such models were exploited as presented in the following sections,
Figure 6.4: Comparison between ABAQUS/Explicit and ABAQUS/Standard results for specimen No. 2 (see Table 3.3 and Figure 3.4)
**Figure 6.5:** Comparison between ABAQUS/Explicit and ABAQUS/Standard results for specimen No. 16 (see Table 3.3 and Figure 3.4)

(a) Load-deformation curve

(b) Strain distribution in the plate in front of the bolt
6.5 NUMERICAL MODELLING OF GUSSET PLATE CONNECTIONS

6.5.1 General

The test results for stainless steel gusset plate connections reported by Ryan (1999b) have been simulated numerically using ABAQUS/Explicit. The specimens consisted of a 1.5 m long member of either angle or tee cross-section connected to gusset plates at both ends with bolts. Some of the tested members were cut from I, channel or rectangular hollow section members. A static tensile load was applied to the gusset plates. A total of 12 specimens were tested by Ryan (1999b). However, during the preparation of test specimens, one of the specimens was partially damaged, and its results were deemed to be unreliable (Ryan, 1999b). In this section, therefore only 11 specimens were numerically simulated. The configurations of these connections are displayed in Figures 6.6 to 6.16. The members were austenitic stainless steel grade 1.4306 or grade 1.4307, while the gusset plates for all specimens were austenitic stainless steel grade 1.4306. Table 6.3 summarises the mechanical properties of the materials.
### Table 6.3: The material properties of the elements in gusset plate connections tested by Ryan (1999b)

<table>
<thead>
<tr>
<th>Steel</th>
<th>Element (mm)</th>
<th>fy (σ0.2) (N/mm²)</th>
<th>fu (N/mm²)</th>
<th>Elongation at fracture (%)</th>
<th>fu/fy</th>
<th>ns</th>
<th>mₙ</th>
<th>K (N/mm²)</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic grade 1.4306</td>
<td>Angle 100x100</td>
<td>321</td>
<td>612</td>
<td>57.0</td>
<td>1.91</td>
<td>6.40</td>
<td>2.84</td>
<td>1596</td>
<td>0.60</td>
</tr>
<tr>
<td>Austenitic grade 1.4307</td>
<td>Angle 80x65</td>
<td>404</td>
<td>628</td>
<td>47.6</td>
<td>1.55</td>
<td>7.89</td>
<td>3.25</td>
<td>1434</td>
<td>0.44</td>
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<td>Angle 110x50</td>
<td>369</td>
<td>658</td>
<td>53.6</td>
<td>1.78</td>
<td>5.93</td>
<td>2.96</td>
<td>1632</td>
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<td>3.63</td>
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<td>Gusset plates</td>
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<td>581</td>
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<td>6.58</td>
<td>2.73</td>
<td>1548</td>
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<td>M12 bolt</td>
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<td>863</td>
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<td>3.73</td>
<td>3.81</td>
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<td>0.24</td>
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<tr>
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<td>M16 bolt</td>
<td>748</td>
<td>955</td>
<td>-</td>
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<td>3.74</td>
<td>1767</td>
<td>0.25</td>
</tr>
<tr>
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<td>M20 bolt</td>
<td>686</td>
<td>852</td>
<td>-</td>
<td>1.24</td>
<td>4.03</td>
<td>3.82</td>
<td>1530</td>
<td>0.23</td>
</tr>
</tbody>
</table>
Figure 6.6: Configuration of Specimen No. 1

All dimensions are in mm
CHAPTER 6 – GUSSET PLATE CONNECTIONS

Figure 6.7: Configuration of Specimen No. 2

(a) Side view

(b) Plan view

(c) Section X-X

All dimensions are in mm
CHAPTER 6 – GUSSET PLATE CONNECTIONS

Figure 6.8: Configuration of Specimen No. 3

Deformation was measured between these points

All dimensions are in mm
Figure 6.9: Configuration of Specimen No. 4
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Figure 6.10: Configuration of Specimen No. 5
**Figure 6.11:** Configuration of Specimen No. 6

All dimensions are in mm.
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Figure 6.12: Configuration of Specimen No. 7
Figure 6.13: Configuration of Specimen No. 8

Deformation was measured between these points

4 bolts 16 mm diameter in 18 mm bolt holes

All dimensions are in mm
Figure 6.14: Configuration of Specimen No. 9

All dimensions are in mm
Figure 6.15: Configuration of Specimen No. 10
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Figure 6.16: Configuration of Specimen No. 11

Deformation was measured between these points

8 bolts 12 mm diameter in 14 mm bolt holes

All dimensions are in mm
6.5.2 Boundary conditions and loading

In order to maximize the computational efficiency, the symmetries of each specimen were utilized to model half the specimen for connections containing angles and a quarter of the specimen for those containing tee sections. This was achieved by applying appropriate boundary conditions at symmetry planes. The displacement-control method was adopted to apply the loading. A uniform translational displacement was applied at the end of the gusset plate with the smooth amplitude shown in Figure 6.3(b). The minimum period required to simulate the static response of the specimens was determined as 10 times the time corresponding to the fundamental frequency of the member as recommended by ABAQUS (2007). Figures 6.17 to 6.19 show the boundary conditions and the applied loading.

6.5.3 Material model and element type

The nonlinear behaviour of stainless steel materials was described by employing the material model given in Chapter 3 by Equations 3.1 and 3.8. The material behaviour was represented by a multi-linear stress-strain curve in terms of true stresses and true plastic strains which were obtained using Equations 3.6 and 3.7 in Chapter 3. The material parameters in these equations are given in Table 6.3. Plasticity was considered by incorporating the von Mises yield criterion with an isotropic hardening rule.

The three-dimensional solid element with full integration – C3D8 in ABAQUS (2007) – was employed to model the gusset plate, the member and the bolts. The optimum size of the FE model mesh was determined through a convergence study.

6.5.4 Failure criteria for FE models

The strain-based criteria adopted in Chapter 3 were employed in the current chapter to define the net section and bolt shear failures for the FE models. Net section rupture of the connected member in the FE models was assumed to occur when the maximum equivalent plastic strain at the edge of the bolt hole which is closest to the applied load
(the reference point shown in Figure 6.9) reached the true localized fracture strain of 100%.

Bolt shear failure was said to have taken place when the critical cross-section of the bolt became fully plastic i.e. when the equivalent plastic strain over the full critical cross-section reached the true plastic strain corresponding to $\sigma_{0.2}$.

Figure 6.17: Boundary conditions applied to specimens with angles (specimen No.4)
Figure 6.18: Boundary conditions applied to specimens with angles reinforced with lugs (specimen No. 5)
6.5.5 Validation of FE models

The FE models were validated against the experimental results of Ryan (1999b). Typical load-deformation curves obtained from the tests and the numerical analyses are displayed in Figures 6.20 to 6.23, in which the overall connection deformation was measured as the extension between two points, one on the gusset plate and the other on the member. The locations of these reference points are shown in Figures 6.6 to 6.16. It is clear that the predicted load-deformation curves are in good agreement with the tests. In addition, the ultimate capacities obtained from the FE models using the proposed failure criteria were compared with those obtained in the tests. The comparison in Table 6.4 shows that the developed models were able to predict the observed ultimate capacities accurately (with the exception of model No. 1) with a mean FE/test ratio of 1.03 and a standard deviation of 0.05. For model No. 1, although the load-deformation
curve from the FE model closely follows that obtained from the test as shown in Figure 6.20, the predicted ultimate capacity is overestimated. To identify the reason of the discrepancy in the failure loads for this specimen, the test data were scrutinised more closely. It was observed that while the images of all the specimens at failure showed clearly fractured connections (either the member or bolts are ruptured), as can be seen in Figures 6.24(a) and 6.24(b), the image of this particular specimen (No. 1) does not show any sign of fracture as shown in Figure 6.24(c). Hence, it is believed that the net section rupture for this specimen may have been imminent but did not occur, the load was removed before the actual net section rupture took place.

A comparison between the deformed specimen for a typical test and the corresponding FE model are shown in Figure 6.25. It can be concluded that the developed FE models are able to replicate the behaviour observed during laboratory testing and can, therefore, be employed to generate further results to investigate the parameters that may affect the net section capacity of these forms of connection.
Figure 6.20: Load-deformation curve for specimen No. 1

Figure 6.21: Load-deformation curve for specimen No. 5
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Figure 6.22: Load-deformation curve for specimen No. 6

Figure 6.23: Load-deformation curve for specimen No. 9
Table 6.4: *Comparison between ultimate capacities obtained from test and FE models for gusset plate connections*

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Type of connection</th>
<th>Test</th>
<th>FE</th>
<th>FE/Test ratio</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Ultimate load (kN)</td>
<td>Failure mode</td>
<td>Ultimate load (kN)</td>
</tr>
<tr>
<td>1</td>
<td>Angle 100x100</td>
<td>642.0</td>
<td>N</td>
<td>817.4</td>
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<td>2</td>
<td>Angle 100x100</td>
<td>322.0</td>
<td>SH</td>
<td>339.9</td>
</tr>
<tr>
<td>3</td>
<td>Angle 100x100</td>
<td>556.7</td>
<td>SH</td>
<td>562.3</td>
</tr>
<tr>
<td>4</td>
<td>Angle 80x65</td>
<td>441.8</td>
<td>N</td>
<td>500.9</td>
</tr>
<tr>
<td>5</td>
<td>Angle 80x65</td>
<td>339.5</td>
<td>SH</td>
<td>339.0</td>
</tr>
<tr>
<td>6</td>
<td>Angle 80x65</td>
<td>514.9</td>
<td>N</td>
<td>516.6</td>
</tr>
<tr>
<td>7</td>
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<td>275.0</td>
<td>SH</td>
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</tr>
<tr>
<td>8</td>
<td>Angle 110x50</td>
<td>308.6</td>
<td>N</td>
<td>335.5</td>
</tr>
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<td>9</td>
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<td>274.9</td>
<td>SH</td>
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</tr>
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</tr>
<tr>
<td>11</td>
<td>Tee 80x82</td>
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<tr>
<td>S.D.</td>
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<td></td>
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</tr>
</tbody>
</table>

SH = Bolt shear failure - N = Net section failure

* No full rupture occurred during testing (has been excluded from the statistical evaluation)
Figure 6.24: Typical failures during testing
Figure 6.25: Deformed shape of specimen No. 6

(a) Plan view of the whole specimen

(b) Side view of the connection zone
6.6 PARAMETRIC STUDIES

6.6.1 General

In order to understand the behaviour of net section rupture of bolted gusset plate connections, parametric studies were conducted by employing the validated FE models. The key parameters, which were thought to exert most influence on the net section capacity of these connections, were included. These are the number of bolts, the spacing between the bolts in the direction of loading (the length of the connection) and the eccentricity of the connection.

Two cases were considered in the parametric studies. The first case addresses equal angles connected by one leg and the second case covers unequal angles connected by the smaller leg. Since there are no standard angles in stainless steel, the geometrical properties for standard carbon steel angles from the Steel Building Design: Design Data Guide (SCI/BCSA, 2009) were adopted for equal-leg angles; these are displayed in Table 6.5. For some unequal-leg angles, the geometry of the standard carbon steel angles was slightly adjusted so as to compare their behaviour to that of similar equal-leg angles. Table 6.6 lists the investigated unequal-leg angles. For both cases four configurations were investigated: two bolts, three bolts and four bolts in one row, in addition to single bolt connections. A wide range of spacing ratios \( p_1/d_0 \) was covered: 2.5, 3.75, 5.0 and 6.25. These values were adopted in order to investigate the key values of spacing ratio \( p_1/d_0 \) in the Eurocode 3 design equations (Table 6.1), these are 2.5 and 5.0. The configuration of these models is presented in Figure 6.26. The material properties of the angles and gusset plates employed in the parametric studies is austenitic grade 1.4307 (taken from the angle section 80x65 in Table 6.3), while bolts were considered as elastic because bolt failure was not intended to feature in the study.

The length of the connected angle has no influence on the net section capacity as concluded by Young (1935) and Wu and Kulak (1993). Therefore, the length of the angles in the parameter studies is kept constant of 2.0 m. The gusset plate dimensions were designed so as the failure occurs at the net section of the angle.
### Table 6.5: Equal-leg angles employed in the parametric studies

<table>
<thead>
<tr>
<th>Angle AxAxt (mm)</th>
<th>Leg size A (mm)</th>
<th>Leg thickness t (mm)</th>
<th>Connection eccentricity x (mm)</th>
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<tr>
<td>50x50x5</td>
<td>50</td>
<td>5</td>
<td>14.3</td>
</tr>
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<td>70x70x6</td>
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</tr>
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<td>90x90x8</td>
<td>90</td>
<td>8</td>
<td>25.5</td>
</tr>
<tr>
<td>100x100x10</td>
<td>100</td>
<td>10</td>
<td>28.7</td>
</tr>
</tbody>
</table>

### Table 6.6: Unequal-leg angles (connected by the smaller leg) employed in the parametric studies

<table>
<thead>
<tr>
<th>Angle AxBxt (mm)</th>
<th>Smaller leg size A (mm)</th>
<th>Larger leg size B (mm)</th>
<th>Leg thickness t (mm)</th>
<th>Connection eccentricity x (mm)</th>
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<td>5</td>
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<td>100</td>
<td>6</td>
<td>31.7</td>
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<td>70x125x6</td>
<td>70</td>
<td>125</td>
<td>6</td>
<td>42.4</td>
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</tbody>
</table>
Figure 6.26: Configuration of FE models used in the parametric studies for gusset plate connections
6.6.2 Results of the parametric studies

The net section efficiency of angles obtained from the parametric studies has been compared to that which results from stainless steel and carbon steel design standards. The net section efficiency of the angles $U_{FE}$ from the FE models was determined using Equation 6.9.

$$U_{FE} = \frac{F_{\text{net,FE}}}{A_{\text{net}} f_u}$$

(6.9)

where $F_{\text{net,FE}}$ is the net section capacity from the FE models obtained by employing the failure criterion discussed in Section 6.5.4, and $A_{\text{net}}$ is the net cross-section area of the angle at the critical section.
In order to determine the net section efficiency from ENV 1993-1-4 (1996) and the SCI/Euro Inox (1996) for equal-leg and unequal-leg angles, Equation 6.5 has been rewritten as follows:

\[
N_{u,ENV3} = \left(0.9 \frac{A_{\text{red}}}{A_{\text{net}}} \right) A_{\text{net}} f_u
\]  

(6.10)

Hence, the net section efficiency for both ENV 1993-1-4 (1996) and the SCI/Euro Inox (1996) is given by:

\[
U_{ENV3} = 0.9 \frac{A_{\text{red}}}{A_{\text{net}}}
\]  

(6.11)

Note that the definitions of \(A_{\text{red}}\) for unequal-leg angles in ENV 1993-1-4 (1996) and the SCI/Euro Inox Design Manual (1996) are different (see Section 6.3.2).

The net section efficiency for an equal-leg angle in EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (2006) is the \(\beta\) factor in Table 6.1. For an unequal-leg angle connected with its smaller leg, net section efficiency can be obtained by rewriting Equation 6.3 as follows:

\[
N_{u,EN3} = \left(\beta \frac{A_{\text{net,uneq}}}{A_{\text{net}}} \right) A_{\text{net}} f_u
\]  

(6.12)

where \(A_{\text{net,uneq}}\) is a reduced net section area equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg. Hence, the net section efficiency is given by:

\[
U_{EN3} = \beta \frac{A_{\text{net,uneq}}}{A_{\text{net}}}
\]  

(6.13)

Tables 6.7 to 6.9 present the results from the FE models and different design standards for equal-leg angles, while those for unequal-leg angles are presented in Tables 6.10 to
6.12. It can be seen that the net section efficiency predicted by EN 1993-1-4 (2006) and SCI/Euro Inox (2006) are overly conservative. The design provisions in AISC (2005) for carbon steel connections generally predict the net section capacity of stainless steel equal-leg angles reasonably well, though, for unequal-leg angles, these provisions tend to overestimate the net section resistance.
### Table 6.7: Comparison between the parametric study results and design standards (equal-leg angles with two bolts)

<table>
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<td>0.59</td>
<td>0.70</td>
<td>0.91</td>
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<td>0.70</td>
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<td>0.59</td>
<td>0.70</td>
<td>0.89</td>
</tr>
<tr>
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<td>0.62</td>
<td>0.70</td>
<td>0.85</td>
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<td>0.55</td>
<td>0.85</td>
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<td>65.0</td>
<td>0.61</td>
<td>0.63</td>
<td>0.40</td>
<td>0.61</td>
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</table>
**Table 6.8:** Comparison between the parametric study results and design standards (equal-leg angles with three bolts)

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<th>Angle dimensions (A×A×t) (mm)</th>
<th>x (mm)</th>
<th>p₁/d₀</th>
<th>L (mm)</th>
<th>Net section efficiency U</th>
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<td>6.25</td>
<td>325</td>
<td>0.96</td>
</tr>
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<td>6.25</td>
<td>325</td>
<td>0.93</td>
</tr>
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<td>100x100x10</td>
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<td>6.25</td>
<td>325</td>
<td>0.92</td>
</tr>
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<td>0.95</td>
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<td>260</td>
<td>0.94</td>
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<td>5.0</td>
<td>260</td>
<td>0.91</td>
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<td>3.75</td>
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<td>130</td>
<td>0.73</td>
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</table>
Table 6.9: Comparison between the parametric study results and design standards (equal-leg angles with four bolts)

<table>
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<th>Angle dimensions (AxAxt) (mm)</th>
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<th>L (mm)</th>
<th>Net section efficiency U</th>
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<tr>
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<td>6.25</td>
<td>487.5</td>
<td>0.96</td>
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Table 6.10: Comparison between the parametric study results and design standards (unequal-leg angles with two bolts)

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### Table 6.11: Comparison between the parametric study results and design standards (unequal-leg angles with three bolts)

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Table 6.12: Comparison between the parametric study results and design standards (unequal-leg angles with four bolts)

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Figure 6.27 shows a deformed connected angle from the parametric studies. It can be seen that necking occurs at the critical net section, where strains are concentrated, as reported by Barth et al. (2002) and Topkaya (2004).

![Typical Deformed shape from the parametric studies](image)

**Figure 6.27: Typical Deformed shape from the parametric studies**

To increase the data set for single angles connected with a single bolt, six angles in addition to those adopted in Tables 6.5 and 6.6 have been included. The results from the parametric studies for connections with a single bolt are presented in Table 6.13.
Table 6.13: Comparison between the parametric study results and design standards
(equal and unequal-leg angles with a single bols)

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6.6.2.1 Effect of connection eccentricity

The influence of connection eccentricity on the net section capacity of tension members has been investigated in many studies. For instance, Chesson and Munse (1963) carried out a comprehensive study on the net section rupture of a large number of truss-type members. The connection eccentricity was considered as one of the geometrical parameters, amongst others, affecting the net section capacity of these structural elements. It was concluded that the net section resistance of tension members decreases with an increase in the connection eccentricity. Bartels (2000) showed that, as the connection eccentricity increased, there was a near linear decrease in the net section capacity.

Both studies demonstrated that connection eccentricity has a significant influence on the ultimate resistance of the net section. However, different explanations were presented for how this parameter affects the net section fracture response. Chesson and Munse (1963) studied the distribution of stresses over the critical net section in I-section members connected through their flanges, as shown in Figure 6.28. Despite the absence of moments arising from eccentricity in this case due to the symmetry in both cross-section and loading, it was observed that the stresses over the web were markedly less than those over the flanges, especially near the centre line of the cross-section (see Figure 6.28). This was attributed to the mechanism by which loading is transferred from the connected part of the member to the unconnected part. The loading is transferred from one layer to another by means of shear stresses acting over these layers. As the distance between a specific unconnected material layer increases, the ability of shear stresses to transmit the force to this layer decreases. As a result, this layer becomes less stressed than a layer closer to the connected material. Chesson and Munse (1963) referred to this phenomenon as ‘shear lag’. To reflect the shear lag phenomenon, which is dependent on the distance between the unconnected material and the connected material, Chesson and Munse (1963) adopted connection eccentricity x as a measure of the ratio of the connected material to the unconnected material. Another explanation was made by Bartels (2000). He concluded that due to the eccentricity of the applied load, moments are induced over the critical net section, resulting in flexural stresses. When these stresses are compounded with the direct tensile stresses arising from the force, an
increase in the resultant tensile stresses in part of the critical section and a decrease in
the resultant stresses in other parts takes place. Consequently, the part subjected to the
compressive flexural stresses becomes less effective in resisting the applied tension
force.

In the current study, the effects of the connection eccentricity on the net section
efficiency were investigated. The results from the parametric studies presented in Tables
6.7 to 6.13 illustrate that the efficiency of the net section in resisting the applied load
decreases with an increase of connection eccentricity. The way that the eccentricity
affects the net section capacity can be attributed to both phenomena explained by
Chesson and Munse (1963) and Bartels (2000).

In current design standards, while AISC (2005) considers this parameter for net section
efficiency determination without distinction between equal-leg and unequal-leg angles,
the provisions in EN 1993-1-4 (2006) and SCI/Euro Inox (2006) do not consider the
connection eccentricity explicitly. In the case of equal-leg angles with different leg sizes
(different connection eccentricities) the same reduction factor $\beta$ is applied. However,
these standards allow for a further net section reduction in the case of unequal-leg
angles connected through the smaller leg. A comparison between the design provisions
given in EN 1993-1-4 (2006) and SCI/Euro Inox (2006) with the FE model results for

![Diagram of connection eccentricity](image.png)

**Figure 6.28**: Definition of connection eccentricity $x$ for symmetrically loaded
members by Chesson and Munse (1963)
equal-leg angles with different leg sizes is given in Figures 6.29 to 6.31. These figures demonstrate that, while the net section efficiencies obtained from EN 1993-1-4 (2006) for two equal-leg angles with different connection eccentricities (different leg sizes) are the same, the results from the parametric studies indicate that there is a significant difference in the net section efficiencies for connections with different eccentricities, especially, when connection length is relatively small. As connection length increases (either by increasing the number of bolts or by increasing $p_1/d_0$ ratio) the effect of connection eccentricity diminishes. This is also explained in Figures 6.32 to 6.37, which show the net section efficiency for equal-leg and unequal-leg angles obtained from the parametric studies with different connection eccentricities but similar connection lengths. It can be concluded that connection eccentricity has a significant effect on the net section efficiency particularly for connections with small lengths.

The strain distributions over the unconnected leg for connections with the same lengths but different eccentricities have also been compared. Figure 6.38 compares between equal-leg angles and Figure 6.39 compares between unequal-leg angles connected by the smaller leg. These show that as the connection eccentricity increases, the strain over the unconnected leg decreases approximately linearly and, consequently, the contribution of the unconnected leg in resisting the applied load decreases. Similar strain distributions were also observed from the experimental investigation conducted by Wu and Kulak (1993).
**Figure 6.29:** Net section efficiency from the parametric studies for equal angles with two bolts

**Figure 6.30:** Net section efficiency from the parametric studies for equal angles with three bolts
Chapter 6 – Gusset Plate Connections

Figure 6.31: Net section efficiency from the parametric studies for equal angles with four bolts

Figure 6.32: Net section efficiency for equal-leg angles with two bolts
Figure 6.33: Net section efficiency for equal-leg angles with three bolts

Figure 6.34: Net section efficiency for equal-leg angles with four bolts
Figure 6.35: Net section efficiency for unequal-leg angles connected with smaller leg with two bolts (70x70x6, 70x100x6 and 70x125x6)

Figure 6.36: Net section efficiency for unequal-leg angles connected with smaller leg with three bolts (70x70x6, 70x100x6 and 70x125x6)
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Figure 6.37: Net section efficiency for unequal-leg angles connected with smaller leg with four bolts (70x70x6, 70x100x6 and 70x125x6)

Figure 6.38: Strain distribution over unconnected leg for equal-leg angles with constant connection length $L = 130$ mm
6.6.2.2 Effect of connection length

The effects of connection length on the net section behaviour were previously observed by many researchers (Chesson and Munse, 1963; Wu and Kulak, 1993; Bartels, 2000). The results from the parametric studies presented in Figures 6.40 and 6.41 clearly show that the net section efficiency generally increases with an increase in the connection length. However, once the connection length becomes relatively large the net section capacity reaches a plateau. Similar trends were observed by Chesson and Munse (1963), Wu and Kulak (1993) and Bartels (2000). Their test results showed that for small connection lengths, an increase in the connection length resulted in a substantial increase in the net section resistance. Yet, for large connection lengths, an increase in the connection length had a trivial increase in the net section capacity. The strain distributions along the unconnected leg for the same size angle but with different connection lengths presented in Figure 6.42 show that for small connection lengths the unconnected leg is less stressed and consequently less effective in resisting the tensile load.

Figure 6.39: Strain distribution over unconnected leg for three unequal-leg angles connected with the smaller leg with constant connection length $L = 130$ mm
The effect of the connection length on the net section efficiency is not considered in the design provisions of ENV 1993-1-4 (1996) nor in SCI/Euro Inox (1996). In the latest versions of these standards, the effect of the connection length is considered in terms of spacing between bolts in the loading direction $p_1$. The AISC (2005) accounts for the connection length explicitly by basing the net section efficiency on both the connection length and connection eccentricity.

To investigate whether the number of bolts in the loading direction or the connection length affects the net section efficiency, the net section efficiencies for connections with a constant connection length of 195 mm, but different numbers of bolts in one row (different $p_1/d_0$ ratios) are compared. Tables 6.14 and 6.15 summarise the results obtained for equal-leg angles and unequal-leg angles respectively. For both types, it is clear that the angle net section capacity is constant as long as the connection length is constant, regardless of the number of bolts. This emphasises the fact that it is the connection length that affects the net section behaviour rather than the spacing between the bolts ($p_1/d_0$ ratio). The strain distribution shown in Figure 6.43 confirms that for connections with the same length the efficiency of the net section is the same irrespective of the number of bolts.
Figure 6.40: Net section efficiency for equal-leg angles with different connection length

Figure 6.41: Net section efficiency for unequal-leg angles connected with smaller leg with different connection length
Figure 6.42: Strain distribution over unconnected leg for 100x100x10 angle with four bolts

Figure 6.43: Strain distribution over unconnected leg for 70x100x6 angle with a constant connection length $L = 195$ mm
### Table 6.14: Connections with the same length but different numbers of bolts

*(equal-leg angles)*

<table>
<thead>
<tr>
<th>Angle dimensions (AxAxt) (mm)</th>
<th>x (mm)</th>
<th>p₀/d₀ (mm)</th>
<th>L (mm)</th>
<th>No. of bolts</th>
<th>Net section efficiency U</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>FE model</td>
</tr>
<tr>
<td>50x50x5</td>
<td>14.3</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.94</td>
</tr>
<tr>
<td>50x50x5</td>
<td>14.3</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.94</td>
</tr>
<tr>
<td>50x50x5</td>
<td>14.3</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.94</td>
</tr>
<tr>
<td>70x70x6</td>
<td>19.7</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.91</td>
</tr>
<tr>
<td>70x70x6</td>
<td>19.7</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.92</td>
</tr>
<tr>
<td>70x70x6</td>
<td>19.7</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.93</td>
</tr>
<tr>
<td>90x90x8</td>
<td>25.5</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.84</td>
</tr>
<tr>
<td>90x90x8</td>
<td>25.5</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.86</td>
</tr>
<tr>
<td>90x90x8</td>
<td>25.5</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.87</td>
</tr>
<tr>
<td>100x100x10</td>
<td>28.7</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.83</td>
</tr>
<tr>
<td>100x100x10</td>
<td>28.7</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.83</td>
</tr>
</tbody>
</table>
Table 6.15: Connections with the same length but different numbers of bolts (unequal-leg angles)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>50x75x5</td>
<td>24.4</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.82</td>
<td>0.43</td>
<td>0.55</td>
<td>0.51</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>50x75x5</td>
<td>24.4</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.81</td>
<td>0.43</td>
<td>0.55</td>
<td>0.44</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>50x75x5</td>
<td>24.4</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.83</td>
<td>0.43</td>
<td>0.55</td>
<td>0.37</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>50x100x5</td>
<td>35.3</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.69</td>
<td>0.34</td>
<td>0.52</td>
<td>0.41</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td>50x100x5</td>
<td>35.3</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.67</td>
<td>0.34</td>
<td>0.52</td>
<td>0.35</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td>50x100x5</td>
<td>35.3</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.70</td>
<td>0.34</td>
<td>0.52</td>
<td>0.29</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td>70x100x6</td>
<td>31.7</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.76</td>
<td>0.49</td>
<td>0.58</td>
<td>0.55</td>
<td>0.84</td>
<td></td>
</tr>
<tr>
<td>70x100x6</td>
<td>31.7</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.78</td>
<td>0.49</td>
<td>0.58</td>
<td>0.47</td>
<td>0.84</td>
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</tr>
<tr>
<td>70x100x6</td>
<td>31.7</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.79</td>
<td>0.49</td>
<td>0.58</td>
<td>0.39</td>
<td>0.84</td>
<td></td>
</tr>
<tr>
<td>70x125x6</td>
<td>42.4</td>
<td>7.5</td>
<td>195</td>
<td>2</td>
<td>0.66</td>
<td>0.41</td>
<td>0.56</td>
<td>0.46</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>70x125x6</td>
<td>42.4</td>
<td>3.75</td>
<td>195</td>
<td>3</td>
<td>0.68</td>
<td>0.41</td>
<td>0.56</td>
<td>0.40</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>70x125x6</td>
<td>42.4</td>
<td>2.5</td>
<td>195</td>
<td>4</td>
<td>0.69</td>
<td>0.41</td>
<td>0.56</td>
<td>0.33</td>
<td>0.78</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 6 – GUSSET PLATE CONNECTIONS

6.7 PROPOSED DESIGN PROVISIONS

6.7.1 Connections with two or more bolts

Bartels (2000) conducted a linear regression analysis as a basis for a design equation for net section efficiency for tension members using their test results. In their proposal, the net section efficiency was assumed to be linearly related to both connection eccentricity and connection length as follows:

\[ U_{\text{Bartels}} = c_1 + c_2 \times c_3 L \]  

(6.14)

where \( c_1 \), \( c_2 \) and \( c_3 \) are constants.

Chesson and Munse (1963) recommended that the influence of the connection eccentricity \( x \) and connection length \( L \) on the net section efficiency can be suitably represented by the \( x/L \) ratio. Using the experimental data, the expression given by Equation 6.15 was found to be suitable. This equation has been adopted by the American Institute of Steel Construction AISC since 1978 (Easterling and Giroux, 1993).

\[ U_{\text{Chesson}} = 1 - \frac{x}{L} \]  

(6.15)

These two forms of the net section efficiency were examined in the light of the parametric study results in Section 6.6. The results from the FE models discussed earlier show that, while, the net section efficiency of the tension members decreases approximately linearly with an increase of connection eccentricity, it is clear that the relationship between the net section efficiency and the connection length is not linear. Therefore, a multiple linear expression combining \( x \) and \( L \) similar to that proposed by Bartels (2000) would not be suitable. Instead, an empirical relationship similar to that recommended by Chesson and Munse (1963) is investigated in a format given in Equation 6.16.
CHAPTER 6 – GUSSET PLATE CONNECTIONS

\[ U_{FE} = q_1 + q_2 \left( \frac{x}{L} \right) \]  
(6.16)

where \( q_1 \) and \( q_2 \) are constants. A linear regression analysis has been performed to determine \( q_1 \) and \( q_2 \), and to assess the quality of the formula in Equation 6.16. Figure 6.44 demonstrates that the format in Equation 6.16 predicts the net section efficiency with very good accuracy with a coefficient of determination \( R^2 \) of 0.89. The regression analysis suggests that the net section efficiency \( U \) is related to the connection eccentricity and connection length according to the following equation:

\[ U_{FE} = 0.98 - 1.12 \left( \frac{x}{L} \right) \]  
(6.17)

Net section efficiency \( U_{prop} \) is proposed by modifying Equation 6.17 slightly as follows:

\[ U_{prop} = 0.95 \left( 1 - 1.15 \frac{x}{L} \right) \leq 0.95 \]  
(6.18)

Hence, the ultimate net section capacity of single angles connected by two or more bolts in a single line may be taken as follows:

\[ N_{u,\text{prop mult}} = \frac{U_{prop} A_{net} f_u}{\gamma_{M2}} \]  
(6.19)

6.7.2 Connections with a single bolt

From Table 6.13, it can be seen that, while the design provisions in ENV 1993-1-4 (1996) and SCI/Euro Inox (1996) are unsafe, those recommended by EN 1993-1-4 (2006) and SCI/Euro Inox (2006) are marginally conservative. Therefore, the ultimate net section capacity for angles connected by one bolt given in Equation 6.2 is adopted with a slight modification as presented in Equation 6.20 – the whole net section of the connected leg can be assumed to be effective in resisting the tensile load.
\[
N_{u,\text{prop,single}} = \frac{(A - d_0)t_c f_u}{\gamma_{M2}} \tag{6.20}
\]

where \(A\) and \(t_c\) are the width and the thickness of the connected leg respectively.

6.7.3 Reliability analysis

Statistical analyses according to the procedure given in Annex D of EN 1990 (2002) have been carried out so as to assess the reliability of the proposed design equations and to determine a suitable partial safety factor \(\gamma_{M2}\) against fracture. The analyses have been conducted following the steps presented in Chapter 4. In these analyses, the data sets obtained from the parametric studies were used.

The design net section resistance of an angle connected to a gusset plate with two or more bolts in a single line is:

Figure 6.44: Net section efficiency from the parametric studies for equal-leg and unequal leg angle connections with two or more bolts
\[ r_t = g_n(X) = U_{\text{prop}} A_{\text{net}} f_u \] (6.21)

and for angles connected with a single bolt is:

\[ r_t = g_n(X) = (A - d_b) t_c f_u \] (6.22)

Comparisons between the net section resistances obtained from the FE models and Equations 6.21 and 6.22 are presented in Figures 6.45 and 6.46 respectively. With a coefficient of determination almost equal to 1.0, it can be concluded that the proposed equations predict the net section capacity with good accuracy. A summary of the statistical analyses is listed in Table 6.16.

Figure 6.45: Comparison between theoretical and numerical resistances for angles connected with two or more bolts in a line
Table 6.16: Summary of statistical analysis results for net section resistance for angles

<table>
<thead>
<tr>
<th>Data set</th>
<th>Number of tests</th>
<th>k_d,n</th>
<th>b</th>
<th>R^2</th>
<th>V_δ</th>
<th>V_r</th>
<th>γ_{M2}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two or more bolts (Equation 6.21)</td>
<td>99</td>
<td>3.19</td>
<td>1.04</td>
<td>0.989</td>
<td>0.0702</td>
<td>0.0906</td>
<td>1.067</td>
</tr>
<tr>
<td>Single bolt (Equation 6.22)</td>
<td>13</td>
<td>4.08</td>
<td>1.03</td>
<td>0.999</td>
<td>0.0513</td>
<td>0.0767</td>
<td>1.061</td>
</tr>
</tbody>
</table>

The maximum required partial safety factor γ_{M2}, given in Table 6.16 is 1.067. Thus, the same recommended value of 1.1 in Chapter 4 is adopted for Equations 6.21 and 6.22. To ensure a ductile failure of a tension member, the ultimate resistance of the net cross-section should be greater than the yield resistance of the gross section (A_{gross} f_y).

6.8 CONCLUDING REMARKS

Previously tested specimens for bolted gusset plate connections in stainless steel were successfully simulated and verified by employing numerical models. The special
features of ABAQUS/Explicit have shown that it is suitable for dealing with the complexities of such models, which arise from the large number of contact interfaces. The validated FE models were then used to investigate the behaviour of the net section rupture of single angles bolted to gusset plate and subjected to tensile force. Comparisons between the parametric study results and the existing design provisions for these members showed that the net section resistance is conservative. Based on the parametric study results, design equations were proposed and their reliability has been demonstrated using statistical analyses. The proposed formulae – Equations 6.21 and 6.22 may be regarded as the most suitable currently available.
CHAPTER 7

CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

7.1 CONCLUSIONS

In this section, the important outcomes of the current research are summarised.

Owing to the lack of detailed comprehensive studies on stainless steel connections, the recommended design rules for these structural elements have, until now, been very largely based on those for carbon steel connections with only minor adjustments aimed to cover differences in mechanical behaviour. The objective of this research was, therefore, to comprehensively understand the behaviour of these particular structural components under static loads, and subsequently to develop design equations that reflect the exact mechanical characteristics of stainless steel.

The review of the previous studies on both carbon steel and stainless steel connections in Chapter 2 showed that very limited investigations into the behaviour of stainless steel connections have been carried out during recent decades. This survey allowed those particular areas that required more detailed investigation to be identified.
The approach employed in conducting the present research was to use the experimental test results for validating numerical models at first; these models were then utilised to study a much wider range of key design variables.

In Chapter 3, all the available test data with sufficient information to be used in a quantitative sense has been reviewed and employed to develop numerical models. The nonlinearity of the material stress-strain relationship, which is the most prominent mechanical difference between stainless steel and carbon steel, was described using the modified Ramberg-Osgood material model. Sophisticated FE models where all the key characteristics of lap bolted connections: hole clearance, friction and bolt preload, have been successfully developed. Numerical analyses including geometrical, material and boundary nonlinearities have been performed. The comparisons between test results and the results obtained from the developed FE models, in terms of deformation response, load carrying capacities, failure modes and deformed shapes, demonstrated that the FE models can accurately replicate the behaviour of the tested specimens. These models have been employed to understand the resistance of stainless steel lap connections against two common modes of failure in Chapters 4 and 5, these are net section rupture and bearing failure.

The net section design equation for cold-formed carbon steel, which is adopted for stainless steel connections, is essentially developed based on the assumption that the cold-formed carbon steel is not sufficiently ductile to permit the extension of plastic strains from the edge of the bolt hole, where stress is concentrated, towards the edge of the connected plate, and that at failure, the nominal stress over the critical net section is less than the material ultimate tensile strength. This assumption was investigated for stainless steel connections in Chapter 4. The FE models, which were developed in Chapter 3, were employed to perform parametric studies to investigate the influence of the key parameters on the net section capacity. The results showed that the ductility of stainless steel is sufficient to overcome the deleterious effects of the concentration of stresses at the edge of the bolt hole, thereby the nominal stress at the critical net section at failure always attains the material ultimate tensile strength and therefore, the reduction factor $k_r$ was found to be unnecessary. Finally, the findings of the parametric studies,
together with the test results, were used in a statistical evaluation of a net section design equation suitable for both thick and thin materials.

In Chapter 5, the bearing behaviour of stainless steel lap connections was studied. It is evident that the deformation behaviour of carbon steel and stainless steel differs significantly. For carbon steel connections, once material yielding takes place, the load-deformation response becomes approximately flat. On the other hand, for stainless steel connections, due to the high nonlinearity and the marked strain-hardening of the material, the load-deformation curve does not exhibit such flattening; rather, the curve continues to rise. Therefore, the limiting deformations often used to define the bearing failure of carbon steel connections were found to be inappropriate for stainless steel connections. Instead, a rational strain-based criterion that defines the true bearing failure has been suggested following the observation of a consistent pattern of behaviour from a number of test studies as well as the FE models. Extensive parametric studies were carried out by employing the verified FE models. Bearing design equations for both thick and thin connections that consider two scenarios, depending on the deformation at the serviceability limit state, have been devised: one considers the deformation at service load and the other does not. In these equations the bearing capacity is defined in terms of the material ultimate strength $f_u$ and therefore, they are consistent with the provisions for carbon steel connections. The proposed equations provide a modest enhancement in capacity compared to the EC3 approach as well as being simpler to use.

Despite the importance and prevalence of gusset plate connections in steel structures, only a single study, as discussed in Chapter 2, has been conducted to investigate the behaviour of these form of connections in stainless steel. As a direct result of the sparse information, the design provisions in EN 1998-1-8 (2005) for carbon steel angles connected to gusset plates are precisely those recommended for stainless steel material in EN 1993-1-4 (2006) and the SCI/Euro Inox Design Manual (2006). In Chapter 6, a rigorous investigation into the net section capacity of austenitic stainless steel single angles under pure static tensile loads was carried out. Firstly, test data on stainless steel gusset plate connections has been simulated using numerical models. The capabilities of ABAQUS/Explicit were exploited in modelling and validating the tested specimens. The most influential parameters controlling the net section rupture were investigated by
CHAPTER 7 – CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

producing additional FE results. Comparisons between the FE results and the current design provisions showed that the net section design provisions are conservative. Thus, generic net section design equations were derived and a suitable partial factor of safety was suggested by performing reliability analyses.

Generally, the investigations in this thesis have provided understanding into the behaviour of commonly used types of stainless steel connections and have allowed improvement of the design rules for these critical structural components.

7.2 SUGGESTIONS FOR FURTHER WORK

The findings of this study can be used as a good basis for extending the investigation on stainless steel connections. In this section, some possible areas for further work are presented.

7.2.1 Extending the current study

The net section and bearing design equations for lap bolted connections were proposed based on the results for austenitic and ferritic stainless steel materials. Despite the similarities in the stress-strain relationships for all stainless steel grades, the validity of these design equations for lean duplex stainless steel is important, since the importance of this stainless steel grade in structural application is growing. This can be investigated with the finite element models developed in this study.

In addition, the proposed net section design equation should be investigated for its applicability to lap connections with staggered bolts, in which the path of net section rupture depends on the spacing between the bolts in both the loading direction and perpendicular to the loading direction.

The bearing behaviour of bolted connections discussed in Chapter 5 could be expanded to study the behaviour of multi-bolt connections. For example, the influence of the spacing between bolts on the contribution of each bolt to the total resistance needs to be investigated.
Further experimental tests on lap bolted connections with a wider range of configurations need to be conducted to increase the currently available information. These tests can be used to validate further FE models.

The outcomes of this investigation can also be used to improve testing procedures to ensure that the measurements taken during experiments provide a complete description of the connection failure. The bearing behaviour can be better observed from laboratory results by employing the proposed strain-based criterion that defines the bearing capacity combined with the suggested criterion that distinguishes between tear-out and bearing failure modes. The strains in tested connections would need to be scrutinised in the connected plates in the region in front of the bolt hole where high concentrations of strains occur and consequently fracture initiates. Moreover, the deformations of the reference points presented in Chapter 5 can also be recorded during testing, in order to identify tear-out and bearing failures more precisely than the visual inspection currently used.

The current design rules in Eurocode 3 for gusset plate connections in both carbon steel and stainless steel are limited to single angles and do not cover other section types, such as tees connected either by the webs or flanges and channels connected through their webs. In the present study, the net section behaviour of stainless steel single angles has been investigated and suitable design equations are proposed. The developed FE models can also be used to study the net section resistance for some other common cases, not only for stainless steel but also for carbon steel by incorporating the appropriate material properties into the FE models.

7.2.2 Further thoughts

Offshore structures are exposed to very aggressive environments. Stainless steel can provide a more cost-effective solution for these structures than ordinary carbon steel. Due to the nature of their service, offshore structures are susceptible to fire and explosion. As discussed earlier, the current study is concerned with the behaviour of stainless steel connections under static loading at room temperature. Since the mechanical behaviour of stainless steel, particularly the stress-strain relationship and the
elastic modulus at elevated temperature is different from that at room temperature, the
behaviour of stainless steel connections under fire should be investigated to propose
efficient design rules for this scenario. In addition, because the connections in these
circumstances may be exposed to blast loading, the response of bolted connections to
this kind of dynamic loading with the consideration of material properties at elevated
temperature should also be considered.
REFERENCES


AISI (1986). Specification for the design of cold-formed steel structural members. *American Iron and Steel Institute*. Washington, DC, USA.


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