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Design Manual For Structural Stainless Steel - Commentary

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PREFACE

Third Edition

This Third Edition of the Design Manual has been prepared by The Steel Construction Institute as a deliverable of the RFCS Project - *Valorisation Project – Structural design of cold worked austenitic stainless steel* (contract RFS2-CT-2005-00036). It is a complete revision of the Second Edition, extending the scope to include cold worked austenitic stainless steels and updating all the references to draft Eurocodes. The Third Edition refers to the relevant parts of EN 1990, EN 1991 and EN 1993. The structural fire design approach in Section 7 has been updated and new sections on the durability of stainless steel in soil and life cycle costing have been added.

Three new design examples have been included to demonstrate the appropriate use of cold worked stainless steel. They were completed by the following partners:

- Universitat Politècnica de Catalunya (UPC)
- The Swedish Institute of Steel Construction (SBI)
- Technical Research Centre of Finland (VTT)

A project steering committee, including representatives from each partner and sponsoring organisation, oversaw the work and contributed to the development of the Design Manual. The following organizations participated in the preparation of the Third Edition:

- The Steel Construction Institute (SCI) (Project co-ordinator)
- Centro Sviluppo Materiali (CSM)
- CUST, Blaise Pascal University
- Euro Inox
- RWTH Aachen, Institute of Steel Construction
- VTT Technical Research Centre of Finland
- The Swedish Institute of Steel Construction (SBI)
- Universitat Politècnica de Catalunya (UPC)

Preface to the Second Edition

This Design Manual has been prepared by The Steel Construction Institute as a deliverable of the ECSC funded project, *Valorisation Project – Development of the use of stainless steel in construction* (contract 7215-PP-056). It is a complete revision of the *Design manual for structural stainless steel*, which was prepared by The Steel Construction Institute between 1989 and 1992 and published by Euro Inox in 1994.

This new edition takes into account advances in understanding in the structural behaviour of stainless steel over the last 10 years. In particular, it includes the new design recommendations from the recently completed ECSC funded project, *Development of the use of stainless steel in construction* (contract 7210-SA/842), which has led to the scope of the Design Manual being extended to cover circular hollow sections and fire resistant design. Over the last ten years a great many new European standards have been issued covering stainless steel material, fasteners, fabrication, erection, welding etc. The Design Manual has been updated to make reference to current standards and data in these standards.

A project steering committee, including representatives from each partner, sub-contractor and sponsoring organisation, oversaw the work and contributed to the development of the Design Manual.

The worked examples were completed by the following partners:

- Centre Technique Industrial de la Construction Métallique (CTICM)
- Luleå Institute of Technology
- RWTH Aachen
- Technical Research Centre of Finland (VTT)
- The Steel Construction Institute (SCI)

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- Research Fund for Coal and Steel (RFCS) (*formerly*, European Coal and Steel Community (ECSC))
- Euro Inox

The contribution made to this and the previous two editions by the European stainless steel producers and other organisations is also gratefully acknowledged.

The following people assisted in the preparation of the Commentary to the Third Edition:

- Dr Leroy Gardner (Imperial College)
- Marios Theofanous (Imperial College)
- Benoit van Hecke (Euro Inox)

Their assistance is gratefully acknowledged.

FOREWORD

This Design Manual has been prepared for the guidance of engineers experienced in the design of carbon steel structural steelwork though not necessarily in stainless steel structures. It is not in any way intended to have a legal status or absolve the engineer of responsibility to ensure that a safe and functional structure results.

The Manual is divided into two parts:

- Part I - Recommendations
- Part II - Design Examples

The Recommendations in Part I are formulated in terms of limit state philosophy and, where appropriate, are in compliance with the following Parts of Eurocode 3 *Design of steel structures*:

EN 1993-1-1 *Design of steel structures: General rules and rules for buildings*

EN 1993-1-2 *Design of steel structures: Structural fire design*

EN 1993-1-3 *Design of steel structures: General rules: Supplementary rules for cold-formed members and sheeting*

EN 1993-1-4 *Design of steel structures: General rules: Supplementary rules for stainless steels*

EN 1993-1-5 *Design of steel structures: Plated structural elements*

EN 1993-1-8 *Design of steel structures: Design of joints*

EN 1993-1-9 *Design of steel structures: Fatigue*

EN 1993-1-10 *Design of steel structures: Material toughness and through-thickness properties*

This Design Manual gives recommended values for certain factors. These values may be subject to modification at a national level by the National Annexes.

The Design Examples contained in Part II demonstrate the use of the recommendations. A cross-reference system locates that section of the examples corresponding to a particular recommendation.

The Recommendations and Design Examples are available at www.steel-stainless.org/designmanual. They are also available at Steelbiz, an SCI technical information system (www.steelbiz.org), and from the Euro Inox web site (www.euro-inox.org).

A Commentary to the Recommendations, which includes a full set of references, is also available online at these web sites. The purpose of the Commentary is to allow the designer to assess the basis of the recommendations and to facilitate the development of revisions as and when new data become available. Opportunity is taken to present the results of various test programmes conducted specifically to provide background data for the Design Manual. The Recommendations, Design Examples and Commentary are also available on CD from Euro Inox.

An online design facility is available at www.steel-stainless.org/software for designing cold-formed stainless steel members subject to axial tension, bending or axial compression. The design facility calculates section properties and member resistances in accordance with the Recommendations in this Design Manual.

The design recommendations presented in this document are based upon the best knowledge available at the time of publication. However, no responsibility of any kind for injury, death, loss, damage or delay, however caused, resulting from the use of the recommendations can be accepted by the project partners or others associated with its preparation.

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C.1 INTRODUCTION

C.1.1 Scope

There are many different types and grades of stainless steel (see Section C.3.1.1). These have been formulated over the last 80 years or so to optimise certain characteristics such as corrosion resistance in specific environments, weldability and mechanical properties. The Recommendations in this Design Manual are applicable to the grades of stainless steel commonly used in construction, as given in Table 3.1.

The Design Manual concentrates on the design of members and elements, not on the behaviour and design of frameworks. Thus no recommendations are given for elastic or plastic global analysis (except that elastic global analysis should be used) and reference should be made to carbon steel codes as necessary. In particular, the designer will need to consider second order effects in stainless steel sway frames. These could be potentially greater than in carbon steel frames if the steel is stressed into the non-linear portion of the stress-strain curve.

No limits to thickness are given; the normal limitations for carbon steel do not apply due to the superior performance of stainless steel materials. However, there will be practical limits for the cold forming of members (approximately 20 mm for the austenitic grades and 15 mm for duplex grade 1.4462).

Pressure vessels, pipework and structures within nuclear installations are not covered. Other codes, such as the ASME pressure vessel code[1], may be consulted.

C.1.2 Symbols and conventions for member axes

As stated, the notation of EN 1993-1-1[2] has been generally adopted, in which extensive use is made of subscripts. It is not necessary to use the subscripts if clarity is not impaired.

Attention is drawn to the use of the x axis as being along the length of the member, and the major axis of bending as being about y-y.

C.2 BASIS OF DESIGN

C.2.1 General requirements

The aims in designing a stainless steel structure are no different from those in carbon steel structures. That is a safe, serviceable and durable structure should result. As well as the more obvious considerations such as strength and stability, the design of a structure should take account of the following:

- Safe transport and handling.
- Safe means of interconnection.
- Stability during erection.

One designer should be responsible for ensuring the overall stability of the structure, particularly if stainless steel is used in conjunction with other materials. In the design of the stainless steel structure, the assumed restraint and stability afforded by other materials should be clearly stated and made known to the engineer responsible.

C.2.2 Limit state design

In limit state design, the performance or capacity of the structure or its components is assessed against various criteria (the limit states) at appropriate load levels. For carbon steel structures, the designer is mainly concerned with the ultimate limit states, which potentially could lead to loss of life, and serviceability limit states, which could lead to loss of function. The reduction in structural performance of carbon steel building structures due to corrosion is not usually specifically considered by the structural designer, reliance instead being place upon paint or other protective coatings. Where corrosion is likely to affect performance, as for marine or offshore structures, the use of a sacrificial corrosion allowance on the thickness or of cathodic protection is common. However, for stainless steel, anti-corrosion measures should form an integral part of the design, from material selection to detailing of member and joints, and must be carried through fabrication and erection. Thus, in Section 2.2 of the Recommendations, the durability limit state is on an equal footing to the ultimate and serviceability limit states.

In Section 2.2, creep is given as an example of a serviceability limit state. Stainless steel can exhibit noticeable creep-like deformations at room temperature if stresses exceed approximately two thirds of the 0,2% proof strength. It is arguable as to whether creep should be considered as an ultimate limit state or as a serviceability limit state. For pressure vessels, creep rupture is clearly an ultimate limit state. In other structures, the situation is not as clear. For instance, in a column any additional creep deformation will influence the load carrying capacity of that column and hence creep should perhaps be considered at the ultimate limit state. However, for beams, creep deformations are manifested by increased beam deflections that may exceed permissible levels; in this instance creep has to be considered at the serviceability limit state. In this Design Manual, the view has been taken that the ultimate limit state can be exceeded by a short-term overload condition, and that creep deformations would be manifested before the overload condition occurs. Thus, if creep were considered at the serviceability limit state it would not be significant at the ultimate limit state for the load factors used in this Design Manual.

The values of the partial safety factor for resistance, γ_M, given in Table 2.1 are the recommend values in EN 1993-1-4[3]. Note that certain European countries may specify modified γ_M values in their National Annexes, and, where this is the case, these values must be used in the place of the values given in EN 1993-1-4.

C.2.3 Loading

It is the responsibility of the designer to consider all load effects (dead loads, imposed loads, effects of temperature and settlement, etc.) and establish the most onerous load case for each member.

As for the γ_M factors, different values of γ_F may be set in the National Annex for the country for which the structure is being designed.

For offshore applications, the partial safety factor for loads for the in-place condition are taken from API RP2A[4]. API RP2A also recommends factors for transportation, earthquake loadings, etc. and should therefore be consulted. Generally, the offshore factors are higher than those onshore. This is generally intended to achieve a higher level of reliability.

C.3 MATERIALS: PROPERTIES AND SELECTION

C.3.1 Material grades

C.3.1.1 Introduction

Stainless steels can be classified into five groups, according to their chemical composition (see Figure C.3.1) and thermomechanical treatment. Each group has different properties, particularly in respect of strength, corrosion resistance and ease of fabrication.

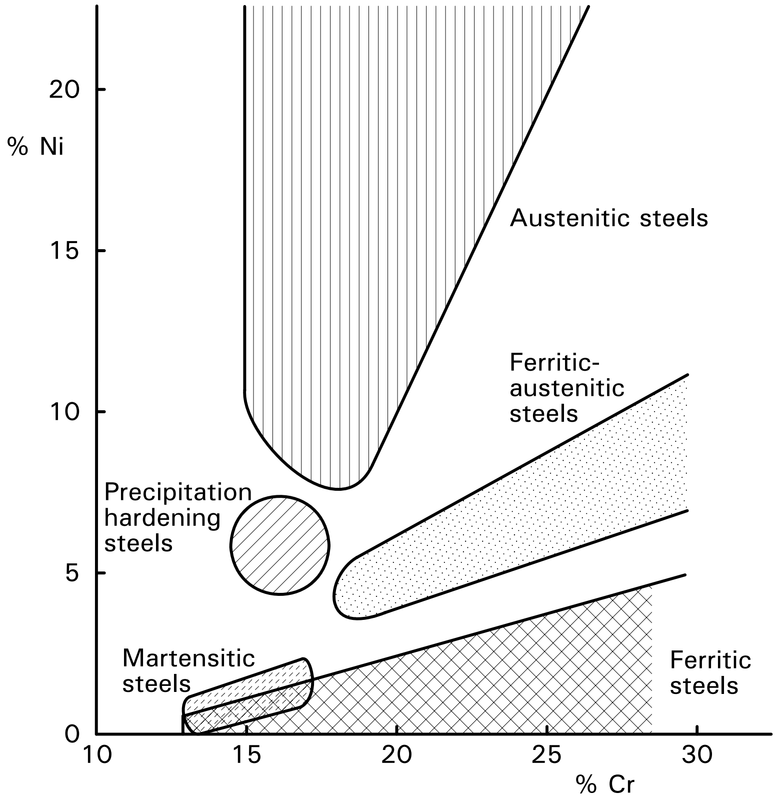


Figure C.3.1 *Classification of stainless steels according to nickel and chromium content*

The five groups can be summarised thus:

Austenitic stainless steels

These are the most commonly used stainless steels. They have an austenitic microstructure at room temperature and generally contain relatively high amounts of nickel. They have high ductility, are easily formed, are readily weldable and offer good corrosion resistance. Their strengths are reasonable and they can only be hardened (i.e. made stronger) by cold working.

Ferritic stainless steels

The ferritic stainless steels contain relatively little nickel and have a ferritic microstructure. Ductility, strength, formability and weldability are not as good as in the austenitic steels. Although they are generally not as corrosion resistant as the austenitic grades, they are superior when considering stress corrosion cracking. As for the austenitic grades, they can only be hardened by cold working.

Martensitic stainless steels

These steels can be hardened by heat treatment and are not normally used in welded fabrication. High strengths can be achieved with these steels but in other respects they are poorer than the other groups.

Duplex stainless steels

These steels have a mixed microstructure and combine the best of the properties of the austenitic and ferritic groups. Compared to the austenitic group they have higher mechanical strengths, similar weldability, lower formability and similar or higher corrosion resistance especially with respect to stress corrosion cracking. They are hardened by cold working.

Precipitation hardening steels

These offer the highest strengths, obtained by suitable heat treatments. They are not normally used in welded fabrications.

Further information on the various groups and types of stainless steels may be found in standard texts[5,6].

Most structural applications use austenitic grades 1.4301, 1.4401 or their low carbon variants 1.4307 and 1.4404. A wide range of product forms is available in these grades. (Note that in Germany, the low carbon version of 1.4301 widely used is grade 1.4306, a slightly higher alloyed version of 1.4307.) Experience of duplex grades 1.4462 and 1.4362 has been gathered in the offshore industry; they offer advantages in mechanical strength and have superior resistance to stress corrosion cracking. For large volume applications requiring high strength, the austenitic grade 1.4318 or a lean duplex such as grade 1.4162 can prove very cost effective.

If there is any doubt as to which of these grades, or indeed any other grade, is suitable for a particular application, specialist advice should be sought. Stainless steel producers commonly give such advice, often free of charge.

The Recommendations are only intended for the rolled forms of the selected alloys. Cast forms generally have equivalent corrosion resistance to that of the rolled forms but several differences exist. One of the more important of these is that the microstructure of cast austenitic stainless steels contains a greater amount of ferrite. This not only facilitates weld repair of castings but also increases the resistance to stress corrosion cracking. Cast steels also differ in mechanical properties, physical properties and chemical composition. Because of the formation of larger grain sizes and other differences in microstructure, mechanical properties of cast steels exhibit a wider range and are generally inferior to rolled steels.

C.3.1.2 Relevant standards

The European material standard for stainless steel is EN 10088, *Stainless Steels*[7] and this covers flat products and long products. Fasteners are covered in EN ISO 3506, *Corrosion-resistant stainless steel fasteners*[8].

When specifying for ordering purposes it is important to provide a complete specification that should include:

- The desired quantity.
- The type of manufacture (hot rolled or cold rolled) and the product form (strip or sheet/plate).
- Where an appropriate dimensional standard is available, the number of the standard, plus any choice of requirements.
- If there is no dimensional standard, the nominal dimensions and tolerances required.
- The type of material (steel) and its name or number designation with the relevant European standard (EN 10088).
- If, for the relevant grade, more than one treatment condition is covered, the symbol for the desired heat treatment or cold worked condition.
- The desired process route and surface finish.
- If an inspection document is required, its designation according to EN 10204[9].

Reference 10 gives tables of chemical compositions, mechanical and physical properties for stainless steels to EN 10088; an interactive database of properties is also available at www.euro-inox.org/technical_tables .

C.3.2 Mechanical behaviour and design values of properties

C.3.2.1 Basic stress-strain behaviour

As well as non-linearity, the stress-strain characteristics of stainless steels also display non-symmetry of tensile and compressive behaviour and anisotropy (differences in behaviour of coupons aligned parallel and transverse to the rolling direction). In the annealed (softened) condition, the stress-strain curves tend to be more non-linear in tension than in compression. Tests on both cold and hot rolled material indicate higher strengths transverse to the rolling direction than in the direction of rolling[11]. Unidirectional work hardening results in a reduced proof stress in the direction opposite to the work hardening direction. As for other stainless steel grades, even for small levels of work hardening, this reduction can be such that the proof stress in compression of a plate work hardened by stretching is below its original value before work hardening[12].

The degree of non-linearity, non-symmetry and anisotropy varies between grades of stainless steel. For an annealed material, the differences due to non-symmetry and anisotropy are not large but nevertheless they have been taken into account in Appendix C. Except for thin sheets (less than, say, 4 mm for which work hardening imparted during rolling may have an impact), there does not appear to be a significant thickness effect on the relationship between the four basic stress-strain curves[13,14].

In discussing the form of the stress-strain curve, it is helpful to consider the Ramberg-Osgood idealised form[15] given by:

$$\varepsilon = \frac{\sigma}{E} + 0{,}002\left(\frac{\sigma}{\sigma_{0,2}}\right)^{n}$$

Inspection of this equation shows that there are three independent parameters required to define a particular stress-strain curve, i.e.

E is Young's modulus

$\sigma_{0,2}$ is the 0,2% proof strength

n is an index

The degree of non-linearity of the stress-strain curve is characterised by the index n; lower n values imply a greater degree of non-linearity, see Figure C.3.2.

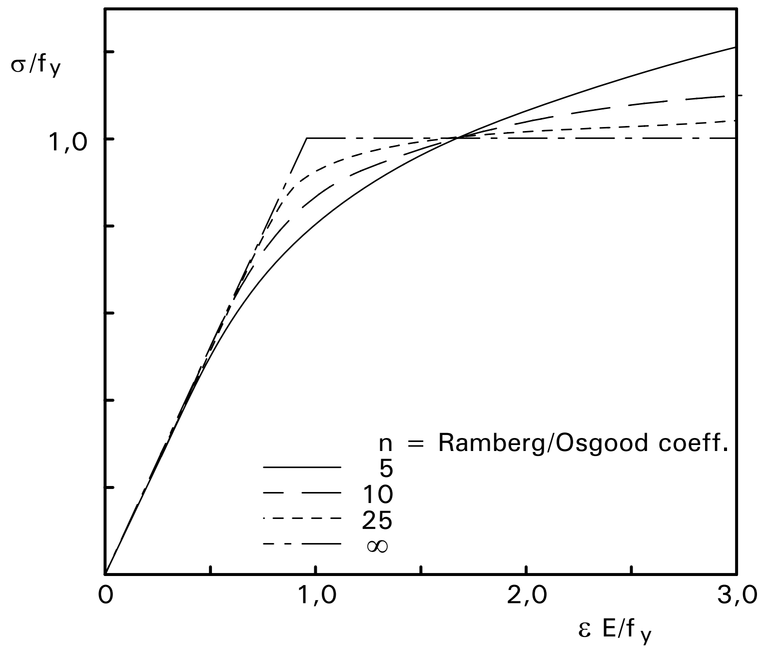


Figure C.3.2 *Effect of the parameter n on the non-linearity of the stress-strain curve*

The value of n may be obtained from the ratio of the stress at the limit of proportionality (conventionally the 0,01% proof strength, $\sigma_{0,01}$) to the 0,2% proof strength, $\sigma_{0,2}$, as follows:

$$n = \frac{\log(0{,}05)}{\log(\sigma_{0,01} / \sigma_{0,2})}$$

and thus the ratio $\sigma_{0,01}/\sigma_{0,2}$ may also be used as an indicator of the degree of non-linearity.

Table C.3.1 shows the averaged stress-strain characteristics obtained from the test programme specifically carried out for the First Edition of this Design Manual[14].

Table C.3.1 *Representative values of stress-strain characteristics for materials in the annealed condition*

Material	Direction & Sense of Stress	0,2% Proof strength (N/mm²)	Modulus of elasticity (kN/mm²)	$\frac{\sigma_{0,01}}{\sigma_{0,2}}$	Index n
1.4307	LT	262	194	0,65	7,1
	LC	250		0,62	6,3
	TT	259	198	0,71	8,8
	TC	255		0,72	9,0
1.4404	LT	277	193	0,65	6,9
	LC	285		0,71	8,6
	TT	286	198	0,70	8,5
	TC	297		0,74	10,0
1.4462	LT	518	199	0,57	5,4
	LC	525		0,56	5,2
	TT	544	207	0,54	4,8
	TC	540		0,59	5,7

LT - Longitudinal tension
LC - Longitudinal compression
TT - Transverse tension
TC - Transverse compression

Note these values should be considered as representative and not as typical or characteristic values. Other data sources were also examined to select the design values in Appendix C[14].

From a structural point of view, the results in Table C.3.1 suggest that anisotropy and non-symmetry of annealed materials are not as important as the non-linearity.

The rounded stress-strain curve affects the strength and stiffness of a member, depending on the stress level in the member. In a compression member for instance, buckling failure is related to the associated value of the tangent modulus; thus, for failure stresses below the proof strength, it can be expected that a stainless steel column will tend to be weaker than a similar carbon steel column of the same proof strength. On the other hand, for failure stresses above the proof strength, a stainless steel column will be stronger than the corresponding carbon steel one. Further explanation is given in Section C.5.3.1.

Although the Ramberg-Osgood formulation gives excellent agreement with experimental stress-strain data up to the 0,2% proof strength, at higher strains the model generally over estimates the stress corresponding to a given level of strain. Mirambell and Real[16] recently proposed the use of two adjoining Ramberg-Osgood curves to achieve improved modelling accuracy at strains above the 0,2% proof strength. The basic Ramberg-Osgood expression is used up to the 0,2%proof stress, then a modified expression re-defines the origin for the second curve as the point of 0,2% proof stress, and ensures continuity of gradients. Figure C.3.3 demonstrates the improved accuracy at higher strains of this compound Ramberg-Osgood expression. Gardner[17] has proposed a modification to Mirambell and Real's model to describe compressive stress-strain behaviour.

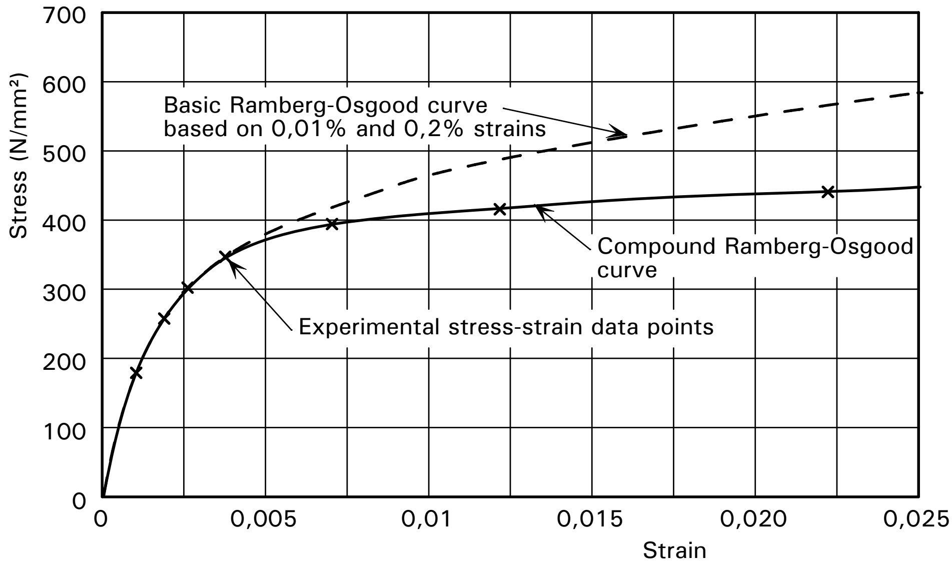


Figure C.3.3 *Comparison between compound and basic Ramberg-Osgood models*

C.3.2.2 Factors affecting stress-strain behaviour

Further information on cold working stainless steel and strain rate effects is available from Euro Inox[18,19].

Cold working

Stainless steels are generally supplied in the annealed (softened) condition and the mechanical properties given in EN 10088 mostly relate to material in this condition. However, austenitic stainless steels (and to a lesser extent duplex steels) develop high mechanical strengths when cold worked. In part this is due to a partial transformation of austenite to martensite. The degree of strength enhancement is affected by chemical composition[13,20]. Austenite stabilising elements, such as nickel, manganese, carbon and nitrogen tend to lower the rate of strength enhancement.

Figure C.3.4, taken from Reference 13, shows the effect of cold work on the 0,2% proof strength, the ultimate tensile strength and elongation at failure for a specific sample of 1.4307. Similar relationships apply to grade 1.4404. The corresponding curves for duplex 1.4462 are shown in Figure C.3.5 obtained from manufacturer's literature.

In general, anisotropy and non-symmetry increase with cold work. It is important to remember that welding or certain heat treatments will anneal, or partially anneal, the cold worked material. This will reduce the strength to some extent, but not below the strength in the annealed unwelded state[21,22]. Deflections may frequently govern the design of cold worked stainless steel rather than strength.

Cold working can occur at two stages in the production of a structural component - during production of the flat product and/or during fabrication of the finished structural component.

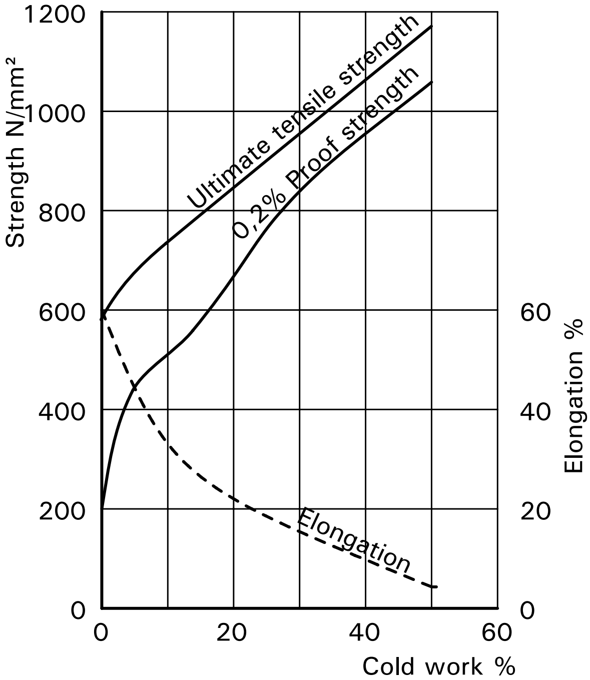


Figure C.3.4 *Effect of cold working on a sample of 1.4307 material*

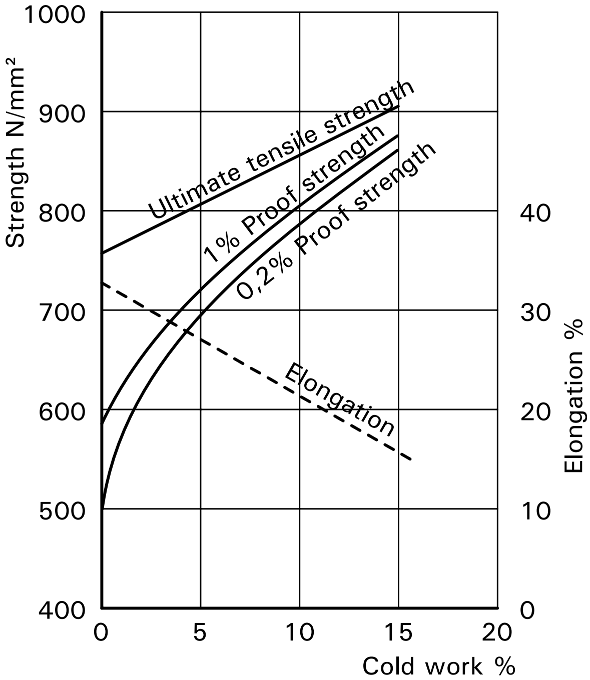


Figure C.3.5 *Effect of cold working on a sample of duplex 1.4462 material*

Cold working during production of the flat product

Stainless steel can be cold worked during production of the strip by a temper rolling or stretching process; the former process is more common. EN 10088 specifies five 0,2% proof strength conditions (CP350, 500, 700, 900 and 1100) for cold worked material. Alternatively, the standard allows material to be specified by its tensile strength level (C700, C850, C1000, C1150 and C1300). Table C.3.2 gives the strengths associated with these conditions, compared with the cold worked conditions or tempers given in the American Code[23].

Table C.3.2 *European and American specifications for strength levels in the cold worked condition for standard austenitic grades*

	Nominal strength class	0,2% proof strength 1) 2) (N/mm^2)	Ultimate tensile strength 3) 4) (N/mm^2)
EN 10088-2	Annealed	210-240	520-750
	CP350	350-500	5)
	CP500	500-700	5)
	CP700	700-900	5)
	CP900	900-1100	5)
	CP1100	1100-1300	5)
	C700	5)	700-850
	C850	5)	850-1000
	C1000	5)	1000-1150
	C1150	5)	1150-1300
	C1300	5)	1300-1500
SEI/ASCE - 8 - 02	Annealed	207	571
	1/16 hard	276	552-586
	1/4 hard	517	862
	1/2 hard	759	1034

1) Intermediate proof strength values may be agreed
2) The maximum product thickness for each proof strength level decreases with the proof strength
3) Intermediate tensile strength values may be agreed
4) Maximum product thickness for each tensile strength level decreases with the tensile strength.
5) Not specified

A recently completed ECSC-funded project studied the behaviour of cold worked stainless steel in the context of structural design in order to develop economic guidance[22]. Experimental and numerical analyses were carried out on material specimens, structural members and connections at room temperature and in fire in order to determine whether the design guidance in the Second Edition of the Design Manual was applicable to cold worked material up to the C850 or CP500 strength conditions. Generally the guidance was shown to be safely applicable, provided the effect of anisotropy was taken into account in the way described in Section 3.2.4.

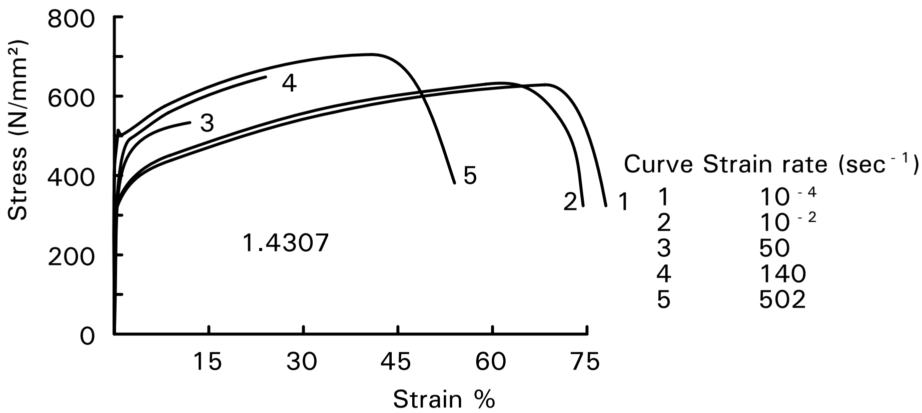
The use of cold worked material for structural applications has great potential that has not yet been exploited.

Cold working during fabrication of the finished structural component

This is generally known as cold forming, and typically occurs at the corners of sections where the 0,2% proof strength can rise between 20% and 100% higher than the 0,2% proof strength of the flat regions. Work has been carried out to develop an expression to predict the corner mechanical properties of cold formed stainless steel material[17,24,25,]. The increased strength is, however, localised at the position of bending (e.g. the corners of rectangular hollow sections). Note also that the increase in strength is dependent on the method of manufacture. For example, Gardner found that sections fabricated (from annealed material) by first forming the material into a circular hollow section, and then shaping it into a rectangular hollow section showed moderate strength enhancements in the flat regions and large enhancements in the corners. By comparison, sections fabricated by direct bending from a flat sheet had essentially unchanged properties in the flat regions, with large strength enhancements at the corners (but not as large as the enhancement with the indirect fabrication method)[17].

Strain-rate sensitivity

Most investigations of strain-rate effects have been concerned with fast strain-rates and have concentrated primarily on the plastic deformation region[26,27,28,29]. Typical stress-strain plots for 1.4307[27] and 1.4404[29] at room temperature are given in Figure C.3.6. More recent test results are shown in Figure C.3.7 and Figure C.3.8[30]. (The cyclic fluctuations in the 0 to 20% strain range in these latter two Figures are due to the dynamic response of the testing machine.) The Figures show that stainless steels have a strong strain rate dependency; strengths are increased (particularly in the region of the 0,2% proof strain) and the rupture strain reduced at higher strain rates. In the design of stainless steel blast walls, where the predominant loading is at a high strain rate, it is customary to apply a strain rate enhancement factor to the design strength in order to take advantage of the increase in strength at higher strain rates.



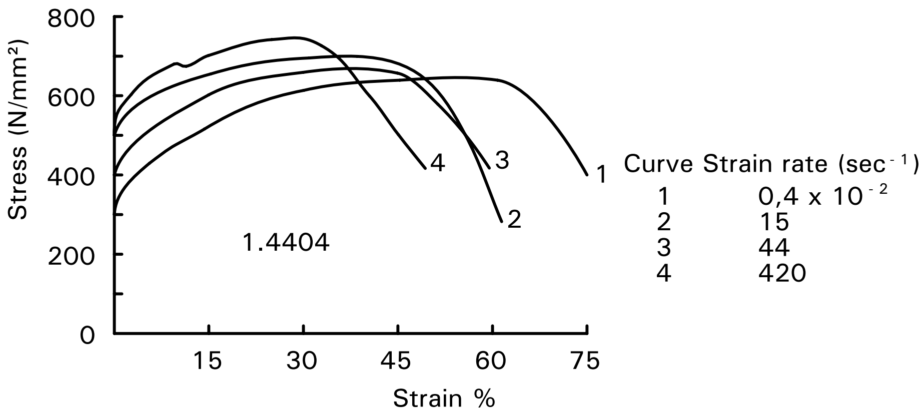


Figure C.3.6 *Strain rate effects on grades 1.4307 and 1.4404*

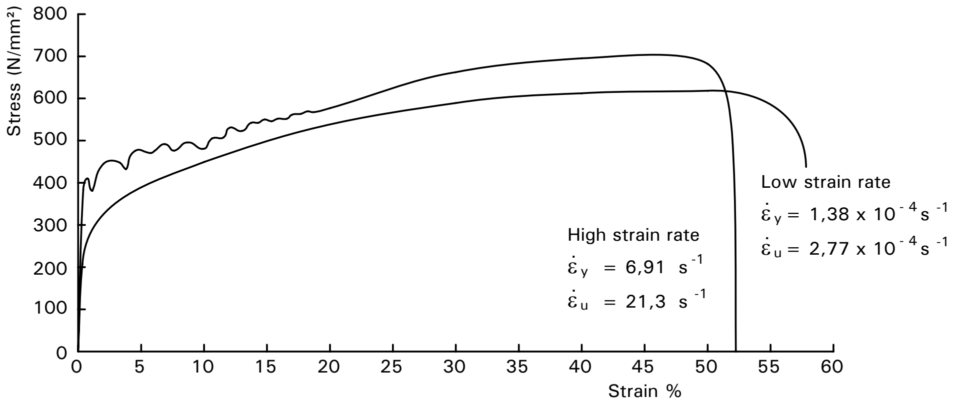


Figure C.3.7 *Strain rate effects on grade 1.4404*

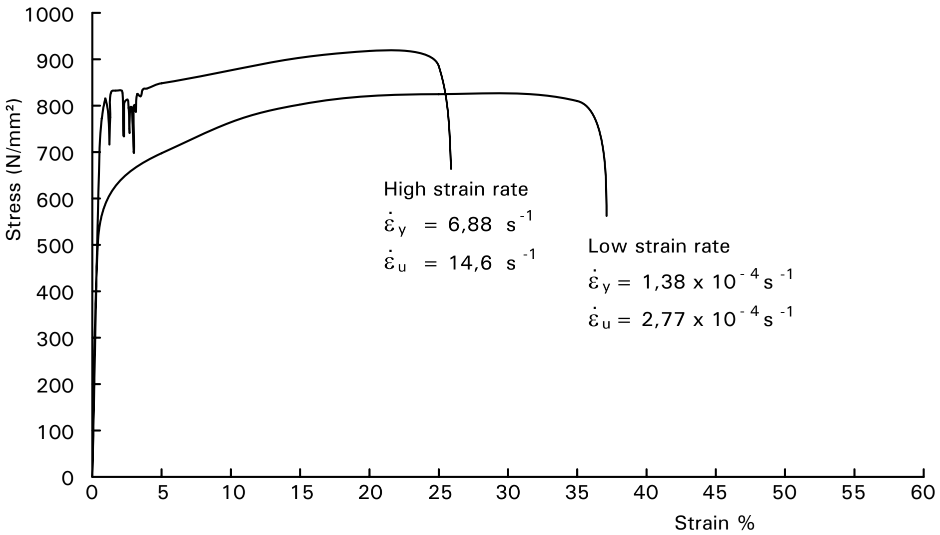


Figure C.3.8 *Strain rate effects on grade 1.4462*

Rather fewer investigations have examined the behaviour under slow strain-rates. The most well-known work is due to Krempl[31], in which annealed type 1.4301 stainless steel was tested at strain-rates of 10^{-3}, 10^{-5} and 10^{-8} per second (note the maximum equivalent strain-rate allowed in specifications is usually 1,5 x 10^{-4} per second). The decreases in the measured 0,2% proof stress due to a change in strain-rate from 10^{-3} to 10^{-5} per second and from 10^{-3} to 10^{-8} per second are about 15% and 30% respectively, i.e. averages per order change of strain-rate of 7,5% and 6% respectively.

In the tests carried out specifically for the First Edition of this Design Manual[14], constant stress-rates of 0,3 to 30 N/mm^2 per second were used. These correspond to strain-rates, in the elastic region, of 1,5 x 10^{-6} and 1,5 x 10^{-4} per second. Although an order change of stress rate gave, in isolated instances, a 6% change in the 0,2% proof stress, on average it was approximately 4%. This average figure applies equally to the three materials tested (1.4307, 1.4404 and duplex 1.4462) and would appear, on the evidence, to apply equally to the longitudinal and transverse directions and to tension or compression.

It should be noted that a constant strain-rate and a constant stress-rate are not equivalent past the proportional limit, even if they correspond to the same rate in the elastic region. A constant stress-rate will give ever increasing equivalent strain-rates as loading continues, since plastic straining does not contribute to stress. Thus constant stress rates generally will lead to higher measured proof stresses than constant strain-rates. This effect disappears at temperatures above about 200ºC, as can be seen in Figure C.3.9 for grade 1.4401 material.

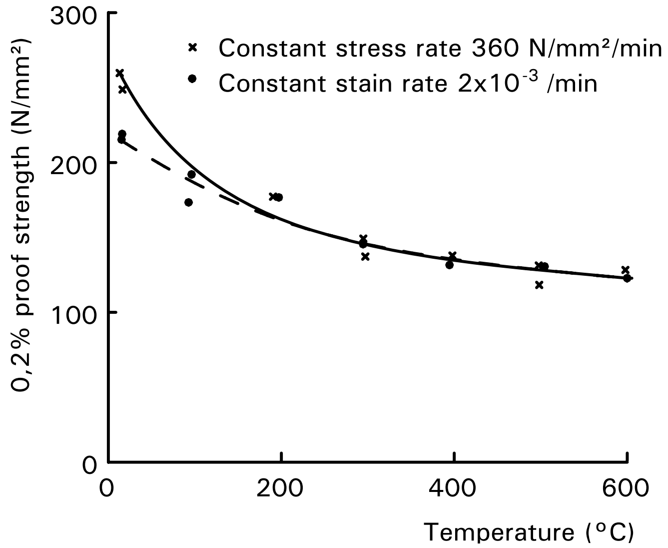


Figure C.3.9 *Effect of loading procedure on the 0,2% proof stress*

C.3.2.3 Typical values of properties

For the First Edition of the Design Manual, mill data was collected and analysed from several European stainless steel producers

C.3.2.4 Design values of properties

Flat products

Three options are offered for defining the design strength. Options (ii) and (iii) can only be used if the actual material to be used in the structure is identified and available at the time of design; however, these options will generally give the more economical use of material.

Figure 3.2 shows the non-symmetry and aniostropy of stainless steel grade 1.4318 cold worked to strength level C850; Reference 32 studies this in greater detail). For cold worked material in the longitudinal (rolling) direction, the strength in compression lies below the strength in tension. Material standards such as EN 10088 typically quote minimum specified values in the transverse tension direction. Therefore, when designing members where compression is a likely stress condition, it is necessary to factor down the quoted minimum specified 0,2% proof strength unless that strength is guaranteed in tension and compression, transverse and parallel to the rolling direction.

From the recent ECSC project[22], it was suggested that along the length of a tubular member, the compression strength f_{yLC} is about 85% of the strength in tension f_{yLT} and 78% of the strength in tension transverse to the axis of the tube f_{yTT}, i.e.

For C850 material (f_y = 530 N/mm^2): $f_{y\mathrm{LC}} = 0.78\, f_{y\mathrm{TT}}$

These figures were based on very few test data, but were in agreement with additional test data from the Finnish manufacturer of cold worked rectangular hollow sections, Stalatube.

The American design code addresses this issue of asymmetry by giving lower strengths for material stressed in longitudinal compression (even in the annealed condition), and higher strengths for material stressed in transverse compression (Table C.3.3). Note that the longitudinal compression strength reduces relative to the transverse tensile strength as the level of cold working increases. It specifies a greater reduction in f_{yLC} relative to f_{yTT} than European data suggests:

For f_y = 350 N/mm^2: $f_{y\text{LC}} = 0.84\, f_{y\text{TT}}$

For f_y = 500 N/mm^2: $f_{y\text{LC}} = 0.68\, f_{y\text{TT}}$

Table C.3.3 *Specified yield strengths (N/mm^2) of stainless steel in the American design code for grades 1.4301 and 1.4401*

Direction of stress	Annealed	1/16 hard	1/4 hard	1/2 hard
Longitudinal tension	206.9	310.3	517.1	758.5
Transverse tension	206.9	310.3	517.1	758.5
Transverse compression	206.9	310.3	620.7	827.6
Longitudinal compression	193.1	282.7	344.8	448.2

Fasteners

It is important that connections in steelwork are ductile at the Ultimate Limit State. For this reason it is traditional to have high factors of safety associated with fasteners. In EN 1993-1-1 the factor of safety is approximately 1,9 to 2,1, the effects of prying action being explicitly calculated.

The provisions in EN 1993-1-1 should give a satisfactory factor of safety against tension failure for stainless steel bolts, especially as stainless steel is more ductile than normal structural bolt materials. Therefore the resistance of fasteners should be based on the ultimate tensile strength of the material as in EN 1993-1-1.

C.3.3 Physical properties

Compared to carbon steels, the higher coefficients of thermal expansion for the austenitic steels (e.g. 1.4301 and 1.4401), and the lower thermal conductivities, give rise to greater welding distortions, see Section 10.4.4 in the Recommendations.

Cold working produces phase transformation (see C.3.2.2). These strain induced phases are magnetic and thus cold worked austenitic stainless steels generally have different magnetic properties from those in the annealed condition. However, unless the application is critical, moderate amounts of cold working may still provide adequate magnetic properties. Annealing has the effect of reversing the phase transformation and thus restoring the non-magnetic properties.

C.3.4 Effects of temperature

Other properties to be considered in elevated temperature applications include creep strength, rupture strength, scaling resistance, etc. Useful information on these and other properties may be found in References 5, 33 and 34.

Information for cryogenic applications may be found in References 5, 34 and 35.

C.3.5 Life cycle costing

Software for carrying out life cycle costing calculations, accompanied by a case study and supplementary guidance is available[36].

C.3.6 Selection of materials

C.3.6.1 Grades

Table 3.7 in the Recommendations is extracted from Reference 37, which also considers other types of stainless steel. It is based on long term exposure of stainless steel sheet samples at a variety of locations.

For environments other than atmospheric, it is advisable to seek the advice of a corrosion engineer or obtain information from stainless steel producers. Reference 38 gives some details of service experience obtained in the following industries:

- Oil and gas industry;
- Food and beverage industry;
- Pharmaceutical industry;
- Power industry;
- Pulp and paper industry;
- Automotive industry;
- Shipping and aerospace industry.

C.3.6.2 Availability of product forms

Table C.3.4 and Table C.3.5 give the standard and special finishes available, taken from EN 10088-2[7]. Note that the availability and cost of the finishes represented in Table C.3.5 may be considerably different from the ones in Table C.3.4; see Section 10.6 of the Recommendations. Further guidance on finishes is also available[39,40].

When investigating product availability, it may be prudent to check delivery times.

Table C.3.4 *Type of process route and surface finish for sheet, plate and strip: hot and cold rolled finishes*[1)]

Abbreviation in EN 10088-2 [2)]	Type of process route	Surface finish	Notes
1U	Hot rolled, not heat treated, not descaled	Covered with the rolling scale	Suitable for products which are to be further worked, e.g. strip for rerolling
1C	Hot rolled, heat treated, not descaled	Covered with the rolling scale	Suitable for parts which will be descaled or machined in subsequent production or for certain heat-resisting applications.
1E	Hot rolled, heat treated, mechanically descaled	Free of scale	The type of mechanical descaling, e.g. coarse grinding or shot blasting, depends on the steel grade and the product, and is left to the manufacturer's discretion, unless otherwise agreed.
1D	Hot rolled, heat treated, pickled	Free of scale	Usually standard for most steel types to ensure good corrosion resistance; also common finish for further processing. It is permissible for grinding marks to be present. Not as smooth as 2D or 2B.
2H	Work hardened	Bright	Cold worked to obtain higher strength level.
2C	Cold rolled, heat treated, not descaled	Smooth with scale from heat treatment	Suitable for parts which will be descaled or machined in subsequent production or for certain heat-resisting applications.
2E	Cold rolled, heat treated, mechanically descaled	Rough and dull	Usually applied to steels with a scale which is very resistant to pickling solutions. May be followed by pickling.
2D	Cold rolled, heat treated, pickled	Smooth	Finish for good ductility, but not as smooth as 2B or 2R.
2B	Cold rolled, heat treated, pickled, skin passed	Smoother than 2D	Most common finish for most steel types to ensure good corrosion resistance, smoothness and flatness. Also common finish for further processing. Skin passing may be by tension levelling.
2R	Cold rolled, bright annealed[3)]	Smooth, bright, reflective	Smoother and brighter than 2B. Also common finish for further processing.
2Q	Cold rolled, hardened and tempered, scale free	Free of scale	Either hardened and tempered in a protective atmosphere or descaled after heat treatment.

Notes:

1) Not all process routes and surface finishes are available for all steels

2) First digit, 1 = hot rolled, 2 = cold rolled

3) May be skin passed

Table C.3.5 *Type of process route and surface finish for sheet, plate and strip: special finishes*[1)]

Abbreviation in EN 10088-2 [2)]	Type of process route	Surface finish	Notes
1G or 2G	Ground	[4)]	Grade of grit or surface roughness can be specified. Unidirectional texture, not very reflective.
1J or 2J	Brushed or dull polished	Smoother than ground.[4)]	Grade of brush or polishing belt or surface roughness can be specified. Unidirectional texture, not very reflective. Typically specified for internal applications.
1K or 2K	Satin polish	[4)]	Additional specific requirements to a 'J' type finish, in order to achieve adequate corrosion resistance for marine and external architectural applications. Transverse $R_a < 0.5$ μm with clean cut surface finish. Typically specified for external applications.
1P or 2P	Bright polished	[4)]	Mechanical polishing. Process or surface roughness can be specified. Non-directional finish, reflective with high degree of image clarity.
2F	Cold rolled, heat treated, skin passed on roughened rolls	Uniform non-reflective matt surface	Heat treatment by bright annealing or by annealing and pickling.
1M	Patterned	Design to be agreed, second surface flat	Chequer plates used for floors
2M	Patterned	Design to be agreed, second surface flat	A fine texture finish mainly used for architectural applications
2W	Corrugated	Design to be agreed	Used to increase strength and/or for cosmetic effect.
2L	Coloured[3)]	Colour to be agreed	
1S or 2S	Surface coated[3)]		Coated with e.g. tin, aluminium

Notes:

1) Not all process routes and surface finishes are available for all steels

2) First digit, 1 = hot rolled, 2 = cold rolled

3) One surface only, unless specifically agreed at the time of enquiry and order

4) Within each finish description, the surface characteristics can vary, and more specific requirements may need to be agreed between manufacturer and purchaser (*e.g.* grade of grit or surface roughness)

C.3.7 Durability

C.3.7.1 Introduction

Although stainless steel will perform satisfactorily in the great majority of applications, there are potential difficulties with corrosion mechanisms in specific environments. It is the intention of Section 3.7 in the Recommendations to bring to the designer an awareness of these mechanisms and the possible pitfalls in the application of stainless steel, without being unduly alarmist. Good design will avoid potential problems.

C.3.7.2 Types of corrosion

The corrosion resistance of stainless steel arises from a passive, chromium-rich, oxide film that forms on the surface of the steel[41,42]. The film is strongly adherent, usually self-repairing, and generally highly resistant to chemical attack. If it is broken down and not repaired, corrosion will occur.

The presence of oxygen is essential to the corrosion resistance of a stainless steel. The corrosion resistance is at its maximum when the steel is boldly exposed and the surface is maintained free of deposits by a flowing bulk environment (e.g. rainwater). Covering a portion of the surface, for example by biofouling, painting, or installing a gasket, produces an oxygen-depleted region under the covered region, and a higher level of alloy content is required to prevent corrosion.

Molybdenum is used to increase the stability of the film and thus grades 1.4401 and 1.4404 exhibit greater corrosion resistance than grades 1.4301 and 1.4307. Duplex 1.4462 is even better in terms of corrosion resistance.

General (uniform) corrosion

Passivity exists under certain conditions for particular environments. When conditions are favourable for maintaining passivity, stainless steels exhibit extremely low corrosion rates. If passivity is destroyed under certain conditions that do not permit the restoration of the passive film (as may occur in strongly acid or alkaline environments), stainless steel will corrode, much like a carbon or low alloy steel.

The corrosion rate in chemical environments can be expressed as either mass loss per unit surface area per unit time (normally g/m^2h) or thickness loss per unit time (normally mm/year). Iso-corrosion curves are available[43] for particular corrosive media that show constant rates of corrosion as a function of, for example, temperature and concentration. It should be noted that these curves can be significantly affected by impurities or additives in the medium.

Abrasive corrosion

Abrasive corrosion could occur, for instance, in flowing water containing suspended particles such as in some rivers, coastal areas, etc.

Pitting corrosion

Pitting initiation is influenced by surface conditions, including the presence of deposits, and by temperature. For a particular grade of stainless steel and a given environment, tests show that pitting will not initiate below a certain ‘critical pitting temperature’ (CPT). This, however, is of limited use when considering chloride-induced attack, as the corrosivity of a particular concentration of chloride solution can be greatly affected by other chemical species. Also, very commonly, the chloride solution may be locally concentrated, such as occurs when evaporation takes place.

In short, for the types of environment for which this Design Manual was prepared, resistance to pitting is best characterised by service experience[44,45].

Crevice corrosion

A crevice will only present a corrosion hazard if it is wide enough to permit entry of a liquid and sufficiently narrow to maintain a stagnant zone. For these reasons crevice corrosion will usually only occur at openings a few tens of microns or less in width and rarely within gaps that are several millimetres wide. As with other types of corrosion, crevice corrosion cannot occur without a liquid corrodant; if the liquid is excluded from the crevice no trouble will occur.

It is therefore possible for some gaps, which may be defined as crevices, to be relatively safe but a precise decision is not really possible without experience of the situation involved and thus the general tendency is to recommend their elimination. It may be possible to seal crevices (see 3.7.3, *Design for corrosion control*).

As for pitting, a 'critical crevice temperature' similarly exists for this form of corrosion and which is specific to the geometry and nature of the crevice and the precise corrosion environment for each grade. Again, this can give a useful guide to preliminary alloy selection in chemical environments.

Intergranular corrosion (sensitisation)

The fact that the selected grades do not generally become sensitised is beneficial not only for intergranular corrosion but also for other forms of corrosion. This is because the low carbon content limits the amount of chromium that is precipitated out, leaving a relatively high amount in solution for imparting corrosion resistance.

Where service temperatures of more than 425°C are required, consideration should be given to the so-called stabilised grades. These grades, commonly designated 1.4541 and 1.4571, have additions of titanium which preferentially form carbide precipitates to chromium.

Bimetallic corrosion

Under certain circumstances, most metals can be vulnerable to this form of corrosion[46].

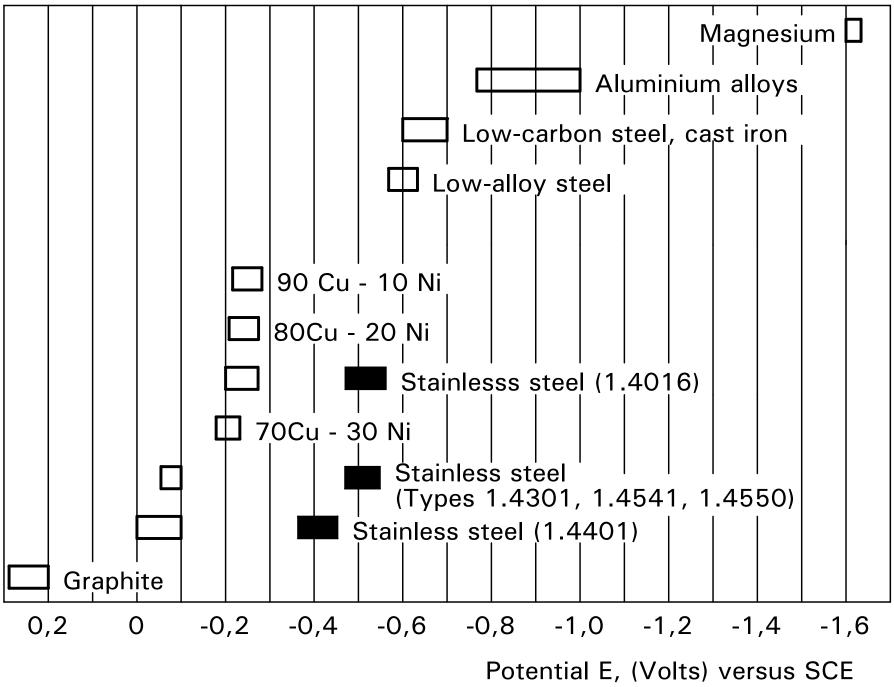
The severity of bimetallic corrosion depends on:

Potential difference

The greater the potential difference between the metals (or other materials), the higher is the rate of corrosion. Figure C.3.10 shows the potentials of various materials in seawater at 10°C to 25°C, flowing at 2,5 to 4m/s[47].

Electrolyte

Increased conductivity of the electrolyte will raise the corrosion rate. Brackish waters and seawaters are very conductive. Fresh water can also be very conductive depending on the level of contaminants; rain can absorb atmospheric pollutants and may become conductive. The period of exposure to the electrolyte, including the effectiveness of drainage and evaporation and the retention of moisture in crevices, is an important parameter.



The black bars indicate potential in low velocity or poorly aerated water and in shielded areas

Figure C.3.10 *Corrosion potentials of various materials in flowing seawater; potentials are measured against saturated calomel electrode (SCE)*

Area relationship

The role of area relationship is discussed in the Recommendations.

Stress corrosion cracking

It is difficult to predict when stress corrosion cracking (SCC) may occur but experience[48,49] would suggest that it should certainly be considered for marine and other environments contaminated by chloride ions, as these are known to promote SCC.

As for other forms of corrosion the period of wetness (including that due to condensation) can affect SCC, as does the concentration of the damaging species (e.g. chloride). It should be noted that SCC can be caused by solutions having initially low chloride concentrations, even as low as parts-per-million levels. This is because the solution may become concentrated due to evaporation.

Relatively high amounts of δ-ferrite are required to effectively block the paths of the cracks. Around 50% δ-ferrite content is the optimum amount[50]. This is approximately the amount of δ-ferrite present in duplex 1.4462 which as a result is much more resistant to SCC than the austenitic grades. Naturally, the morphology and the distribution of the δ-ferrite, particularly at and within weldments, must be carefully controlled to achieve such benefits. This calls for adequate welding procedures to be utilised.

Detailed guidance on the use of stainless steel in swimming pool buildings, taking due regard of the risk of SCC, was published in 1995[51], however, the recommendations in this reference on grade selection are now superseded. A guidance note on SCC of stainless steels in swimming pool buildings, including preventative measures and inspection procedures, has also been published[52].

Up-to-date guidance on grade selection is given in Reference 53 which aligns with clause A.4.1(10) in the Informative Annex A to EN 1993-1-4.

C.3.7.3 Corrosion in selected environments

General guidance is given in this section of the Recommendations and no further comment is given here.

C.3.7.4 Design for corrosion control

Many of the recommendations given in this section are simply a matter of good engineering practice and also apply to the design of carbon steel structures. However, they assume more importance with stainless steel structures.

Fabrication processes play an important part in corrosion resistance and reference should also be made to Section 10 in the Recommendations.

C.4 PROPERTIES OF SECTIONS

C.4.1 General

Section 4 of the Recommendations is concerned with the local behaviour of members; overall buckling is addressed in Section 5. For a member not subject to overall buckling, e.g. a stub column, the resistance (strength capacity) is solely dictated by local behaviour and therefore the provisions of Section 4 are sufficient for its determination.

The local capacity of a member, i.e. the cross-sectional resistance, is dependent on the resistances of the constituent elements that make up the cross-section. Elements, and hence the cross-section, may be affected by certain structural phenomena, such as local buckling and shear lag, which reduce their effectiveness to carry load. As in the case of carbon steel rules, these phenomena are catered for in the Recommendations by the use of effective widths.

In deriving the First Edition of the Design Manual in Section 4, carbon steel codes[2,54,55], stainless steel codes[23] and experimental data for stainless steel members have been consulted. When revising the Recommendations for the Second Edition, further test data were available, generated in the *Development of the use of stainless steel in construction project*[56]. In addition, the ENVs for cold formed carbon steel, fire resistant design, stainless steel and plated structures were also used[57,58,59,60]. When revising the Recommendations for the Third Edition, new test data were available from the *Structural design of cold worked austenitic stainless steel project*[22] as well as the following parts of Eurocode 3: EN 1993-1-1[2], -2[61], -3[62], -4[3], -5[63], -8[64], -9[65] and -12[66].

C.4.2 Maximum width-to-thickness ratios

Limiting width-to-thickness ratios are provided for various types of elements. Limits are placed not so much that thinner sheets cannot be used but because the rules may become inaccurate. The ratios have been set as the smaller of the limiting values given in EN 1993-1-3 for cold formed, thin gauge carbon steel and the American cold formed stainless steel specification[23].

It can be argued that at the low stresses associated with the high slendernesses, carbon and stainless steel elements should behave very similarly and thus justify the use of the greater ratios of EN 1993-1-3 for all stainless steel elements. It is, however, considered prudent to use the values in Reference 23, where they are more limiting, due to the paucity of data relating to stainless steel and the fact that experience has already been gained with these values in a previous version of the American provisions.

The note concerning b/t ratios and visual distortion is based on Reference 23 and the b/t values are derived from the critical stress in the flange elements.

C.4.3 Classification of cross-sections

C.4.3.1 General

The classification of cross-sections according to their ability to resist local buckling and to sustain load with deformation has proved a useful concept for the design of carbon steel members and indeed for members of other metals (e.g. Ref. 67). Classification is usually defined in terms of a cross-section's moment capacity, i.e. whether it can reach the plastic moment (with and without rotation capacity), the elastic moment, or a lower value due to the onset of buckling.

As the definition of yield strength of non-linear materials is rather arbitrary, so are the definitions of yield and plastic moments for members composed of such materials. The obvious definitions to apply are the elastic and plastic section moduli multiplied by a proof stress, conventionally defined as the stress giving a 0,2% permanent strain. This is discussed further in C.4.7.

The cross-section's moment capacity is a function of the behaviour of the elements that constitute the cross-section.

Table 4.2 gives limiting width-to-thickness ratios for the classification of elements according to their type. The limiting ratios for Class 3 elements given in the table are derived from experimental stainless steel data whereas the limiting ratios for Classes 1 and 2 have been derived by making reference to other data and applying engineering argument.

In Table 4.2, the Class 3 limiting ratios for elements under pure compression are found when the reduction factor ρ in Section 4.4.1 is set equal to unity. Thus, for an internal element such as a web (for which the buckling factor $k_\sigma = 4$):

$$\rho = \frac{0{,}772}{\bar{\lambda}_{\mathrm{p}}} - \frac{0{,}125}{\bar{\lambda}_{\mathrm{p}}^{2}} = 1 \text{ and } \bar{\lambda}_p = \frac{d/t_w}{28{,}4\varepsilon\sqrt{k_\sigma}} = \frac{d/t_w}{56{,}8\varepsilon}$$

which solves to give $d/t_{\mathrm{w}} = 30{,}7\ \varepsilon$

The Class 3 limiting ratios for outstand elements under compression are similarly derived. The Class 3 limiting ratios for elements in bending, or bending and compression, are inferred from the pure compression values by using the buckling factor k_σ. For example, for the web element considered above in pure bending, $k_\sigma = 23{,}9$ and therefore the limiting ratio is calculated as:

$$d/t_{\mathrm{w}} = 30{,}7\varepsilon \times \sqrt{23{,}9/4} = 75{,}0\ \varepsilon$$

(A minor adjustment has been made in Table 4.2, in which the value is shown as 74,8, to remove inconsistencies arising from rounding errors in the factors given for combined bending and compression.)

The use of the buckling factor in the above manner, for deriving limiting width-to-thickness ratios for elements subject to a degree of bending, removes anomalies present in carbon steel codes (e.g. Refs. 2 and 54). These relate to the existence of vertical cut-offs in the design curve of the reduction factor, ρ for bending elements in the carbon steel codes. In effect, the limiting ratios are increased in the carbon steel codes when bending is present. A similar increase may, in fact, also be applicable to stainless steel elements in bending, but there are no available data to support or to quantify this.

There are insufficient data to establish experimentally the Class 1 and Class 2 limiting ratios for stainless steel. However, numerical and experimental studies[68,69,70] on element load/end-shortening behaviour confirm that strain hardening materials exhibit longer plateaus and less steep unloading characteristics than non-hardening materials such as carbon steel.

Thus, if a carbon steel element may be classified as a Class 1 element, then a stainless steel element of the same slenderness will have at least as great a deformation capacity and likewise be classified as Class 1. It may be noted that with lower Class 3 limits, but with the same Class 1 limits, a smaller range between Class 1 and 3 exists for stainless steel than for carbon steel. There even exists the possibility that Classes 1 and 2 could collapse to a single class for stainless steel, though this potential simplification is left for future research.

In the absence of suitable data, a prudent approach has been taken in defining the Class 1 and 2 limits for stainless steel. Starting with outstand elements, the Class 1 limits for compression are the same as given for carbon steel in EN-1993-1-1. The Class 2 limits are set in the same proportions between the Class 1 and Class 3 limits that apply to carbon steel in EN 1993-1-1. For internal elements in compression, the Class 1 limits for carbon steel are already higher than those for Class 3 stainless steel elements. This is evidence for the collapse of classes for strain hardening materials referred to above. For these elements, therefore, the Class 1 and Class 2 limits for stainless steel were derived using the same proportions pertaining to outstand elements in compression.

The Class 1 and 2 limits in bending were established from the compression limits by applying the same factors that relate the carbon steel limits in EN 1993-1-1 to each other. Finally, for Class 1 and 2 stainless steel elements under combined bending and compression, suitable interaction formulae were established having the same form (linear, reciprocal functions, etc) as used in EN 1993-1-1 for carbon steel.

Since there is no sharply defined yield point, placing cross-sections into discrete behavioural classes is less appropriate for stainless steel than it is for carbon steel. Gardner[17] has proposed a continuous method of cross-section classification and member design: using a more appropriate material model, member strengths are assessed using a local buckling strength derived from the deformation capacity of the cross-section. It can be viewed as a continuous method of section classification and member design.

C.4.3.2 Classification limits for parts of cross-sections

The classification limits in the Design Manual have been verified against all available experimental results to ensure safe design. These include both stub column tests, by means of which the limits for Class 3 parts (internal, outstand, CHS, angles) under pure compression are verified, and in-plane bending tests, by means of which the actual limits for elastic (Class 3) and plastic (Class 2) moment resistance are compared to the ones in the present guidance. For both stub column and in-plane bending tests, the ultimate resistance normalised by the relevant theoretical resistance is plotted against the slenderness of the most slender element of the cross-section. The relevant class limit is also depicted in the graphs.

The aforementioned experiments and the accompanying graphs are described below:

Internal elements (stub column tests)

Seven sources of data exist for internal elements under pure compression (see Figure C.4.1). These include SHS, RHS, lipped channel sections and H-shaped sections.

Two 80×80×3 SHS stub column tests were reported by Rasmussen and Hancock[102].

Kuwamura[71] tested twelve SHS, sixteen H-shaped sections and eight lipped channel sections in 3 mm nominal thickness and grades 1.4301 and 1.4318 material. Four more tests on lipped channel section stub columns in 1 mm nominal thickness and grade 1.4301 were also reported. The SHS were cold-formed and laser welded ranging from 50×50 to 200×200, the H-shaped sections were fabricated by laser or TIG welding of individual plate elements ranging from 50×50 to 200×150 and the lipped channel sections were press-braked ranging from 100×50×20 to 200×75×25.

Gardner and Nethercot[17, 72] reported seventeen SHS (80×80 to 150×150) and sixteen RHS (60×40 to 150×100) stub column tests in 2-8 mm nominal thickness and grade 1.4301 material. Talja and Salmi[73] reported three stub column tests (60×60×5, 150×100×3 and 150×100×6) in grade 1.4301.

Young and Liu[74] tested four 70×70 SHS in 2 and 5 mm thickness and eight roll-formed RHS (120×40 and 120×80) in 2-6 mm thickness. All specimens were in grade 1.4301 material. Young and Lui[75] reported six SHS stub column tests in 1.5-6 mm thickness and 1.4301 and duplex material grades. The sections tested range from 40×40×2 to 150×150×6. Two RHS (140×80×3 and 160×80×3) in duplex and one 200×110×4 in grade 1.4301 were also reported.

Recent test results reported by Gardner, Talja and Baddoo[76] include four SHS (80×80 and 100×100) and four RHS (120×80 and 140×60) stub columns in 3 mm thickness and grade 1.4318 (in either annealed or cold-worked condition).

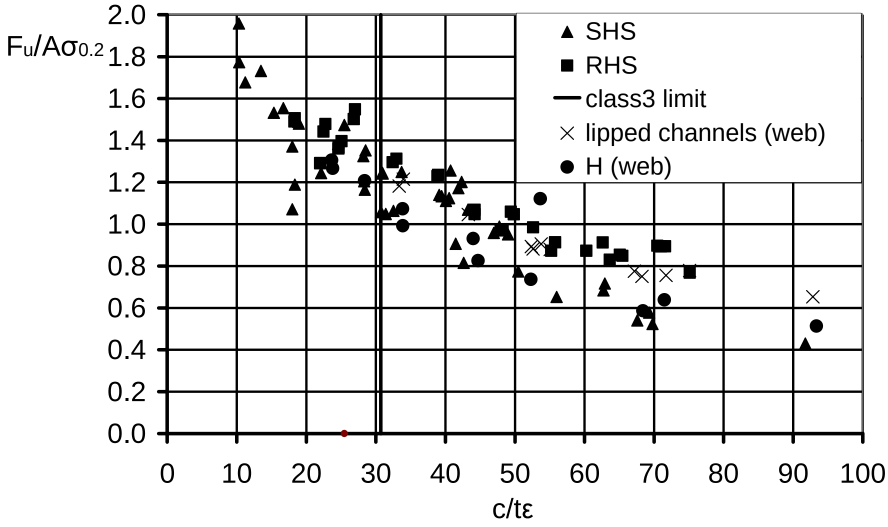


Figure C.4.1 *Experimental resistance over squash load vs. web width to thickness ratio*

Outstand elements (stub column tests)

The relevant stub column test data plotted in Figure C.4.2 include eight of the total sixteen H-shaped section tests conducted by Kuwamura[71] (described above) and eleven tests on channel sections in 3 mm thickness and grades 1.4301 and 1.4318 material. The sections range from 50×25×3 to 150×50×3 as reported in reference 70[71].

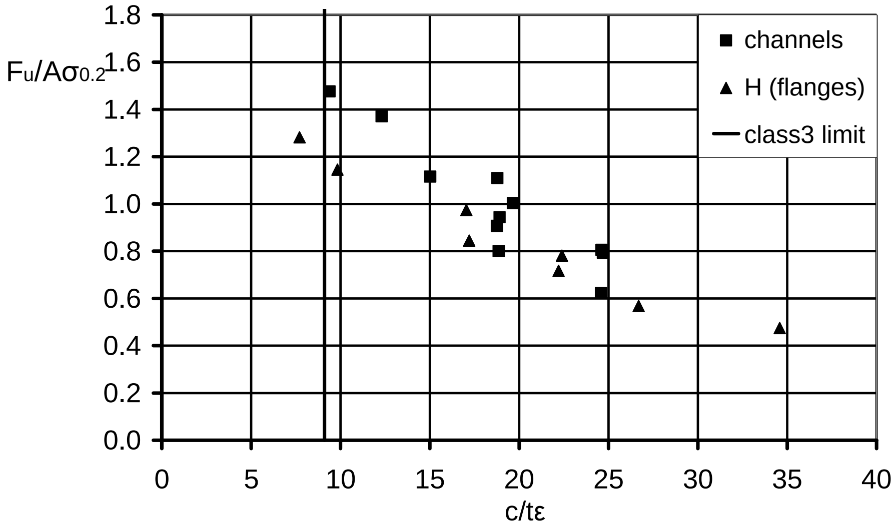


Figure C.4.2 *Experimental resistance over squash load vs. flange width to thickness ratio*

Angles (stub column tests)

Twelve angle specimens in grades 1.4301 and 1.4318 ranging from 25×25×3 to 60×60×3 were tested by Kuwamura[71]. Since the class 3 limit for equal angles in pure compression is stricter than the respective limit for outstand elements the relevant angle stub column results are depicted separately in Figure C.4.3.

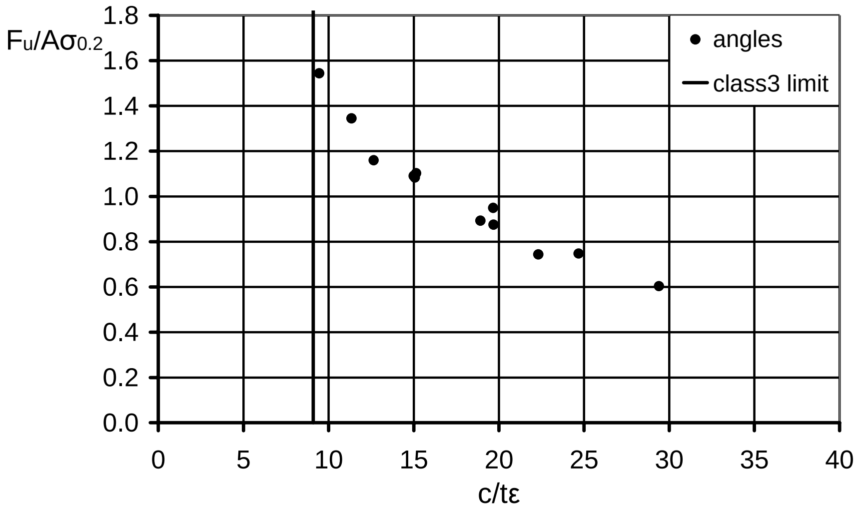


Figure C.4.3 *Experimental resistance over squash load vs. angle leg width to thickness ratio*

CHS (stub column tests)

Four researchers have reported tests on CHS stub columns. Rasmussen and Hancock[102] tested two CHS 101.6×2.85 stub columns in grade 1.4301. Kawamura[71] tested ten CHS specimens in grades 1.4301 and 1.4318 ranging form 49×1.5 to 166×1.5. Young and Hartono[77] tested four CHS stub columns in 1.4301 material ranging from 89×2.78 to 322.8×4.32. Gardner[17] reported four CHS tests in 1.5 mm thickness and grade 1.4301. The diameters examined were 103 and 153 mm. The results are depicted in Figure C.4.4.

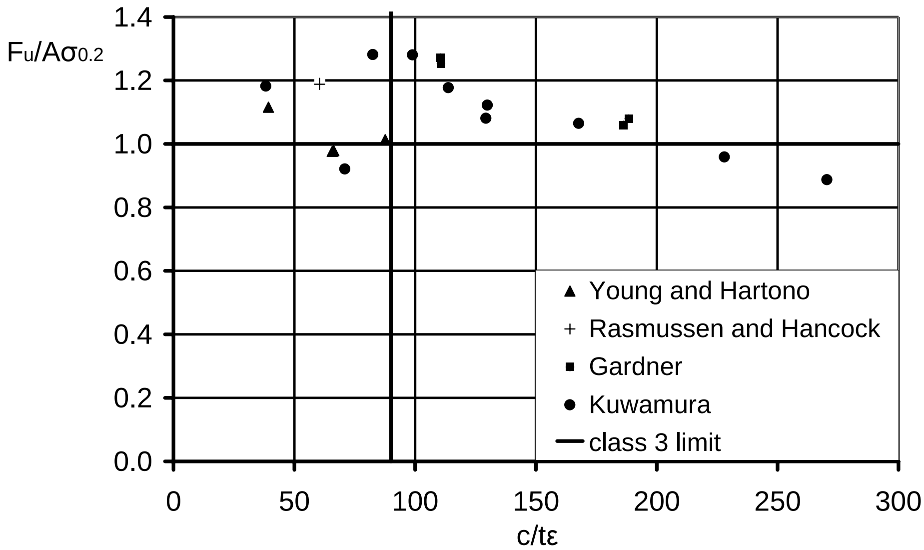


Figure C.4.4 *Experimental resistance over squash load vs. CHS diameter to thickness ratio*

CHS (bending tests)

Chryssanthopoulos and Kiymaz[78] reported eight bending tests on stainless steel CHS, four of which were grade 1.4301 and the remaining four were duplex grade. The specimens ranged from 103×1.5 to 219.1×3.76 and failed predominantly by local buckling or combined yielding and buckling, with the exception of the 219.1×2.5 specimen which failed underneath the loading collars as a result of bearing. Talja[91] conducted three CHS bending tests. The CHS 140×4 was grade 1.4541, whilst the CHS 140×3 and CHS 140×2 were 1.4435. Rasmussen and Hancock[102] reported one CHS 101.6×2.85 test in grade 1.4301. All specimens were subjected to 4 point-bending. The reported failure moments normalised by the plastic moment resistance and elastic moment resistance versus the diameter to thickness ratio are depicted in Figure C.4.5 and Figure C.4.6 respectively.

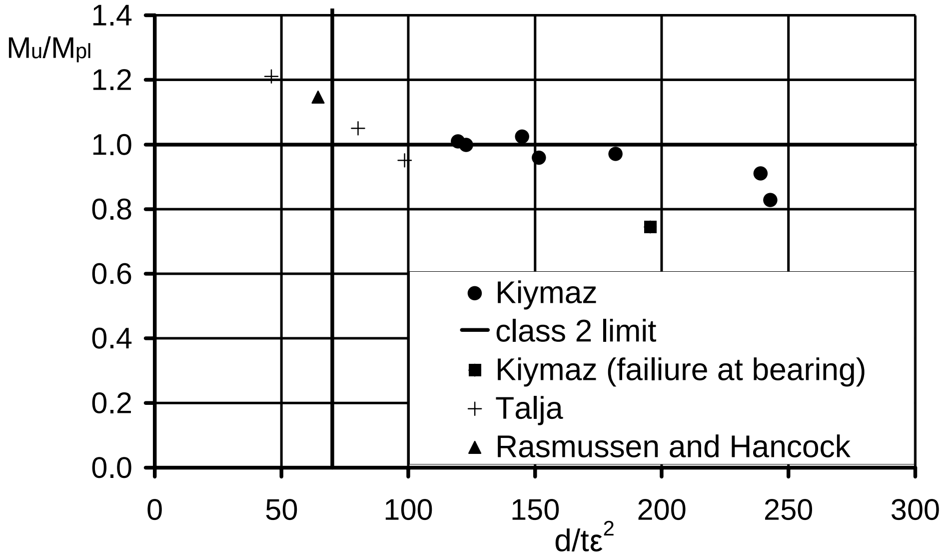


Figure C.4.5 *Experimental moment resistance over plastic moment resistance vs. CHS diameter to thickness ratio*

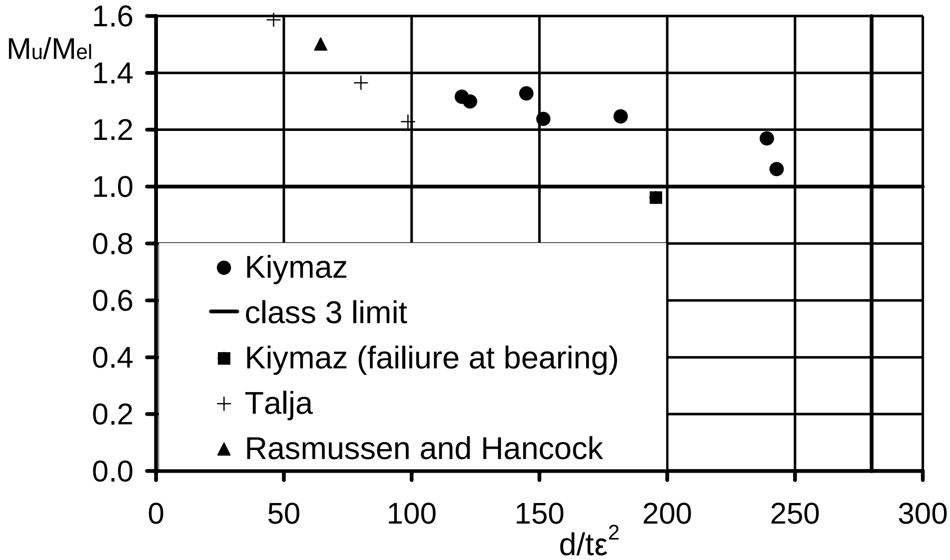


Figure C.4.6 *Experimental moment resistance over elastic moment resistance vs. CHS diameter to thickness ratio*

Internal elements (bending tests)

Six series of tests on beams comprising internal elements exist, including both SHS and RHS. Real[113] reported two SHS 80×80×3 and two RHS 120×80×4 simply supported bending tests in grade 1.4301. Three SHS 60×60×5, three RHS 150x100x3 and three RHS 150×100×6 in grade 1.4301 bending tests were reported by Talja and Salmi.[73] Gardner[17] reported five SHS (80×80 to 100×100) and four RHS (60×40 to 100×50) in-plane bending tests in 2-8 mm nominal thickness and grade 1.4301 material. Zhou and Young[79] reported eight SHS bending tests (from 40×40 to 150×150) in 1.5-6 mm thickness and seven RHS bending tests (100×50×2 to 200x110x4). All specimens were in 1.4301 and duplex grades. Gardner, Talja and Baddoo[76] tested two SHS 100×100×3 and four RHS beams (120×80×3 and 140×60×3) in grade 1.4318 (both annealed and cold-worked condition). One SHS 80×80×3 beam test reported by Rasmussen and Hancock[102] is also included in Figure C.4.7 and Figure C.4.8, the first depicting test moment normalised by plastic moment and the second by the elastic moment versus flange width to thickness ratio.

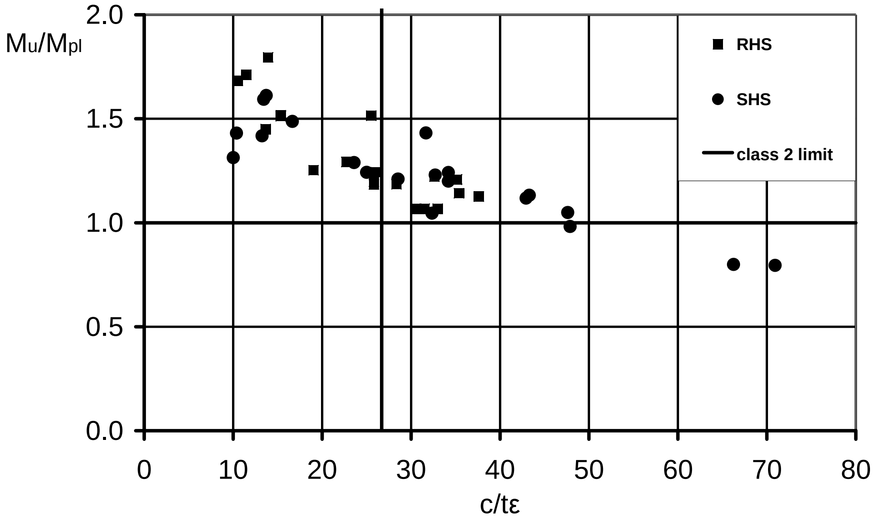


Figure C.4.7 *Experimental moment resistance over plastic moment resistance vs. flange width to thickness ratio*

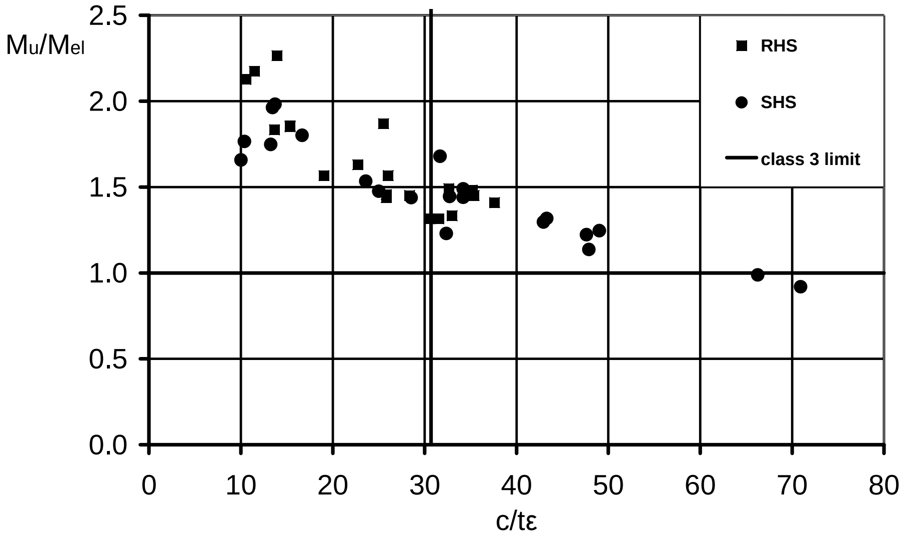


Figure C.4.8 *Experimental moment resistance over elastic moment resistance vs. flange width to thickness ratio*

Outstand elements (bending tests)

Two test series comprising a total of six I-section in-plane bending tests have been reported. The specimens were subjected to four-point bending and were short enough not to be susceptible to lateral torsional buckling. Talja[91, 80] conducted experiments on three I-sections (160×80, 160×160 and 320×160) with 10 mm flange and 6 mm web thickness in grade 1.4301 and one 160×160 with 10 mm flange and 7 mm web thickness in grade 1.4462. Real[113] reported two tests on I 100×100 beams in 8 mm thickness. The experimental ultimate moments normalised by the plastic moment are plotted against the flange width to thickness ratio in Figure C.4.9. The Class 2 and Class 3 limits for outstand elements are also depicted.

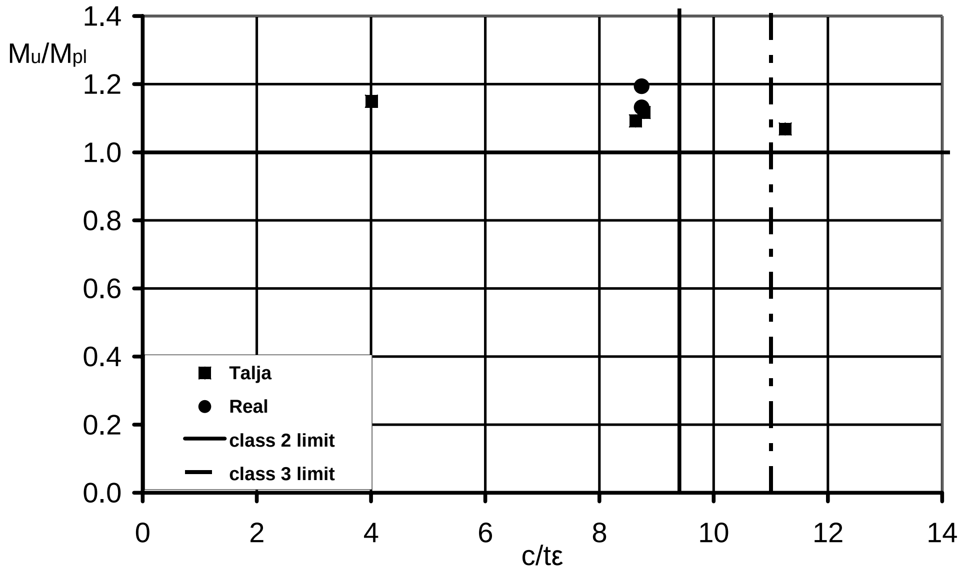


Figure C.4.9 *Experimental moment resistance over plastic moment resistance vs. CHS diameter to thickness ratio*

As shown in Figure C.4.1 to Figure C.4.9, the design rules for cross-sectional classification are safe for the vast majority of the reported experimental results. All of the stub column sections consisting of flat parts classified as Class 3 or above easily surpass the squash load, as did some sections classified as Class 4. This is largely due to the effect of the cold-worked corners which have a greater proof stress than the flat parts of the cross-sections and to the effect of strain-hardening. Both enhanced corner properties and strain-hardening are not explicitly accounted for in the design procedure. Ashraf, Gardner and Nethercot[24] proposed simplified formulae to account for the enhanced corner properties. Only two CHS stub column classified as Class 1-3 did not reach the theoretical squash load. The absence of corners in these cross-sections partly explains the moderate underestimations (and the two specimens failing prematurely) in the cross-sectional resistance, compared to the rest of the stub columns (with corner properties), for which the guidance is more conservative.

The in-plane bending tests demonstrate the significant moment resistance of stainless steel beams due to strain hardening. All of the specimens performed better than expected. Even some Class 4 specimens surpassed the plastic moment resistance. Again the specimens including corner regions (SHS and RHS sections) displayed an enhanced resistance, sometimes surpassing the plastic moment by more than 50%. All of the SHS and RHS classified as Class 2 surpassed the M_{pl} by at least 20%. The I-sections and CHS performance is closer to the predicted one.

C.4.4 Effective widths

C.4.4.1 Effective widths of elements in Class 4 cross-sections

The use of effective widths and effective cross-sections is well established for the structural design of Class 4 cross-sections. The concept is illustrated in Figure C.4.10 for an internal element under pure compression. In general, rules are required for calculating both the magnitude of the effective width as a function of element slenderness and stress distribution, and on how the effective width is distributed over the element. For the simple case in Figure C.4.10, the effective width is distributed as two equal zones, located at each unloaded edge of the element. Tables 4.3 and 4.4 give distribution rules for other cases and are the same as those used in EN 1993-1-5.

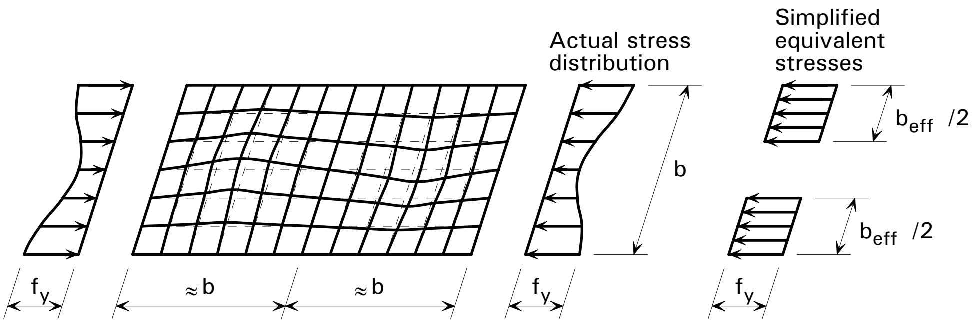


Figure C.4.10 *The effective width concept*

The effective width is normally found by applying a reduction factor, ρ, to the full width. An examination of the reduction factor given by Winter[81] for carbon steels and the American stainless steel code[23] has found it unsatisfactory for use with stainless steel. Rather, three separate expressions have been derived for various types of elements (cold formed or welded; internal; or outstand) by fitting characteristic curves to experimental data.

The curves are expressed in the form $\rho = \dfrac{a}{\bar{\lambda}_p} - \dfrac{b}{\bar{\lambda}_p^2}$ where a and b are constants and $\bar{\lambda}_p$ is a non-dimensional plate slenderness in order to resemble the corresponding expression in EN 1993-1-5.

The $\bar{\lambda}_p$ parameter has been proven numerically, as well as experimentally, to be suitable for (non strain hardening) carbon steel elements. It is not strictly accurate for strain hardening materials where the 'yield' strength is given in terms of an offset proof strength (as used throughout this Design Manual); rather a secant proof strength should be used[69]. However, it has been shown[82] that the offset proof strength gives sufficiently accurate results for design purposes, even for materials having a wide range of E values and yield strengths. In particular, Reference 83 describes one series of tests on magnesium, aluminium and stainless steel alloys with 0,2% proof strengths ranging from 184 to 1340 N/mm^2; the results are closely banded with $\bar{\lambda}_p$ based on the 0,2% proof strength.

The recommended curves, and their experimental basis, are described below:

Cold formed elements - Internal elements

Two sources of data exist for cold formed internal elements. Johnson and Winter[84] tested ten flexural hat members in grade 1.4301 material. Only four tests were reported in sufficient detail to allow the effective widths to be assessed. The sheet thicknesses used for these four tests were 0,78 and 1,25 mm. In Figure C.4.11 only the effective widths at the maximum applied loads are shown; these are not necessarily the ultimate loads. Nine internal element tests were carried out by Wang and Winter[85] of which seven tests were for members in flexure and two for members in compression. Beam materials included grade 1.4310 (formerly known as grade 301) (½ hard) in thicknesses 0,83 to 1,6 mm and grade 1.4301 of thickness 0,8 mm. Column materials were grade 1.4310 (½ hard) in thicknesses 8,2 mm and 15,7 mm. It may be noted that the 1.4310 (½ hard) grade had pronounced anisotropy. The results shown in Figure C.4.11 include sub-ultimate values found by substituting the yield strength in $\overline{\lambda}_p$ by the measured edge stress. Superimposed on the experimental data are the carbon steel curve from EN 1993-1-5 and the recommended curve for stainless steel given by Equation 4.1a in the Recommendations. The inclusion of the elastic data shows that the recommended curve is valid for subcritical stresses.

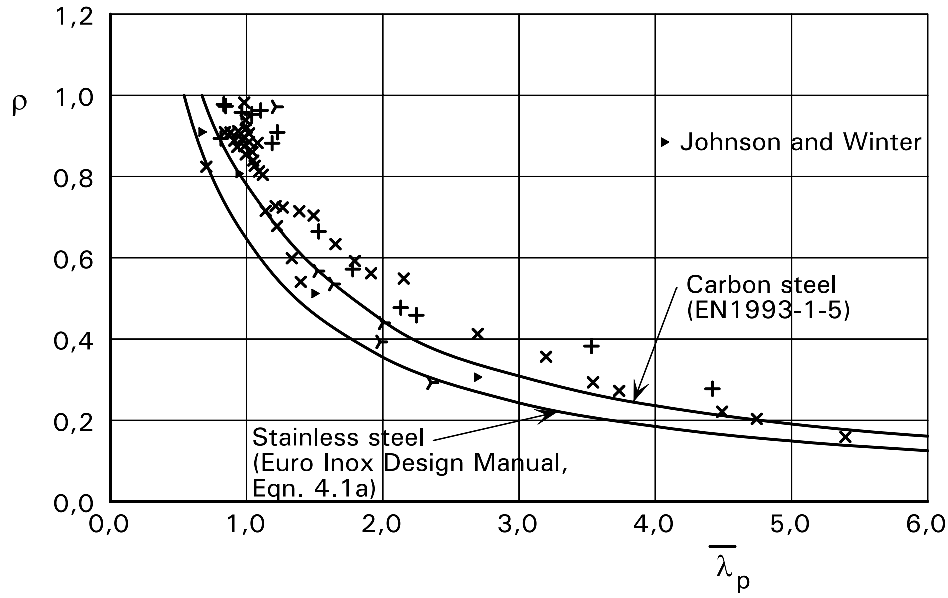


Figure C.4.11 *Reduction factor versus plate slenderness for cold formed internal elements*

Cold formed elements - Outstand elements

Johnson and Winter[86] tested sixteen columns, each comprising two channels, glued back-to-back, brake pressed from 1.4301 material nominally 0,9mm thick. Wang and Winter[85] carried out four tests on similar columns but in nominally 0,83mm thick 1.4310 (½ hard) material. The results, again including sub-ultimate values, are shown in Figure C.4.12. A design curve for stainless steel lying very close to the EN 1993-1-5 curve for carbon steel is recommended.

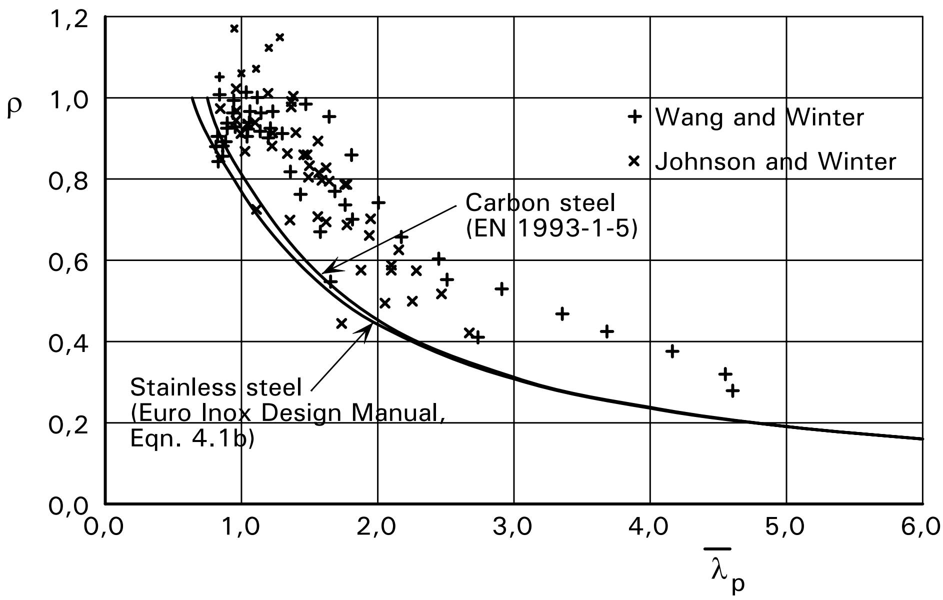


Figure C.4.12 *Reduction factor versus plate slenderness for cold formed outstand elements*

Welded elements

Only one series of tests is known in which local buckling of welded stainless steel elements is considered[87]. Twenty four stub columns, with various permutations of flange and web slendernesses, were fabricated in 1.4301 type materials of 2 and 3 mm thicknesses. It is not possible to evaluate how the load is shared between the flanges and web in any one test but it may be assumed that it is in the same ratio that is calculated from a design line, say the EN 1993-1-5 curve for carbon steel. For an individual test this does not yield any further useful information but when several results are processed, involving specimens of various combinations of flange and web slendernesses, a pattern emerges. It is particularly useful when the average value of the inferred reduction factors pertaining to each web or flange slenderness is considered. The results obtained with this procedure, more fully detailed in Reference 88, are shown in Figure C.4.13.

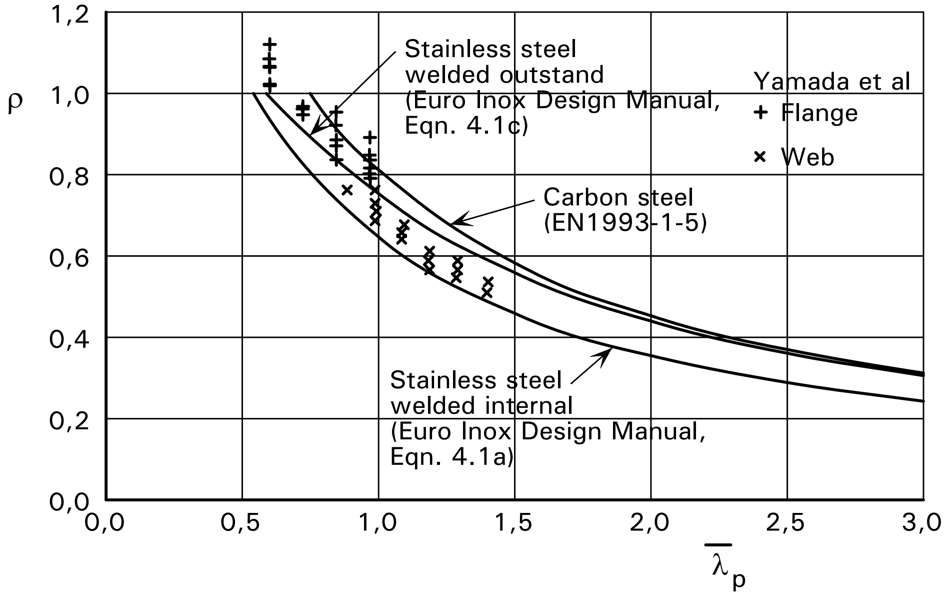


Figure C.4.13 *Reduction factor versus plate slenderness for welded elements*

Superimposed on the figure are: (a) the EN 1993-1-5 design curve for carbon steel elements, (b) the recommended stainless steel internal element curve discussed above for cold formed elements, which is to be compared with the web data, and (c) the recommended curve for welded stainless steel outstands, which is to be compared with the flange data. It is seen that the recommended internal element curve appears satisfactory for both cold formed and welded elements. However, the data supports that cold formed and welded outstands should be treated differently. This is partially recognised for carbon steel outstands where different Class 3 limits are given in EN 1993-1-1, i.e. different vertical cut-off lines are used, though only one design curve is applied.

The results from the stub column tests described in C.4.3.2 are utilized to verify that the design curves for the effective width of Class 4 elements are safe. All stub columns with one flat part (or symmetric flat parts) classified as Class 4, are used to derive the actual width reduction factor for the Class 4 element which is plotted against the relevant plate slenderness. For consistency with the guidance in the Design Manual, the enhanced strength of the corner regions and the strain-hardening behaviour of the material are ignored, i.e. the corners are assumed to have the same proof stress as the flat plate elements, the fully effective parts of the cross-sections are assumed to be stressed up to the proof stress and the Class 4 parts are assumed to carry more load than they really do. These assumptions are not conservative for stocky elements, but will be more accurate for the slender sections, since the slender elements buckle below the 0.2% proof stress. Since the corner properties and the strain-hardening material behaviour are not explicitly accounted for in design, the described approach is considered adequate for slender plate elements. The effective width factor ρ is plotted against element slenderness $\overline{\lambda}_p$ in Figure C.4.14 and Figure C.4.15 for internal and outstand parts respectively.

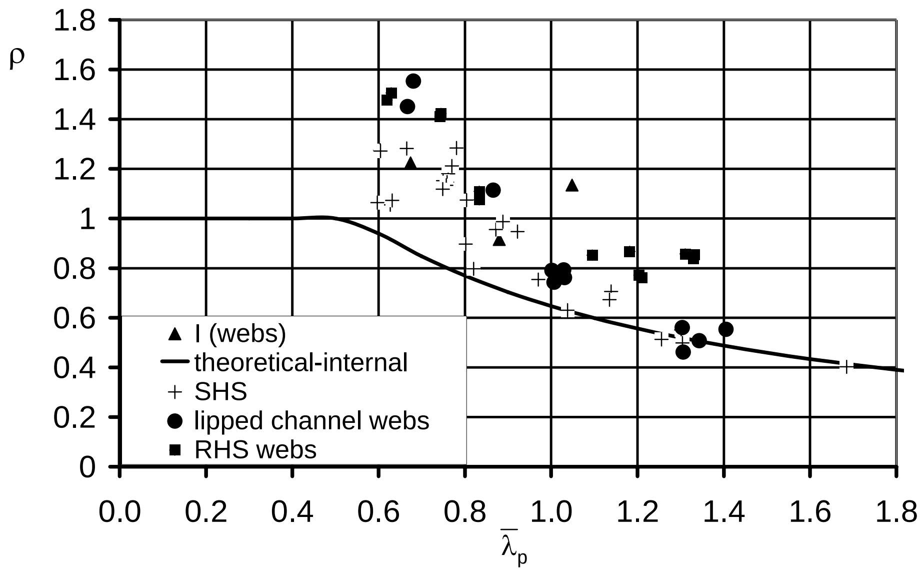


Figure C.4.14 *Reduction factor versus non-dimensional plate slenderness for internal elements*

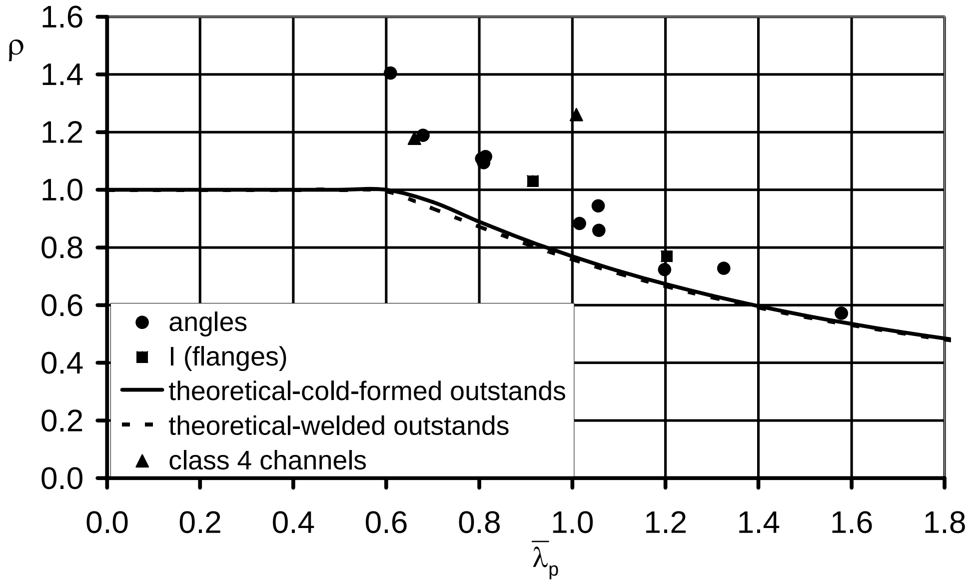


Figure C.4.15 *Reduction factor versus non-dimensional plate slenderness for outstand elements*

The design curves in the Design Manual are safe for the vast majority of the experimental results considered.

C.4.4.2 Effects of shear lag

Shear lag is a phenomenon that has been widely studied in the context of aeronautical, ship and bridge structures[89]. Rather fewer studies have examined the problem of interaction effects between shear lag and local buckling. Although no work is known which specifically looks at the effects of strain hardening on shear lag behaviour, studies on elements under combined shear and compression[68,90] would suggest that no significant difference exists between hardening and non-hardening materials.

The guidance in EN 1993-1-5 is considered applicable to stainless steel.

C.4.4.3 Flange curling

When a beam is subject to bending, the out-of-plane stress components arising from flange curvature deflect those parts of the flange remote from the web towards the neutral axis. This gives rise to flange curling as illustrated in Figure C.4.16. It only becomes significant for unusually wide thin flanges or where the appearance of the section is important.

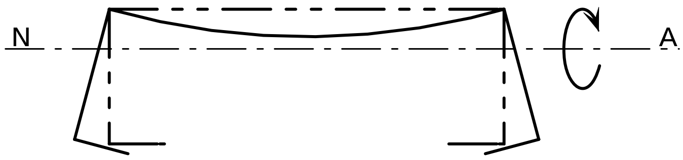


Figure C.4.16 *Deformations in flange curling*

The guidance in EN 1993-1-3 is considered applicable to stainless steel.

C.4.5 Stiffened elements

Guidance is given to ensure that edge stiffeners are adequate if a flange is to be treated as an internal element, see Figure C.4.17.

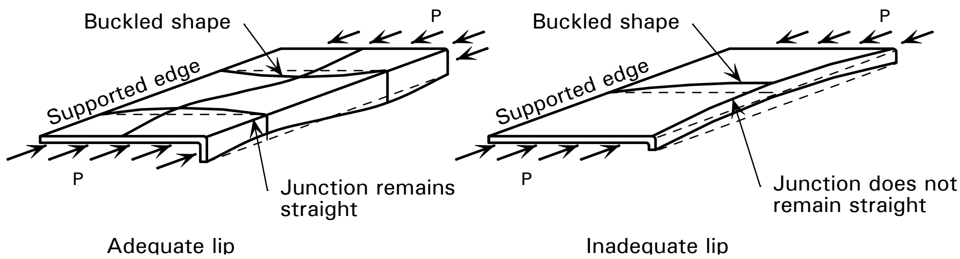


Figure C.4.17 *Adequate and inadequate lips*

Talja[91] carried out single span tests on three different stainless steel trapezoidal sheeting profiles (unstiffened, one stiffener in the flanges, one stiffener in the flanges and two in the webs). The mean value of the plate thickness was 0,61 mm. The bending resistance of the sheeting was determined under gravity loading and uplift. Further tests were subsequently carried out on profiles of thickness 0,5 and 0,8 mm[92]. The test results were compared with the resistances predicted by EN 1993-1-3; good agreement was found, so it was concluded that the guidance for carbon steel is applicable to stainless steel. The guidance in Section 4.5.3 is taken from EN 1993-1-3. Note that the effective width formulae for stainless steel given in Section 4.4.1 should be used when assessing the effectiveness of stiffeners.

Test programmes on stiffened and unstiffened trapezoidal profiles made from cold worked stainless steel confirmed the applicability of these recommendations for cold worked material up to strength level CP500 (C850)[22].

C.4.6 Calculation of section properties

Cross section properties are used to calculate member slendernesses for overall buckling; net areas are used for local tensile strength; and the effective section is used for local and member buckling resistance of Class 4 cross-sections.

Being material independent, the geometric properties of a stainless steel section may be calculated by the same formulae as used for carbon steel members. Nevertheless, when considering thin gauge cold formed sections, some formulae and techniques may be unfamiliar, due to the nature of these products. This particularly applies to linear methods of calculation (in which the properties of line elements are multiplied by the sheet thickness to derive cross-section properties), and to the calculation of the warping constant. For the former, a good source of information is the AISI cold formed specification[93] and, for the latter, standard texts (e.g. Reference 94) may be consulted.

The simplifications given in the Recommendations for ignoring or approximating the rounding of corners are as given in EN 1993-1-3.

The recommendations given in Section 4.6.4 for calculating the net area follow those given in EN 1993-1-1.

C.4.7 Resistance of cross-sections

The resistance of a cross-section under various forces and moments, as given in 4.7.2 to 4.7.6 inclusive, is limited either by plasticity or local buckling. The formulae are generally based on EN 1993-1-1 and follow common sense.

For cross-sections in bending, the appropriate second moment of area (W_{pl}, W_{el} or W_{eff}) must be taken for the neutral axis about which the moment acts.

Class 4 cross-sections which are not doubly symmetric will, under external compression, experience a shift in the neutral axis giving rise to a secondary moment. These sections should thus be assessed using the provisions of 4.7.6. The shift in the neutral axis depends on the effective widths, which themselves depend on the assumed stress distribution across the cross-section. To avoid undue iteration, the provisions in Section 4.4.1 should be used; these are based on studies carried out for carbon steel members[95].

The expression for the ultimate resistance of the net cross-section at holes for fasteners contains a new parameter, k_r, which is discussed in Section C.6.2.3.

The potential benefits of taking the strain hardening properties of stainless steel into consideration are recognised in 4.7.7.

It has already been noted in the commentary to Section 4.3 that the proof stress is conventionally defined. For structural purposes, the 0,2% proof stress is normally used, whereas the 1% proof stress is favoured for pressure vessels; pressure vessels do not usually suffer from instability and changes in overall diameter are acceptable. Therefore, if a structure is not subject to instability and deformations are not critical, larger proof stresses than the 0,2% value should be permissible. The difficulty is to define to what degree strain hardening can be utilised. For extremely stocky members, this could be high but a lack of suitable data does not permit precise guidance to be given.

For the First Edition of the Design Manual, a limit of 1,2 times the 0,2% proof stress for the design strength was suggested. This was based on the results of the beam tests carried out for the First Edition of the Design Manual[96]. In these tests, Class 3 and Class 4 cold formed and welded beams exceeded the enhanced plastic moment (1,2 $f_{0.2}$ W_{pl}). The fact that the Class 3 and even the Class 4 cross-sections exceeded M_p is a reflection of the variability of test data. The results, shown in Figure C.4.18, also support the suspicions concerning the collapse of classes mentioned in C.4.3. It should be noted that for the particular material (6,3 mm thick 1.4404), 1.2 times the 0,2% proof stress corresponds approximately to the 1% proof stress. Thus, large deformations should be expected where enhanced strength is to be taken. The limit of 1,2 times the proof strength in Section 4.7.7 was removed in the Second Edition of the Design Manual, on the basis that it was unnecessarily conservative for certain situations.

Enhanced strength should not be used for long term loading, because of creep considerations.

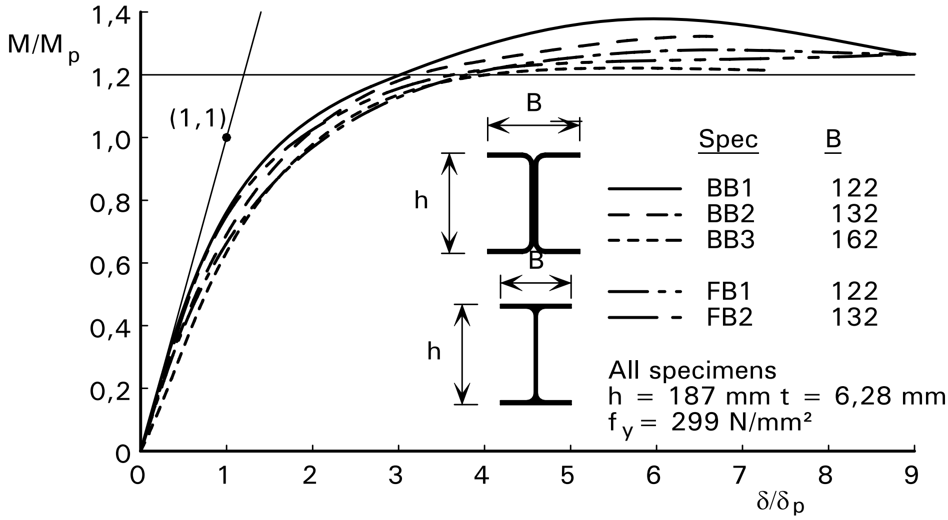


Figure C.4.18 *Moment/deflection response of tested beams*

C.5 MEMBER DESIGN

C.5.1 Introduction

No matter what the material of the member, a structural member essentially supports loads much in the same manner (e.g. by flexure or strut action). It is therefore perhaps a rather obvious statement that similar checks have to be carried out for stainless steel members as those for members in carbon steel. However, the designer should be aware of possible differences in design behaviour, such as second order effects or overall frame stability that are not covered in this Design Manual, but may be found in some carbon steel structural codes[2].

Elastic global analysis is recommended for establishing forces and moments in members. Although in principle plastic global analysis could be used, there are presently certain difficulties to be addressed in design. These difficulties are associated with the strain hardening properties of stainless steel and in particular the moment-rotation characteristic of a stainless steel beam likewise displaying hardening behaviour. In the formation of a plastic mechanism, plastic hinges are required to undergo various degrees of rotation. Thus, where strain hardening occurs, the moments at the hinges will be above the nominal plastic moment (plastic modulus multiplied by the 0,2% proof stress) by amounts depending on the degrees of rotation. Therefore the calculation of the distribution of moments around a frame would involve kinematic considerations. With further study, it may be possible to enable bounds to be put on the additional moments (above the nominal plastic moment) to circumvent these analytical difficulties. Connections would have to resist the enhanced moment. Alternatively, it may be possible to show that connections can provide the required rotation to realise the mechanism.

In considering instability caused by member buckling, reference is made to the tangent modulus approach. This approach is adopted by the American code for cold formed stainless steel[23]. The approach is based on replacing Young's modulus (in carbon steel buckling provisions) by the tangent modulus E_t corresponding to the buckling stress in the stainless steel member. Since E_t varies with stress and the buckling stress is a function of E_t, the approach generally requires iterations to find the solution.

In some of the recommendations given in this Design Manual, an effective design curve was derived by the tangent modulus approach, the necessary iterations having already been carried out for the designer. The derivation can be best demonstrated by way of an example.

Suppose it is required to find the stainless steel curve corresponding to the Euler buckling curve for carbon steel columns. For carbon steel (and any linear elastic material), the limiting stress f_{lim} is given by:

$$f_{\mathrm{lim}} = \pi^2 E\left(\frac{l}{i^2}\right)$$

Defining non-dimensional parameters:

$$\chi = \frac{f_{\text{lim}}}{f_y} \quad \text{and} \quad \bar{\lambda} = \frac{\ell / i}{\pi}\sqrt{\frac{f_y}{E}}$$

gives the limiting (Euler) curve, expressed as:

$$\chi = \frac{1}{\bar{\lambda}^2}$$

For stainless steel, E is replaced by the tangent modulus E_t:

$$f_{\text{lim}} = \pi^2 E_t \left(\frac{l}{i^2} \right)$$

Using the Ramberg-Osgood relationship for describing the stress-strain curve

$$\varepsilon = \frac{f}{E} + 0{,}002\left(\frac{f}{f_y} \right)^n$$

the tangent modulus can be derived as

$$E_t = \frac{df}{d\varepsilon} = \left[\frac{1}{E} + \frac{0{,}002n}{f_y}\left(\frac{f}{f_y} \right)^{n-1} \right]^{-1}$$

and therefore

$$\frac{E_t}{E} = \left[1 + 0{,}002\frac{nE}{f_y}\left(\frac{f}{f_y} \right)^{n-1} \right]^{-1}$$

But, at buckling $f = f_{\text{lim}}$, $(f_{\text{lim}}/f_y) = \chi$ and $\chi = \frac{1}{\bar{\lambda}^2}\left(\frac{E_t}{E} \right)$ so

$$\chi = \frac{1}{\bar{\lambda}^2}\left[1 + 0{,}002\frac{nE}{f_y}\chi^{n-1} \right]^{-1}$$

In general, to solve χ = function$(\bar{\lambda})$, an iterative approach is required since χ appears on both sides. However, on rearrangement:

$$\bar{\lambda} = \left[\chi + 0{,}002\frac{nE}{f_y}\chi^n \right]^{-1/2}$$

From this equation, a family of curves can be generated for each value of n depending on the ratio of E/f_y. Some example curves are compared with the original Euler curve for carbon steel in Figure C.5.1. All the designer has to do now is to calculate $\bar{\lambda}$ using the initial modulus value (the modulus of elasticity within the limit of proportionality) and then find χ directly using the appropriate curve.

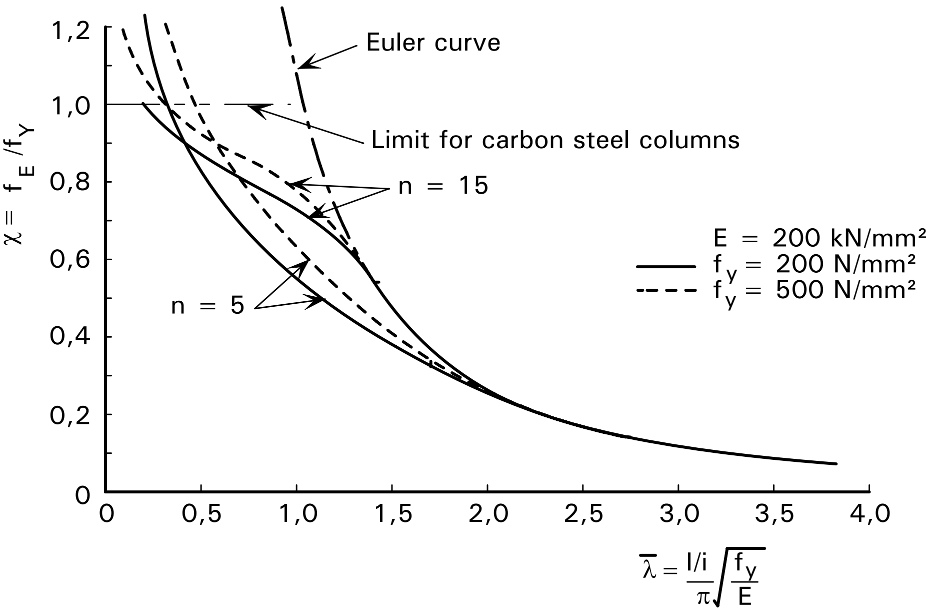


Figure C.5.1 *Effective 'design' curves for Euler column buckling in stainless steel*

As can be seen the curves with the lower *n* value, which implies a lower limit of proportionality, diverge from the carbon steel curve at lower stresses than do the curves associated with the higher *n* value. However, at stresses above 0,9 f_y, the curves with low *n* value lie above those of high *n*; this follows from the fact that the tangent modulus of the low *n* material is greater than that of the high *n* material in this stress range. It may be noted that a carbon steel stress-strain curve may be closely approximated by very high *n* values (say > 30), in which case the design curve departs from the Euler curve and becomes a horizontal plateau at $\chi = 1,0$.

Although the above technique has been occasionally used to derive effective design curves, greater credence has been attached to establishing the recommended curves with available experimental data. For instance, it is known that the Euler curve discussed above is a poor representation of the true strength of columns within the practical slenderness range because of the influence of factors including initial out-of straightness, eccentricity of loading and residual stresses.

The last paragraph in Section 5.1 states that the design recommendations should not be applied to members having cross-sections not possessing any axis of symmetry. Carbon steel codes are similarly restricted.

C.5.2 Tension members

In general, tension members and their connections should be detailed such that the applied load acts along the member's centroidal axis. This is not always possible and the eccentric load will induce bending, which should be allowed for by reference to 4.7.6.

However, in the case of angles, recommendations are given for simple design, ignoring the moments due to eccentricity, using a modified expression for the ultimate tensile resistance in Section 6.2.3.

C.5.3 Compression members

C.5.3.1 General

The various forms of buckling listed in the Recommendations are in common with those pertinent to carbon steel columns. Indeed, the behaviour of stainless steel columns and carbon steel columns can be expected to be broadly similar, differing only in quantitative aspects. It may be helpful to consider how the non-linear stress-strain curve of stainless steel affects the comparison between the buckling strengths of similar stainless steel and carbon steel columns and members in general. There are three distinct regions of slenderness:

(a) At high slendernesses, i.e. when the axial strength is low, stresses in the stainless steel member are sufficiently low so that they fall in the linear part of the stress-strain curve. In this range, little difference would be expected between the strengths of stainless and carbon steel members assuming similar levels of geometric and residual stress imperfections. The limiting slenderness beyond which similar behaviour can be expected depends on the limit of proportionality and hence the *n* factor in the Ramberg-Osgood representation of the stress-strain curve. This dependence can be seen in Figure C.5.1.

(b) At low slenderness, i.e. when columns attain or exceed the squash load (area x proof strength), the benefits of strain hardening become apparent. For very low slenderness, materials with higher hardening rates, i.e. materials of low *n* factors, will give superior column strengths to materials having high *n* factors and in particular carbon steels.

(c) At intermediate slendernesses, i.e. when the average stress in the column lies between the limit of proportionality and the 0,2% proof strength, stainless steel is 'softer' than carbon steel. This leads to reduced column strengths compared to similar carbon steel columns.

C.5.3.2 Flexural buckling

The buckling resistance in the Recommendations is given as the product of a reduction factor (χ) and the stub column resistance ($\beta_A\ A_g\ f_y$) divided by the 'material' factor for buckling (γ_{M1}). The reduction χ depends on the non-dimensional column slenderness $\bar{\lambda}$ and the appropriate column curve selected according to the constants given in Table 5.1. The reduction factor is derived from the lower root of the following equation and is based on the work of Ayrton and Perry (1886) in the UK and others on the continent:

$$(p_y - p_c)\ (p_E - p_c) = \eta\ p_E\ p_c$$

in which:

$p_y = \beta_A f_y$

$p_c = \chi\ p_y$

$p_E = \pi^2 E/(\ell\ /i)^2$

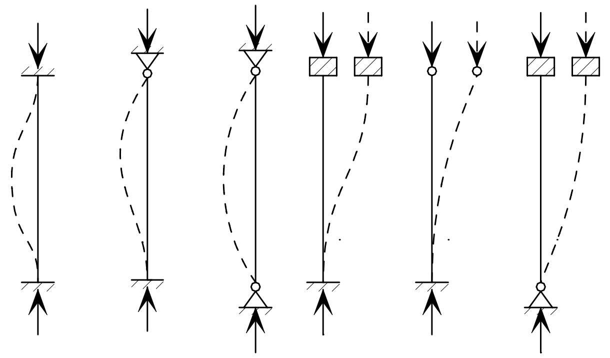
η is an empirically defined imperfection coefficient, each buckling curve having its associated value.

The equation is based on column failure being attained when the maximum stress in the compression fibre reaches p_y and takes into account the amplification of secondary 'imperfection' moments by the axial load.

The reduction factor is given as a function of the non-dimensional slenderness $\overline{\lambda}$ which is proportional to the effective length ℓ of the column. The effective length of a column is the length of a pin-ended member, of the same cross-section, that has the same buckling resistance as the actual member under consideration. Note that the length of a compression member, and hence the effective length, may be different for the two planes of buckling. The effective length factor of a compression member is dependent upon the conditions of restraint afforded to the member at its restraints and theoretically may vary from 0,5 to infinity. In practical structures the variation is somewhat less, ranging from 0,7 to perhaps no more than about 5.

Six idealised cases are illustrated in Figure C.5.2. For rigid jointed frames the restraining influence of incoming beams may be taken into account by reference to, for example, ENV 1993-1-1: Annex E.

Buckled shape of column is shown by dashed line



Theoretical effective length factor	0,5	0,7	1,0	1,0	2,0	2,0
Effective length factor for use in design	0,7	0,85	1,0	1,2	2,0	2,0

Figure C.5.2 *Effective length factors*

In some carbon steel codes[2], effective non dimensional slendernesses, $\overline{\lambda}_{\text{eff}}$, are given for angles in compression such that the effects of secondary moments, induced at the ends due to connection eccentricity, do not have to be explicitly considered. These expressions are empirical and cannot be verified for stainless steel angles, due to lack of data. Based on other evidence, it is likely that $\overline{\lambda}_{\text{eff}}$ would be slightly larger for stainless steel.

The constants α (imperfection coefficient) and $\overline{\lambda}_0$ (length of plateau region) in Table 5.1 were chosen after considering available data as follows.

Cold formed members (α = 0,49, $\overline{\lambda}_0$ = 0,40)

Hammer and Petersen[97]

This paper contains by far the largest single source of column test data for stainless steel. Over 200 specimens of annealed, ¼ hard, ½ hard and fully hard type 1.4310 stainless steel were tested in slenderness ratios (ℓ/i) varying from 15 to 120. Specimens were prepared parallel and transversely to the rolling direction. Material thicknesses varied from 0,5 mm to 1,9 mm. All columns were built up from two cold formed hat sections spot welded to form a closed member. The section dimensions were designed so as to avoid local buckling of elements.

The non-dimensionalised results are plotted in Figure C.5.3. In the paper, no stub column proof loads are given; only the sheet proof strengths in the various tempers can be ascertained and these vary from 203 N/mm^2 (annealed condition) to 1571 N/mm^2 (fully hard condition, transverse direction). The close banding of the results confirms the adequacy of the non-dimensional variables. The apparent conservatism of the recommended design curve at intermediate slendernesses arises from the fact that sheet proof strengths, rather than stub column proof strengths, are used in the non-dimensional variables and hence strength enhancement in the cold worked corners of the specimens is ignored.

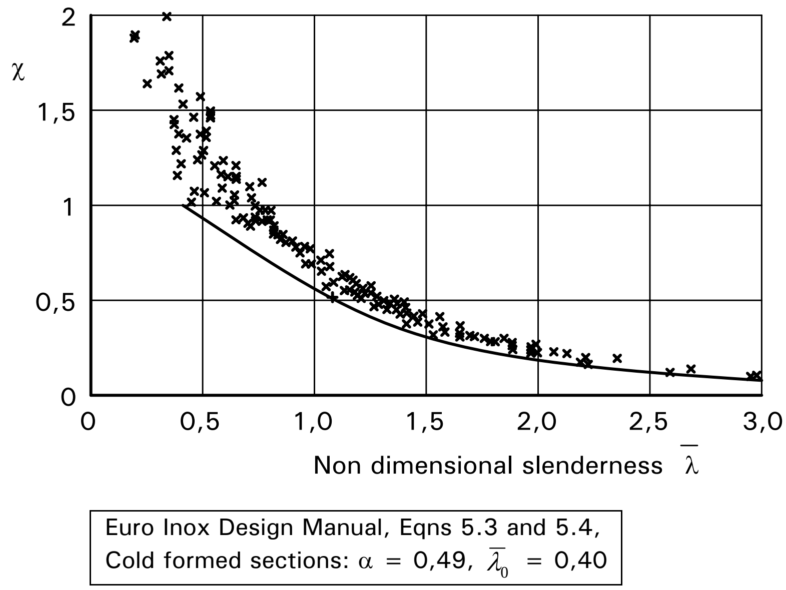
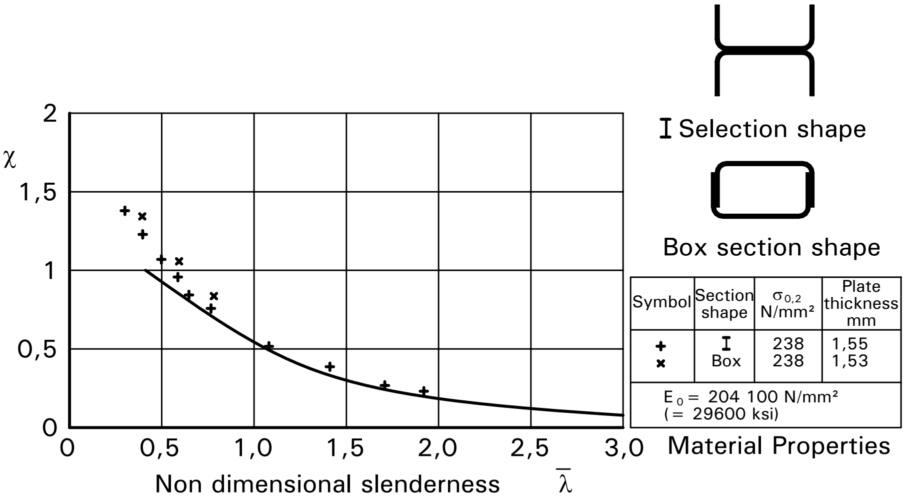


Figure C.5.3 *Reduction factor versus non-dimensional slenderness for cold formed sections from grade 1.4310 material* [97]

Johnson and Winter[98,99]

A total of 15 column tests were carried out on type 1.4301 annealed stainless steel. The columns were built up using 1,5 mm thick cold formed channel sections. These were placed back to back to produce 11 I-sections and placed (nested) together to form 4 box sections. The two pieces were joined by means of a structural adhesive. The range of slenderness ratios (ℓ/i) tested was from 28 to 177. All sections were designed to be Class 2, i.e. to avoid local buckling.

The proof strength used in the non-dimensional quantities is based on the 'typical' stress-strain curve given in the references. The results of the column tests are shown in Figure C.5.4.



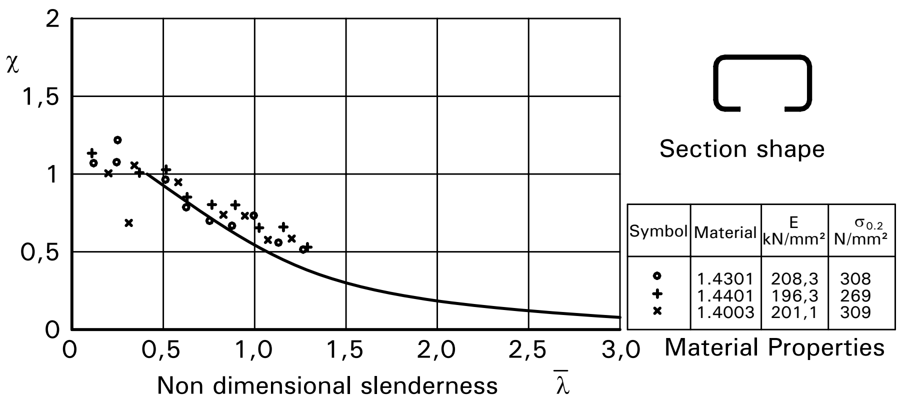
Euro Inox Design Manual, Eqns 5.3 and 5.4,
Cold formed sections: α = 0,49, $\bar{\lambda}_0$ = 0,40

Figure C.5.4 *Reduction factor versus non-dimensional slenderness for cold formed sections from 1.4301 material* [98,99]

Coetzee et al[100]

A total of 30 column tests were performed on three different grades of stainless steel. The materials under consideration were grades 1.4301, 1.4401 and the ferritic grade 1.4003. Ten lipped channel sections were produced from each material by a press braking process and cut into lengths which gave slenderness ratios (ℓ/i) ranging from 10 to 104. All materials used were approximately 2,5mm thick.

The results are given in Figure C.5.5.



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Cold formed sections: α = 0,49, $\bar{\lambda}_0$ = 0,40

Figure C.5.5 *Reduction factor versus non-dimensional slenderness for cold formed sections from grades 1.4301,1.4401 and 1.4003 materials* [100]

Rhodes, Macdonald and McNiff[101]

A total of 22 pin-ended lipped channel section columns in grade 1.4301 material were tested in minor axis buckling. The specimens consisted of eleven 28×15×8×2.5 (2.5 mm nominal thickness) and eleven 38×17×10×3 (3 mm nominal thickness) lipped channel sections, covering a broad range of slenderness. The results are shown in Figure C.5.6.

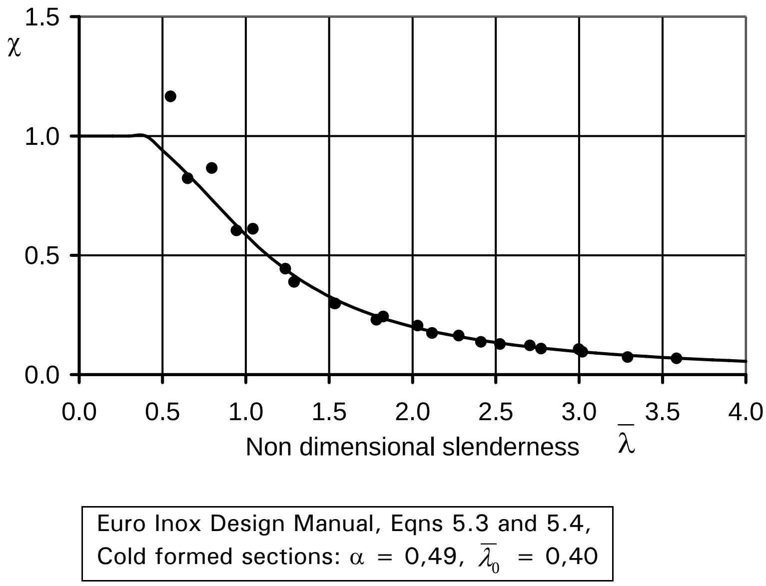


Figure C.5.6 *Reduction factor versus non-dimensional slenderness for cold formed sections from grade 1.4301*

Cold formed and seam welded members (α = 0,49, $\bar{\lambda}_0$ = 0,40)

Rasmussen and Hancock[102]

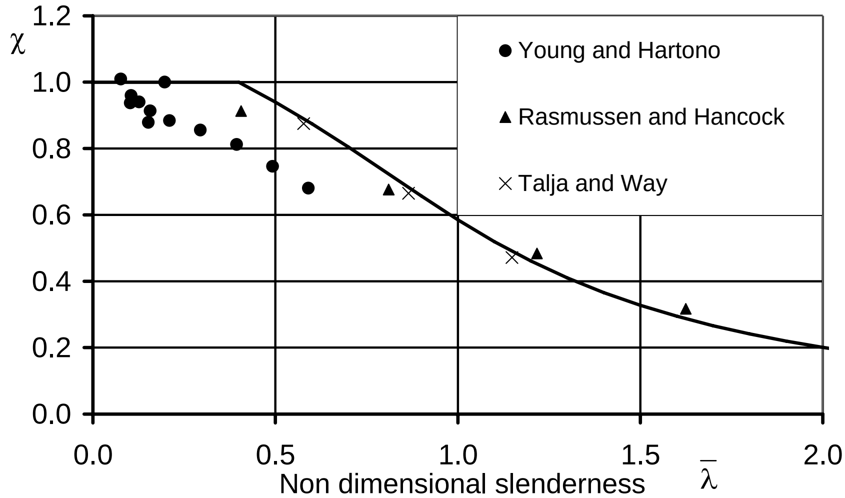
A total of 18 hollow section columns in 1.4307 material were tested. The specimens consisted of 8 square hollow sections of 80 mm x 3 mm and 10 circular hollow sections of 101,6 mm x 2,85 mm, of various lengths. They were formed by cold rolling and subsequent seam welding. The specimens were paired with one specimen in each pair tested with a slight eccentricity of load to simulate a geometric imperfection. The results are shown in Figure C.5.7.

Talja[91] *and Way* [103]

Nine circular hollow sections of diameter 140 mm and thickness varying from 2 to 4 mm in grades 1.4541 (a stabilised version of grade 1.4301) and 1.4435 (a slightly higher alloyed version of 1.4404) were tested. The test results are also shown on Figure C.5.6. The 4 circular hollow sections of 140 mm x 2 mm in grade 1.4435 were classified as class 4 cross-sections and as there is no guidance on the calculation of the effective cross-sectional area for Class 4 CHS, the results were not plotted. All sections were loaded concentrically in compression.

Young and Hartono[77]

Twelve fixed-ended circular hollow section columns in grade 1.4301 material were tested. The specimens consisted of five CHS 89×2.78, three CHS 168.7×3.34 and four CHS 322.8×4.32 and their non-dimensional slenderness ranged from 0.08 to 0.59. The results of all CHS tests are shown in Figure C.5.7. Many of these test results fall below the design curve for cold formed and seam welded members. Consideration should be given to reducing the limiting non-dimensional slenderness $\overline{\lambda}_0$ from 0,4 to 0,2 in future revisions of this Design Manual and EN 1993-1-4.



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Cold formed and seam welded sections: α = 0,49, $\overline{\lambda}_0$ = 0,40

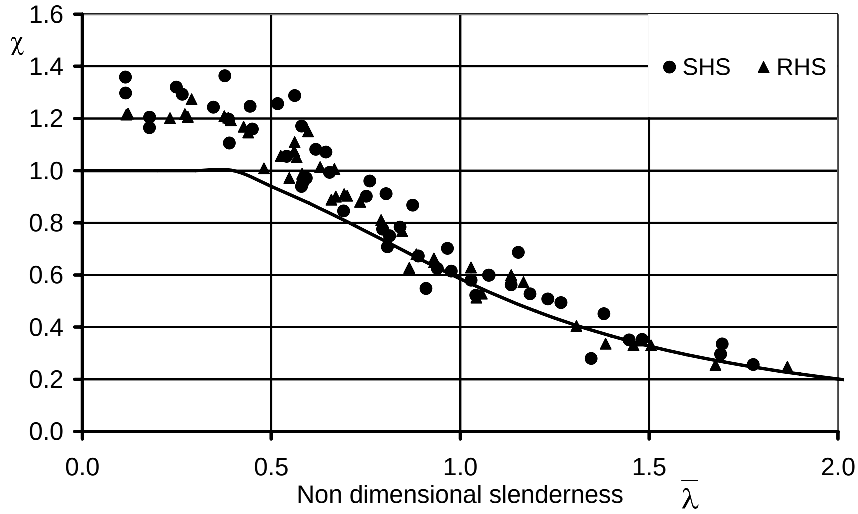
Figure C.5.7 *Reduction factor versus non-dimensional slenderness for circular hollow section columns*

Square and Rectangular hollow sections from cold worked material[22,80]

12 flexural buckling tests were carried out on SHS from cold worked austenitic material C700 and C850. The tests and subsequent numerical analysis showed that the expressions for calculating the flexural buckling resistance for annealed material are equally applicable to cold worked material.

Further tests on Square and Rectangular hollow sections

Further test results, as reported by Gardner and Nethercot[17, 104] (eight SHS and fourteen RHS pin-ended specimens in 1.4301 material), Talja and Salmi[73] (three SHS and six RHS major axis pin-ended specimens in 1.4301 material), Young and Liu[74] (eight SHS and sixteen RHS minor axis fixed ended specimens in 1.4301 material) and Young and Lui[75] (sixteen SHS and eight RHS fixed ended specimens in duplex grade) are depicted in Figure C.5.8. The cross-sections considered are essentially identical to the relevant stub column tests described in: C.4.3.2. As shown in Figure C.5.8, the buckling curve for cold formed and seam welded members *(*α = 0,49, $\overline{\lambda}_0$ = 0,40*)* ensures safe design for both square and rectangular hollow sections.



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Cold formed and seam welded sections: α = 0,49, $\bar{\lambda}_0$ = 0,40

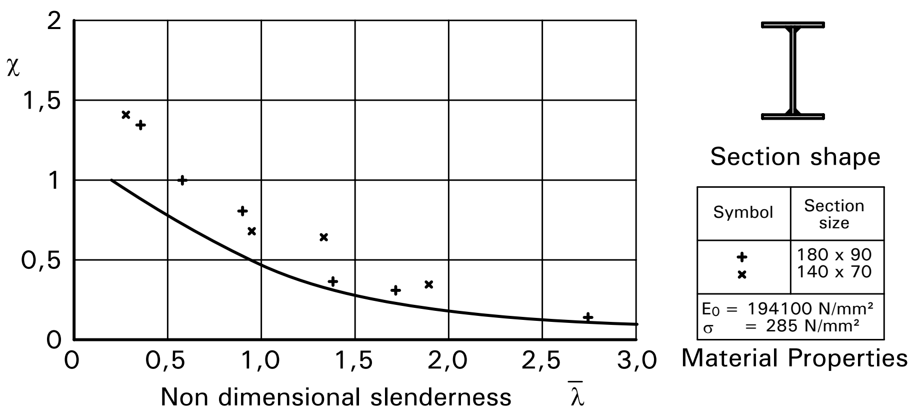
Figure C.5.8 *Reduction factor versus non-dimensional slenderness for square and rectangular hollow section columns*

Members fabricated by welding (α = 0,76, $\bar{\lambda}_0$ = 0,20 minor axis,
α = 0,49, $\bar{\lambda}_0$ = 0,20 major axis)

van den Berg et al [105]

A total of 13 column tests were carried out on I section columns fabricated from type 1.4003 material. The columns buckled about the minor axis. Seven columns were nominally sized 140 x 70 mm and the remaining six were sized 180 x 90 mm. Both section sizes used different plate thicknesses for web and flange. Column slendernesses (ℓ/i) varied from 24 to 230.

It should be noted that the section sizes used in the column tests were not categorically stated anywhere in the paper. The sizes quoted above were inferred from the predicted failure loads given and the geometrical properties of the sections used. The results are shown in Figure C.5.9.

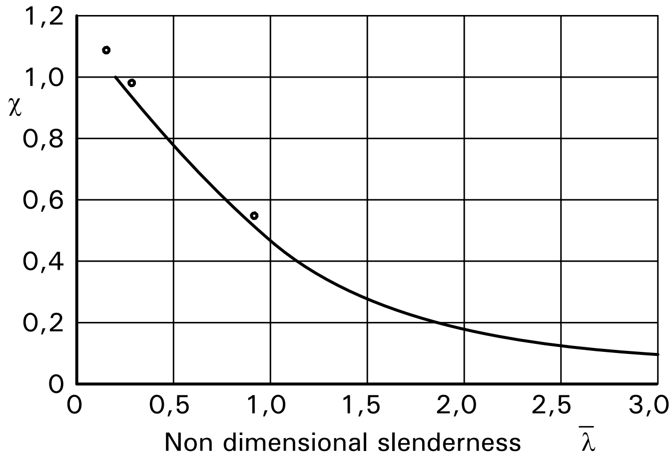


Euro Inox Design Manual, Eqns 5.3 and 5.4,
Welded sections, minor axis buckling: α = 0,76, $\bar{\lambda}_0$ = 0,20

Figure C.5.9 *Reduction factor versus non-dimensional slenderness for welded columns from grade 1.4003 material*[105]

Steel Construction Institute[96]

Three welded columns in 6,28 mm thick 1.4404 material, having the same cross-section but different lengths, were designed and tested to provide information for the First Edition of the Design Manual. All cross-sections were of 187 mm overall depth x 132 mm flange width. The columns buckled about the minor axis. The measured 0,2% compressive proof strength was 299 N/mm^2. The results are shown in Figure C.5.10.



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Welded sections, minor axis buckling: α = 0,76, $\bar{\lambda}_0$ = 0,20

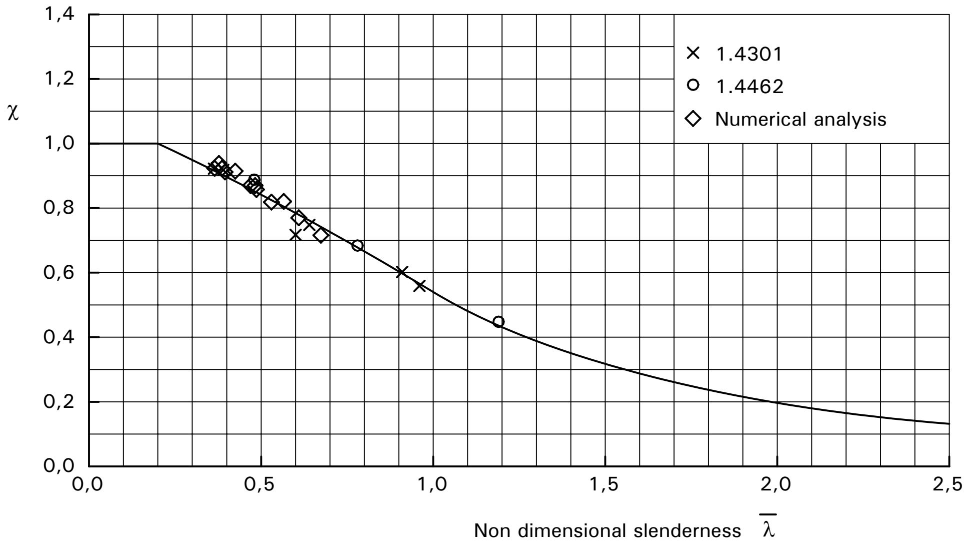
Figure C.5.10 *Reduction factor versus non-dimensional slenderness for welded columns in grade 1.4404 material*[96]

The curve recommended for welded stainless steel columns buckling about the minor axis (α = 0,76) is somewhat below the curve in EN 1993-1-1 for similar carbon steel cold formed columns. The welded stainless steel columns were measured as being reasonably straight and it may be inferred that the reduced strengths were due to the presence of the severe residual welding stresses to be expected in austenitic stainless steel welds (see Section 10.4.4 in the Recommendations). It is to be noted that the ferritic stainless steel columns in Figure C.5.9, in which residual stresses would have been closer to those found in carbon steel columns, did not suffer the same degree of reduction in capacity. Furthermore, it may be conjectured that duplex 1.4462 columns would perform similarly to ferritic steel. However, in the absence of data, it is recommended to use the α = 0,76 curve for minor axis buckling, even for duplex 1.4462 welded columns. Finally, it is also possible that the 0,76 curve is too conservative for hot produced products as, again, residual stresses would not be expected to be as severe as those in welded columns.

Talja[91] *and Stangenberg*[106]

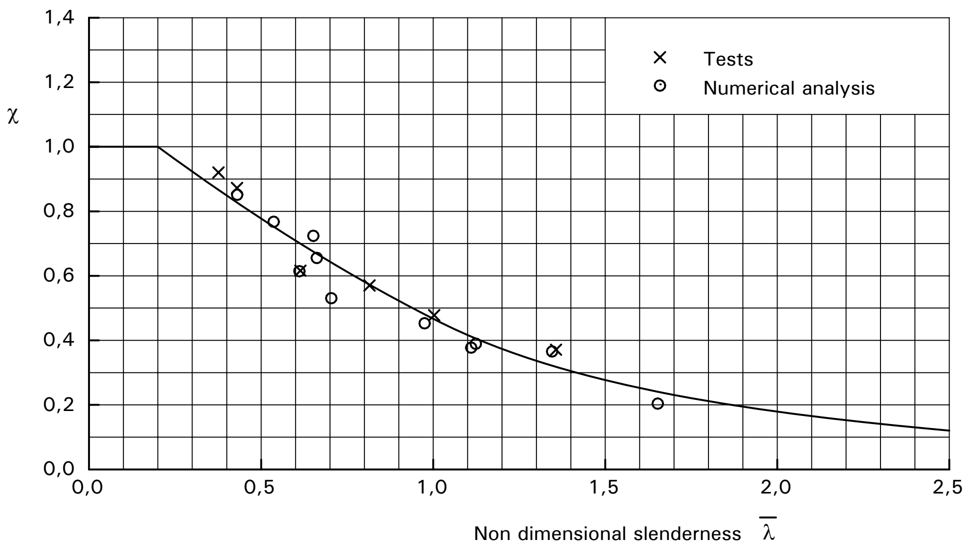
Recent tests were carried out on welded I section columns fabricated from grades 1.4301 and 1.4462. Three sections of 160 x 80 mm and 3 sections of 160 x 160 mm in 1.4301 material and 3 sections of 160 x 160 mm in grade 1.4462 were tested for buckling about the major axis. The results are shown in Figure C.5.11. In addition, three welded I sections of 160 x 80 mm and 3 of 160 x 160 mm in grade 1.4301 were tested for buckling about the minor axis (Figure C.5.12). In all cases, the sections were loaded in concentric compression.

The tests were modelled using a finite element analysis program and good agreement was obtained between the numerical model and test results. A parametric study looked at a wider range of slendernesses than that tested. The results of this numerical study are also shown on the Figures. The results of the tests and numerical analysis indicate that for major axis buckling, a more favourable buckling curve, $\alpha = 0{,}49$ can be used.



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Welded sections, major axis buckling: $\alpha = 0{,}49$, $\bar{\lambda}_0 = 0{,}20$

Figure C.5.11 *Reduction factor versus non-dimensional slenderness for welded columns buckling about the major axis*



Euro Inox Design Manual, Eqns 5.3 and 5.4,
Welded sections, minor axis buckling: $\alpha = 0{,}76$, $\bar{\lambda}_0 = 0{,}20$

Figure C.5.12 *Reduction factor versus non-dimensional slenderness for welded columns buckling about the minor axis*

C.5.3.3

C.5.3.4 Torsional and torsional-flexural buckling

The torsional and torsional-flexural buckling modes are treated in a very similar manner to the flexural buckling mode in 5.3.3. That is, the elastic critical stresses pertaining to these modes are used instead of the flexural critical stress (the Euler stress) in the Perry-type column analysis.

The column curve selected (α = 0,34 and $\overline{\lambda}_0$ = 0,2) for these modes is the same as that given for carbon steel columns in EN 1993-1-3. This recommendation is based on an assessment of the test data reported in References 107 and 108. These data were obtained from tests on cold formed hat sections produced from four different types of stainless steel and a carbon steel. Up to three sizes of hat sections were used with any one material. The results are presented in Figure C.5.13 in terms of the reduction factor χ and torsional-flexural slenderness $\overline{\lambda}_{TF}$, the stub-column proof strengths being used in all calculations.

It should be noted that $\overline{\lambda}_{TF}$ is a function of the effective length for twisting which, for the tests, is difficult to be precise about, due to the nature of the supports used - a ball bearing at each end. It was assumed that the axial load would provide sufficient friction at the bearings to prevent twisting at the ends of each column and thus an effective length factor for twisting of 0,7 (see C.5.4.2) was taken. It should be noted that different assumptions for the effective length for twisting would displace the data points either to the left or right of their positions in Figure C.5.13. Thus the design line and the data points should not be regarded as being fixed relative to each other. However, the above assumption is considered to be probably conservative but, more importantly, the inclusion of carbon steel columns and their ensuing results gives confidence that stainless steel columns are at least equal in strength to carbon steel columns for torsional-flexural failure.

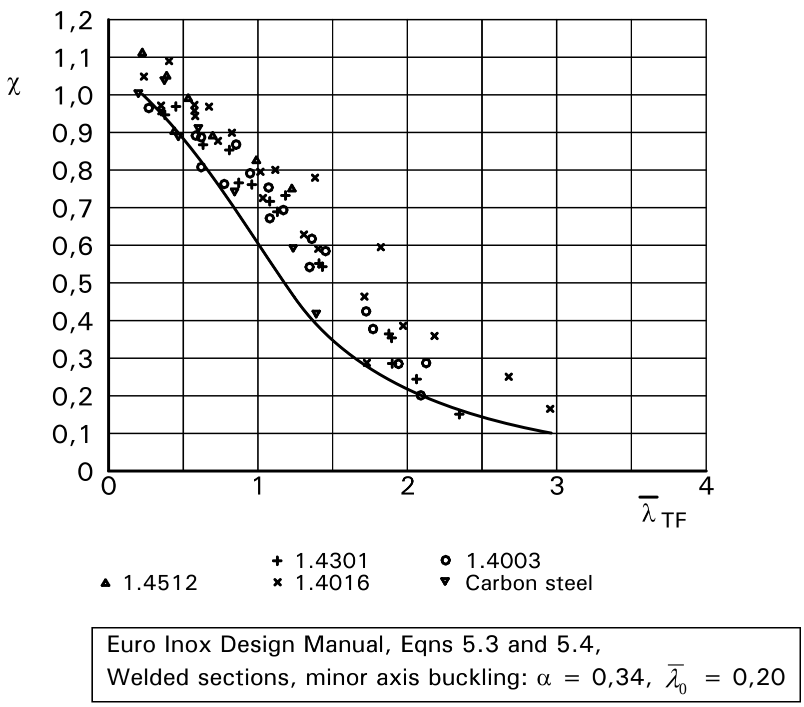


Figure C.5.13 *Reduction factor versus non-dimensional slenderness for torsional-flexural buckling*

C.5.4 Flexural members

C.5.4.1 General

Again, checks for establishing the resistance of a stainless steel beam are similar to those carried out for carbon steel beams.

With respect to overall member buckling (i.e. lateral-torsional buckling), the general comments given in C.5.3.1 also apply here.

C.5.4.2 Lateral-torsional buckling

When the compression flange of a beam is not fully restrained, it has a tendency to buckle sideways. The tension flange, on the other hand, tries to remain straight, with the net effect that the beam twists about its longitudinal axis as the beam buckles laterally. Restraints may be considered to be effective against lateral-torsional buckling if they provide either resistance to lateral movement or prevent twisting of the section. No guidance is given in the Recommendations as to what constitutes an adequate restraint but there is no reason why rules developed for carbon steel beams should not suffice, e.g. lateral restraints should be capable of sustaining a nominal force of 2½% of the compression flange force and should be connected to a stiff part of the structure.

Note that lateral-torsional buckling is not a possibility when bending is about the minor axis; in this case flexural buckling always governs.

For an idealised perfectly straight elastic beam, there are no out-of-plane deformations until the applied moment reaches the critical moment M_{cr} when the beam buckles by deflecting laterally and twisting. The failure of an initially straight slender beam is initiated when the additional stress induced by elastic buckling reaches yield. An initially straight beam of intermediate slenderness may yield before the critical load is reached, because of the combined effects of in-plane bending stresses and residual stresses, and may subsequently buckle inelastically. For very stocky beams, the inelastic buckling load may be higher than the in-plane plastic collapse load and the resistance moment of the beam is not affected by lateral-torsional buckling. Real beams differ from the idealised beams in much the same way as real compression members differ from idealised struts. Following the approach adopted for column design, beam design to 5.4.2 is based on an empirical adaptation of the Perry formula.

In a strut, the compression is generally constant throughout its length, but in a beam the bending moment and therefore the force in the compression flange usually varies along its length. The variation of the flange compression along the beam affects the buckling load of the member. This is taken account when calculating the slenderness λ_{LT} in Appendix B. Likewise the effect of various restraint conditions and whether the load is destabilising or not are also accounted for in the calculation of λ_{LT}.

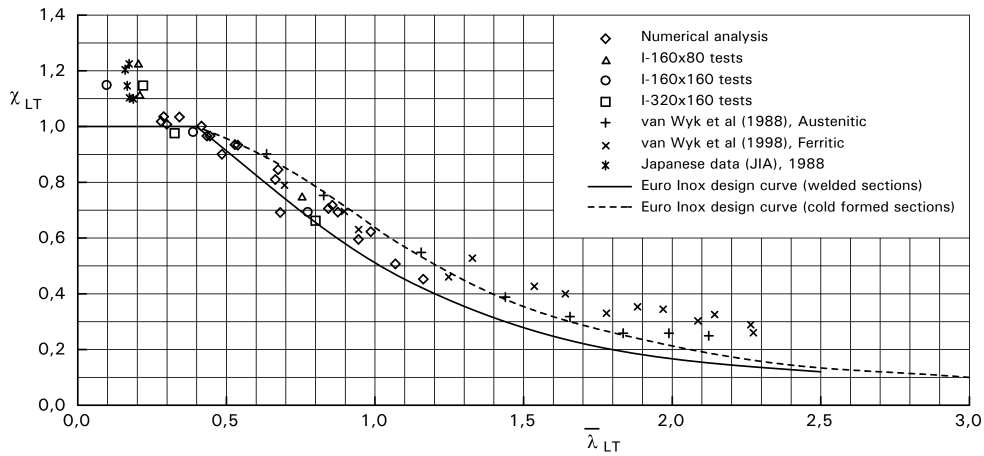
Tests by van Wyk et al[109] involved beams in three materials (types 1.4301, 1.4016 and 1.4003) of lengths ranging from 300 mm to 1600 mm under three point bending. The same cross-section was used in all tests and comprised two cold formed 50 mm x 15 mm channels joined back-to-back. The load was applied above the top flange and could move with the beam as it buckled, i.e. it was a destabilising load. The results are shown in Figure C.5.14. Note that the ordinate is a reduction factor applied to the plastic moment of resistance.

The other data available at the time of preparing the First Edition was Japanese data[110] for short welded I beams. Discounting those beams which failed prematurely by local flange buckling, the Japanese data fall around $\bar{\lambda}_{LT} = 0,18$ in Figure C.5.14. There were no other data available at the time the First Edition was written relating to lateral-torsional buckling of welded stainless steel beams.

The design line proposed in the First Edition for cold formed sections was based on an imperfection coefficient of $\alpha = 0,34$ and a limiting slenderness $\bar{\lambda}_{0,LT} = 0,2$ (as compared to $\alpha = 0,21$ and $\bar{\lambda}_{0,LT} = 0,2$ for cold formed carbon steel members in EN 1993-1-1). However, carbon steel data suggested that the plateau region is much longer and in EN 1993-1-1 no allowance needs to be made for lateral torsional buckling when $\bar{\lambda}_{LT} \leq 0,4$. A vertical step is thus introduced into the design curve. For stainless steel there were insufficient data to support this and a more conservative requirement that no allowance needed to be made for lateral torsional buckling when $\bar{\lambda}_{LT} \leq 0,3$ was introduced, again leading to a vertical step in the design curve.

Since the buckling curve recommended for stainless steel cold formed sections ($\alpha = 0,34$) was the next lower curve to that for carbon steel cold formed sections ($\alpha = 0,21$), it was suggested that $\alpha = 0,76$ may be suitable for welded stainless steel sections (compared to $\alpha = 0,49$ for welded carbon steel sections).

The Japanese data verified that no allowance needed to be made for lateral torsional buckling when $\bar{\lambda}_{LT} \leq 0,3$ for welded beams and hence also was conservative for cold formed beams.



Euro Inox Design Manual, Eqns 5.12 and 5.13,
Cold formed sections: $\alpha = 0,34$, $\bar{\lambda}_0 = 0,40$
Welded sections: $\alpha = 0,76$, $\bar{\lambda}_0 = 0,40$

Figure C.5.14 *Reduction factor versus non-dimensional slenderness for lateral torsional buckling*

For the Second Edition of the Design Manual, tests were carried out on three different sized welded I sections[91,106]. Three sections of 160 x 80 mm, 3 of 160 x 160 mm and 3 of 320 x 160 mm in 1.4301 material were tested. Also 3 welded I sections of 160 x 160 mm in grade 1.4462 were tested. These beams were tested in four-point bending. The levels under the force were free to move in the horizontal plane. There was also free rotation about the vertical axis, free movement in the horizontal plane and sideways translation. The results are plotted also in Figure C.5.14. These tests were modelled using a finite element analysis program and good agreement was obtained between the predicted results and test results. A parametric study looked at a wider range of slendernesses. The results of this study are also shown on the Figure.

The results of the tests and numerical analysis indicate that it is safe to increase the limiting slenderness, $\bar{\lambda}_{0,LT}$ to 0,4 and increase the limit on $\bar{\lambda}_{LT}$ above which it is necessary to allow for lateral torsional buckling from 0,3 to 0,4. The vertical step in the design curve in the First Edition was thus removed.

C.5.4.3 Shear resistance

The general approach for establishing the shear resistance of webs is based on the simple post-critical method of EN 1993-1-1. In comparison to the alternative tension field method, the simple post-critical method is more widely applicable (the tension field method is restricted for web panel aspect ratios *a*/*d* between an absolute lower limit of 1,0 and an economic upper limit of 3,0) and is simpler in application.

In common with other forms of plate buckling, slender plates under shear are able to reach ultimate strengths higher than the elastic critical stress values. The method takes advantage of this in the design line for carbon steel. This enhancement is also to be expected for slender stainless steel webs, as the stresses are low (see C.5.3.1). However, where web slendernesses are such that the elastic critical stress is approximately equal to the yield stress (at $\bar{\lambda}_w = 1{,}0$), a relatively large reduction in strength occurs.

There are few data on the shear behaviour of stainless steel webs. Whilst it is true that a number of tests have been conducted on beams, these have been to examine flange behaviour; flange failure prevented the development of high shears in the webs.

Carvalho et al[111] tested short span cold formed beams of varying depths in three materials (stainless steel grades 1.4301, 1.4016 and 1.4003). Each half of the beams was of square aspect ratio. When the First Edition of the Design Manual was prepared, these were the only shear buckling test data available and consequently a design curve was derived which gave a satisfactory lower bound to the experimental data. The design curve was subsequently adopted in ENV 1993-1-4. Since then, the validity of these data has been questioned. For example, the short member lengths led to results which were difficult to analyse correctly because the basic kinematic assumption by Bernoulli was less accurate. The method chosen to transfer the load into the webs also led to difficulties in analysing the results. In addition, the test specimens were cold formed profiles with internal radii which gave less favourable conditions for a tension field to develop, compared to an I-section.

A later test programme by Olsson[112] included 8 tests on welded I-sections. Four different cross-sections and two different stainless steel grades (1.4301 and 1.4462) were considered. The beams were doubly symmetric, with the same flange dimensions and the same web height. The web slenderness h_w/t_w varied between 37,5 and 200, and the web aspect ratio varied between 2 and 3. The test results showed that the existing design procedure was very conservative. The tests were modelled using a finite element program and good agreement was achieved between the predicted results and test results. A subsequent parametric study analysed the shear buckling resistance in a wider range of I-section beams than was tested.

A new design procedure was developed which did not take into account the work carried out by Carvalho et al. The new procedure is closely based on the procedure in ENV 1993-1-5 (and subsequently retained in EN 1993-1-5). Figure C.5.15 shows the test results and design curves in ENV 1993-1-5 (for carbon steel), the previous conservative approach included in ENV 1993-1-4 and the new design curve. This design approach was subsequently adopted in EN 1993-1-4.

Real has also carried out an experimental and numerical investigation to study the response of stainless steel plated girders subjected to shear load[113]. A method for predicting the shear resistance of stainless steel beams based on the tension field method in ENV 1993-1-1 is proposed by Real, including new design expressions to determine the initial shear buckling stress.

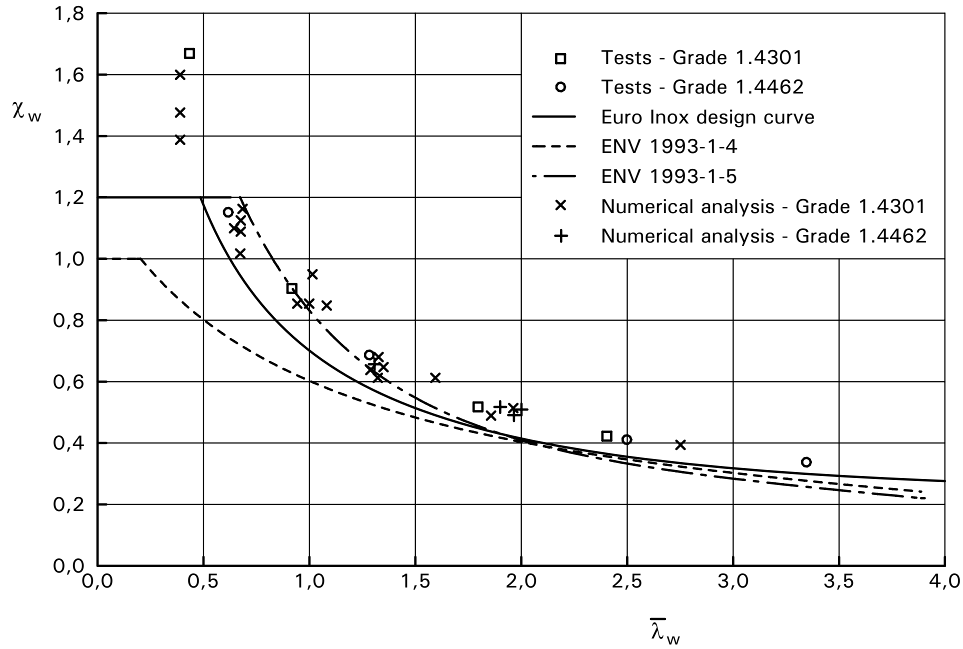


Figure C.5.15 *Reduction factor versus non-dimensional slenderness for shear buckling of webs*

C.5.4.4 Web crushing and crippling

When the First Edition of this Design Manual was written, there were no relevant test data on stainless steel webs, and so the use of guidance for carbon steel was recommended. Since then, a test programme was carried out to measure the web crushing and crippling resistance of stainless steel plate girders[114]. Nine grade 1.4301 welded I-section beams were subjected to concentrated point loads. On five of the beams, the load was applied far from the girder end (patch loading) and on the remaining four beams the load was applied near an unstiffened end (end patch loading).

For the patch loading, the beams were doubly symmetric, with h_w/t_w varying from 50 to 110 and the lengths of the beams varying from 996 mm to 1682 mm. Both ends of the beams were stiffened with vertical steel plates. Loading plates of width 40 mm and 80 mm were used. The load was applied at the midspan of the simply supported beam, on the upper flange, centrally over the web.

For the end patch loading, the beams were doubly symmetric with h_w/t_w varying from 50 and 80 and the lengths of the beams varying from 996 mm to 1682 mm. The width of the loading plates varied from 20 mm to 60 mm. One end of the beam was stiffened with a vertical steel plate and the load applied at varying distances from the unstiffened beam end.

The patch load tests were modelled using a finite element analysis program and good agreement with the test results was obtained. A parametric study was carried out to study the behaviour of a wider range of web slendernesses.

The test and numerical results were analysed and comparisons made with the guidance given in ENV 1993-1-1[2], ENV 1993-1-1 Annex J and ENV 1993-1-5[59]. (The existing guidance given in ENV 1993-1-4 refers simply to ENV 1993-1-1.) The results indicated that the design procedure given in ENV 1993-1-5 (and subsequently retained in EN 1993-1-5) gives the best agreement between test and predicted values for both patch loading and end patch loading. In this model the characteristic resistance, F_r is a function of the yield resistance F_y, the elastic buckling load, F_{cr} and a resistance function $\chi(\lambda)$. Figure C.5.16 shows the results of the tests, numerical analyses and the design curve.

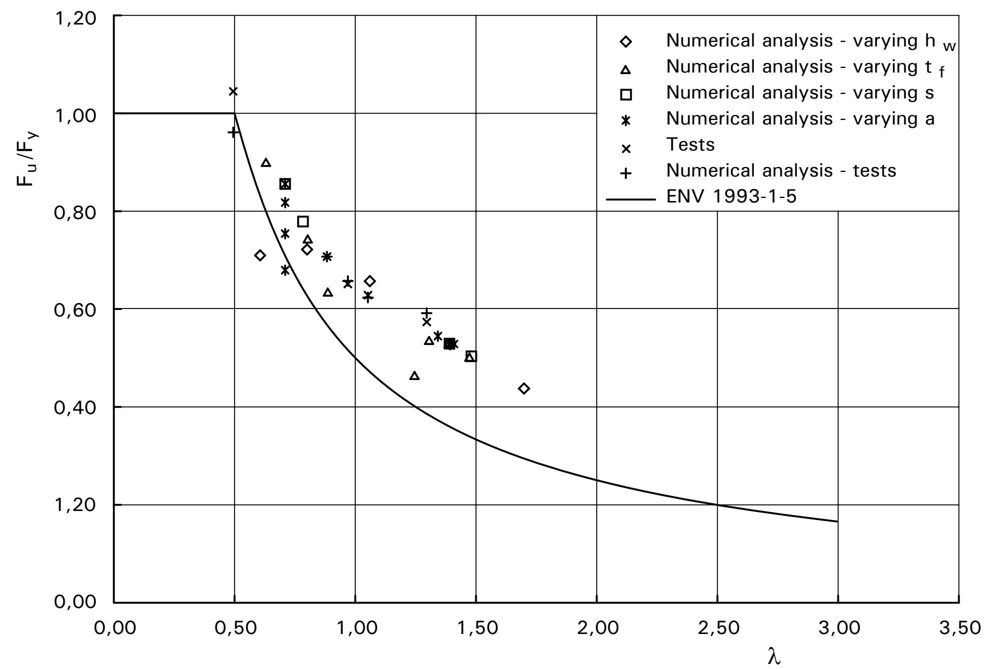


Figure C.5.16 *Web crippling of stainless steel beams – test data and design curve*

C.5.4.5 Transverse stiffeners

In essence, transverse stiffeners are to be treated as compression members requiring a check on cross-sectional resistance (bearing check) and buckling resistance. For intermediate stiffeners not subject to external loads, the axial loads are fed in gradually via shear in the web and the bearing check can be dispensed with.

The effective cross-section of the stiffener includes a proportion of web plate of up to $11\varepsilon t_w$ on either side of the stiffener flat. This effective width of web plate corresponds to the Class 3 limiting width for outstands in Table 4.2 of the Recommendations, i.e. the portion of web plate that can develop its proof load.

The buckling check is to be carried according to 5.3.3 or 5.5.2 depending on whether symmetric stiffeners or asymmetric stiffeners are used. In the latter case an eccentricity moment of $M = N_s e$ has to be allowed for, where N_s is the axial force in the stiffener and e is the distance of the centroid of the effective stiffener section from the mid thickness of the web.

The expression given for the force in an intermediate stiffener with no external loading is taken from EN 1993-1-5.

The requirements given for the minimum second moment of area are to ensure that the stiffeners are sufficiently rigid to prevent web buckling. They are the same empirical expressions as those used in EN 1993-1-5 and other steel codes.

C.5.4.6 Determination of deflections

The accurate calculation of the deflections of members composed of stainless steel materials is a complex matter. The shape of the load-deflection curve is affected by the non-linear material stress-strain relationship[115] and may be influenced by local buckling effects in the compression flange. Whereas in the case of carbon steel members the modulus is constant (i.e. equal to Young's modulus) down the beam depth and along the length of the beam, for stainless steel members the (tangent) modulus may vary throughout the beam according to the value of stress at each point. An accurate deflection calculation will generally require the use of iterative techniques and this is unsuitable for design.

In the Recommendations an approximate method is given. It uses the secant moduli (see Figure C.5.17) corresponding to the stresses in the extreme fibres as a basis for estimating deflections. This approach has been shown[99] to give adequate deflection estimates for design purposes. It should be borne in mind that deflection calculations can only provide estimates of the actual deflection that will occur in practice. Uncertainties in member restraint, element thicknesses, material behaviour (Appendix C), let alone the loading, imply that it is unreasonable to expect or seek mathematical exactitude in estimating deflections.

Calculating deflections by using a unique value of the secant modulus appropriate for the most highly stressed cross-section in the member leads to an over-estimation of deflections. The magnitude of the over-estimation depends on the distribution of the bending moment along the member; for example, the error is less significant for a beam with a uniform bending moment. A new methodology for calculating deflections in stainless steel beams, which takes fully into account the material non-linearity, has been proposed by Real[113]. It is based on an analytical expression for the moment-curvature relationship for stainless steel cross-sections. A new expression for an equivalent elastic modulus, which represents the general behaviour of the beam, is defined. The procedure involves integration along the length of the member. This method has been shown to give more accurate predictions of the deflections in stainless steel beams than the simple method given in the Recommendations.

It should be noted that beams may suffer some permanent deflection on removal of the load; this will be approximately $(1 - E_s/E)$ times the estimated total deflection (see Figure C.5.17).

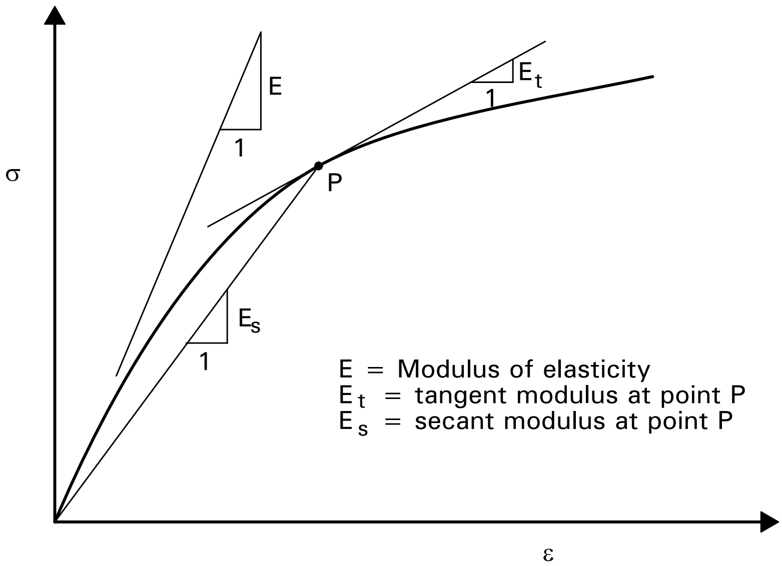


Figure C.5.17 *Young's tangent and secant moduli*

C.5.5 Members subject to combinations of axial loads and bending moments

C.5.5.1 Axial tension and bending

The expression is taken from EN 1993-1-1.

C.5.5.2 Axial compression and bending

The formulae given in the Recommendations for combined compression and bending are derived from the interaction formulae given in EN 1993-1-1. The interaction factors k_y, k_z and k_{LT} are complex functions dependent on the slenderness of the member and are a synthesis of the results of numerical work carried out by Greiner[116] and others.

For the Second Edition, six beam column tests were carried out on welded I-sections in grade 1.4301 stainless steel[91,106]. Three of the tests were numerically modelled and satisfactory agreement with the test results was obtained. In addition, eight pin-ended CHS columns were tested, with an axial load applied eccentrically through the centre of the wall thickness[91,103].

Both sets of test results were compared against the results predicted by the expressions in the existing Recommendations, and it was concluded that the design method predicted results with a satisfactory margin of safety[106,103].

C.6 JOINT DESIGN

C.6.1 General recommendations

C.6.1.1 Durability

The designer should consider ways of preventing corrosion at all stages of connection design.

Corrosion problems are most likely to occur at connections, whether they are bolted or welded connections. This is due to a number of potential deleterious features at connections such as crevices, dissimilar metal contact, heat affected zones, etc. As always, corrosion only occurs if there is a source of moisture. Sections 3.7 and 10.4 contain further information.

C.6.1.2 Design assumptions

The general recommendations given here are no different from those for carbon steel. Connections work, even where the assumed path is not actually realised, because of steel's great ductility and hence the potential for stress redistribution. In this respect, stainless steel, and particularly austenitic grade, is superior to carbon steel. Nevertheless, the deformation capacity of the fastening elements should be considered; it is not generally safe to share the load in a connection between different types of fasteners. For example, in a hybrid connection, fillet welds could fail before bolts in shear have taken up the clearances in the bolt holes.

C.6.1.3 Intersection and splices

Reducing bending moments at intersections and splices by avoiding eccentricities reflects good engineering practice.

At mid-height, the extreme fibres of a column are fully stressed (to f_y) at the ultimate limit state, even for a slender column (the reduction in strength due to column slenderness is matched by the stress due to the moment arising from strut action). Thus, any splice at mid-height has to be designed for forces and moments corresponding to the full design resistance.

C.6.1.4 Other general considerations

Although standardised details can be advantageous for carbon steelwork, the greater material cost of stainless steel favours a move away from uniformity of details to reduce such costs, even if increased labour charges result.

Again, the designer should be aware of the requirements of fabrication as given in Section 10. Control of welding distortion in particular should be noted, see Section 10.4.

C.6.2 Bolted connections

C.6.2.1 General

A variety of stainless steel fasteners is available, including bolts, rivets and self-tapping screws. The recommendations apply to bolts or set screws with washers under both the bolt head and the nut. Because of the soft surface of annealed austenitic stainless steel grades, hardened washers may be necessary to prevent any tendency to dig into the plate surface.

Stainless steel members will be connected to each other with bolted connections having similar geometric forms to those used in carbon steel structures. This being so, and with the expected broad similarity between stainless steel and carbon steel connection behaviour, the recommendations have been developed by verifying through testing the existing rules for carbon steel as set out in EN 1993-1-8. New provisions are introduced to limit bearing deformations.

C.6.2.2 Preloaded bolts

In carbon steel structures subject to vibration, high strength friction grip bolted connections are a viable option. The requirement that testing be carried out to prove the acceptability of stainless steel connections designed as slip resistant arises from the following concerns:

- Variable torque characteristics of stainless steel bolts.
- Stress relaxation in stainless steel bolts.
- Low coefficients of friction for stainless steel.
- The possibility of fretting corrosion.

Note also that welding the nut to the bolt to prevent the former from unscrewing is a practice to be avoided.

C.6.2.3 Connected parts

Holes

The standard hole sizes are in common with carbon steel values. For holes with greater clearances or for slotted holes, there are no data yet available for stainless steel and specific testing would have to be carried out.

Position of holes

The minimum criteria for pitch, end and edge distance are given for the following reasons:

- To give sufficient clearance for tightening bolts.
- To limit any adverse interaction between high bearing stresses on neighbouring bolts.
- To eliminate any tendency for bursting or in-plane deformation during drilling or punching; this reason particularly relates to minimum edge distance criteria.
- To provide adequate resistance to tear-out of the bolts.

These reasons are common to carbon steelwork[117]. The minimum spacings have been aligned with those for carbon steel in EN 1993-1-8 for the Third Edition of the Design Manual.

Maximum criteria are set for carbon steelwork to eliminate local buckling of the plies and to ensure that a continuous paint film is maintained across the plies, thus preventing corrosion at the interface. For stainless steel, the latter reason does not really apply and therefore the criteria in the Design Manual may be relaxed.

The position of holes is expressed in terms of the bolt hole diameter, d_0 rather than the bolt diameter, d, in accordance with EN 1993-1-8.

Bearing resistance

The great ductility of stainless steel permits a greater degree of redistribution of forces between fasteners than is the case with carbon steel. This allows the resistance of a connection to be assessed by summing the individual resistances at each bolt instead of taking the lowest resistance and multiplying it by the number of bolts. Nevertheless, care needs to be exercised, as failure by bolt shearing, in which the deformation is low compared to other failure modes, would limit the degree of redistribution possible.

The various failure modes that need to be considered for a bolted connection are illustrated in Figure C.6.1. These apply equally to connections in stainless steel or carbon steel.

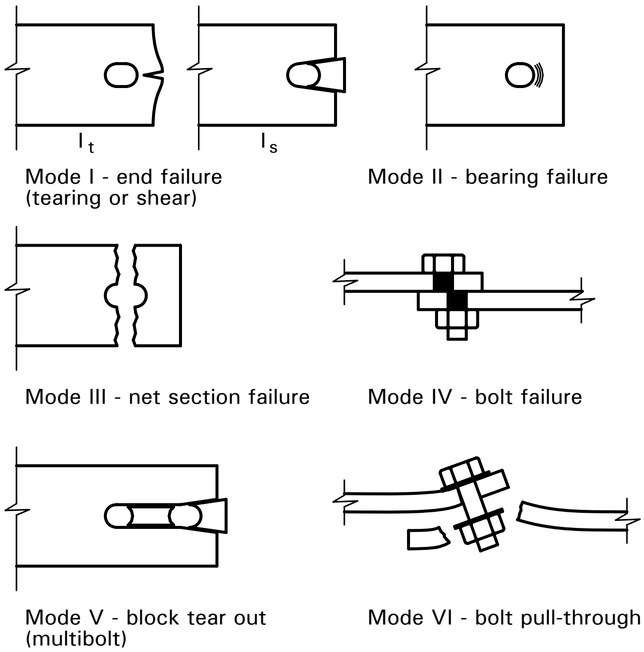


Figure C.6.1 *Bolted connection failure modes*

Excluding failure mode types III and IV (which are dealt with in Section 6.2.3 *Tension resistance* and 6.2.4 respectively), the bearing resistance is related to the α factor, which is given as follows:

mode I: $\alpha = e_1/3d$

mode II: $\alpha = 1,0$

mode V: $\alpha = p_1/3d - ¼$

mode VI: $\alpha = 0,6$

The suitability of these factors is verified for stainless steel bolted connections by reference to the following data:

Tests conducted for the First Edition of the Design Manual [118]

Some thirty-one bolted connection specimens were designed to fail in variety of modes, as shown in Table C.6.1. Specimens 1 to 25 were single bolt; specimens 26 to 31 had two or three bolts. Since the connected plies were under test, rather than the bolt, all bolts were of carbon steel up to grade 12.9.

Table C.6.1 *Summary of bolted connection tests*[118]

Specimen No	Steel grade	Nominal dimensions (mm)			$\frac{e_1}{d}$	Failure mode[1)]		∀exp ($0{,}4\ \Phi_{max\ bearing}/f_u$)	$\alpha_{exp}/\alpha_{predicted}$		Comments
		W	t	d		Predicted	Actual		Predicted mode	Actual mode	
1	1.4307	90	2	12	1,5	I	I_t	0,73	1,43	1,43	
2	1.4307	90	2	12	2,5	VI (I)	VI	1,09	1,81	1,81	
3	1.4307	90	2	12	3,5	VI (II)					Spec 3 scrapped
4	1.4307	90	2	12	4,5	VI (II)	VI	0,98	1,63	1,63	
5	1.4307	40	2	12	4,5	VI (III)	VI	0,89	1,48	1,48	
6	1.4307	40	2	12	4,5	VI (III)	III	0,88	1,46	1,02	
7	1.4404	90	2	12	1,5	I	I	0,76	1,46	1,46	
8	1.4404	90	2	12	2,5	VI (I)	VI	1,17	1,95	1,95	
9	1.4404	90	2	12	3,5	VI (II)	VI	1,06	1,76	1,76	
10	1.4404	90	2	12	4,5	VI (II)	VI	1,16	1,94	1,94	
11	1.4404	40	2	12	4,5	VI (III)	III	0,88	1,47	1,03	
12	1.4404	40	2	12	4,5	VI (III)	III	0,91	1,52	1,07	
13	1.4404	100	6,3	20	1,5	I	I_t	0,69	1,38	1,38	
14	1.4404	100	6,3	24	3,5	VI (II)	VI	0,72	1,19	1,19	
15	1.4404	75	6,3	24	3,5	VI (III)	VI	0,69	1,16	1,16	
16	1.4462	90	2	12	1,5	I	I_t	0,74	1,43	1,43	
17	1.4462	90	2	12	2,5	VI (I)	IV	0,98	1,63	-	bolt failure
18	1.4462	90	2	12	3,5	VI (II)	VI	1,05	1,75	1,75	
19	1.4462	90	2	12	4,5	VI (II)	IV	0,95	1,58	-	bolt failure
20	1.4462	40	2	12	4,5	VI (III)	III	0,87	1,45	0,99	
21	1.4462	40	2	12	4,5	VI (III)	III	0,85	1,41	0,99	
22	1.4462	90	5,2	24	1,0	I	I_s	0,45	1,25	1,25	
23	1.4462	90	5,2	24	2,0	VI (I)	VI	0,73	1,21	1,21	
24	1.4462	100	5,2	24	3,5	VI (II)	VI	0,76	1,26	1,26	
25	1.4462	100	5,2	24	4,5	VI (II)	VI	0,75	1,26	1,26	
26	1.4404	90	2	12 (2L)	2,5	V	V	0,86	1,24	1,24	2 bolts longitudinally, $p_1/d = 2.5$
27	1.4404	90	2	12 (3L)	2,5	V	III	0,73	1,09	0,86	3 bolts longitudinally, $p_1/d = 2.5$
28	1.4404	90	2	12 (2L)	2,5	V	III	0,94	1,07	0,74	2 bolts longitudinally, $p_1/d = 3.5$
29	1.4462	90	2	12 (2L)	2,5	V	III	0,91	1,30	0,72	2 bolts longitudinally, $p_1/d = 2.4$
30	1.4404	80	2	12 (2T)	3,5	VI (III)	III	0,95	1,58	1,09	2 bolts transversely
31	1.4404	120	6,3	20 (2T)	3,5	VI (III)	VI	0,67	1,11	1,11	2 bolts transversely

Notes:
1) See Figure C.6.1. Mode IV excluded from predicted mode. Mode shown in brackets is next critical predicted mode after VI.
2) All ∀ factors based on actual dimensions.
3) Net section failure load is taken as $A_{net}\ f_u$.

Inspection of the α factors will show that, for a single bolt specimen, mode II is not obtainable, as mode VI always intervenes. Indeed, this proved to be true because mode II never occurred. Nevertheless, it would appear that the margin of strength over and above the predicted failure load for mode VI is highly variable, and in some instances the predicted failure load for mode II also was exceeded.

The bearing stress at failure divided by the ultimate tensile strength of the material for each specimen is shown in Figure C.6.2, together with the design lines (with $\alpha = 1$) for modes I, II and VI. In all cases, the experimental value exceeds the design value (as can be seen in the tenth column of Table C.6.1). However this is only made possible by modifying the carbon steel rules for mode VI. In ENV 1993-1-1, this rule is worded to apply only to lap joints containing a single bolt (it is understood that this may not in fact be the intention). In specimen 31, containing two bolts, the critical mode would then become a net section failure for which $0{,}9\ A_{net}\ f_u = 258$ kN, whereas only 253 kN was actually measured in the test. Although the shortfall is marginal (–2%), this result should be compared to other net section failure data which generally show an excess of +3% (+9% in the case of specimen 30). Since specimen 31 actually failed in mode VI, the carbon steel rules are extended from single bolt lap joints to include any number of bolts in a single line lying transversely to the direction of stress.

Specimens 26 to 29, comprising two or more bolts disposed in the direction of stress, demonstrate that summing the individual bolt loads leads to conservative design values for the connections' resistances. The relatively low utilisation of the net section strength is discussed under *Tension resistance,* below.

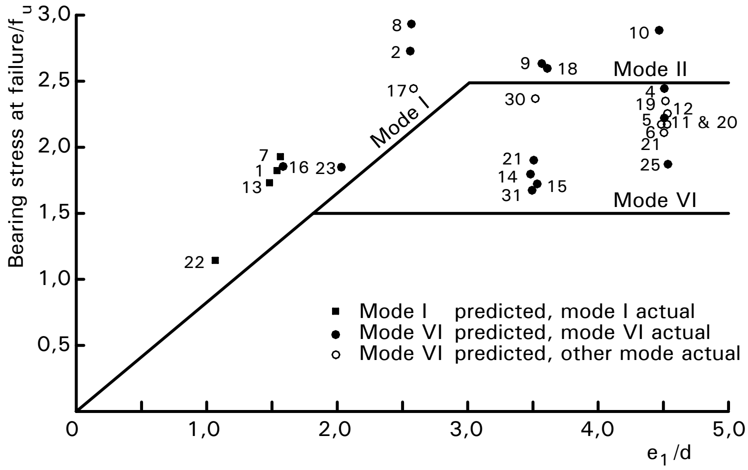


Figure C.6.2 *Comparison of experimental data with design lines*

The above is concerned with rupture, or extreme gross deformation, of the connection at the ultimate limit state. If the design rules were based on f_u, no rupture would occur but severe deformation would still be apparent at the ultimate limit state and an unacceptable level of deformation would exist at serviceability loads. The Recommendations therefore include provisions for using a reduced value of f_u (i.e. $f_{ur} = 0{,}5\ f_y + 0{,}6\ f_u$) to limit deformation. This formula has been derived by examining the loads at which the deformation is 3 mm for those specimens undergoing bearing deformation (i.e. specimens suffering net section failures are excluded). The formula is an approximation to the 'best fit' line through the data and the degree of fit is shown in Figure C.6.3. At the ultimate limit state, the deformation will be rather less than 3 mm because of the application of γ_{Mb}. At serviceability loads the deformation will be substantially less, due to the 'absence' of the load factor γ_F, and is likely to be of the order of 1 mm. This must be seen in the context of a possible slip of up to 2 mm (for M16 bolts and upwards) as the bolt goes into bearing.

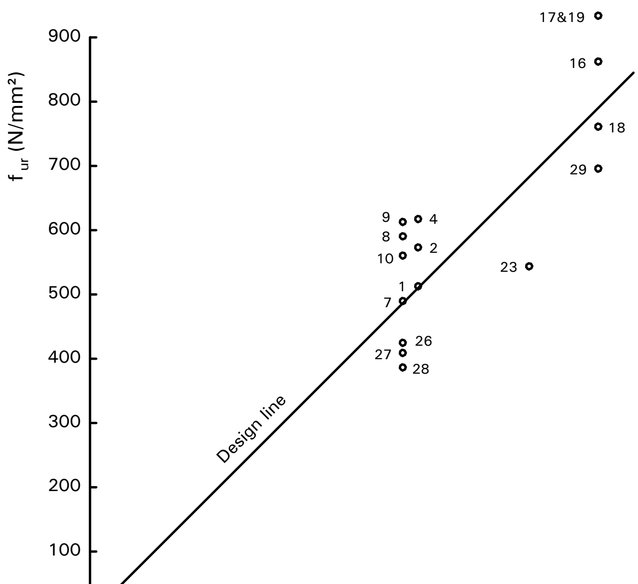


Figure C.6.3 *Comparison of f_{ur} design line and experimental data*

Errera et al [119,21]

An investigation into the connection behaviour of thin gauge (up to approximately 1,5 mm thick) grade 1.4310 ½ hard stainless steel sheets was undertaken. (½ hard relates to a temper or strength obtained by work hardening in cold rolling. It is used in America but not in Europe.) The results of specimens which were reported to have failed in mode I or mode II are shown in Figure C.6.4. Although the material exhibited pronounced anisotropy, and almost all specimens were single shear lap specimens so that mode VI would play a part, the results generally support the findings reported in the tests carried out for the First Edition of the Design Manual.

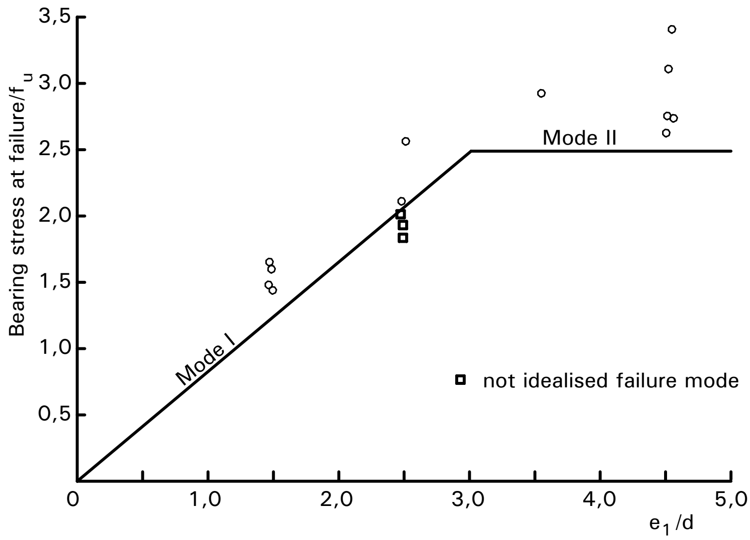


Figure C.6.4 *Comparison of experimental data from Cornell University data with design lines*[21]

van der Merwe [120]

Some 66 tests on bolted connections in ferritic stainless steels (types 1.4016, 1.4512 and 1.4003) are reported. Single shear and double shear specimens, with and without washers, were designed to fail in either mode I, II or III. Most specimens had one or two bolts, though three had four bolts. Insufficient information is given to permit graphical representation. However, all mode I specimens (30 in number) exceeded the design values (with γ = 1) by ratios ranging from unity (for the 4 bolt specimens) to 1,84, with a mean at 1,38. The mode II specimens with washers again exceeded the design values but reduced bearing strengths were found for specimens without washers. The mode III results are discussed under *Tension resistance,* below.

New test data from CTICM[121]

Tests were carried out on bolted cover plate connections in austenitic (grade 1.4306), duplex (grade 1.4462) and ferritic (grade 1.4016) stainless steels. Twelve cover plate connections with bolts in double shear were tested in each grade of steel. The number of bolts in the connections was varied from 2 (two different configurations) to 4. Table C.6.2 gives a summary of the test specimens and Table C.6.3 and Table C.6.4 gives the test results and predicted resistances for the tests on the austenitic and duplex grades.

The behaviour under test of this connection was typical of many of the other tests in austenitic and ferritic steels. Overall yielding of the gross section was first attained, followed by bearing failure. There was clear evidence of increased deformations occurring just beyond the predicted gross section yielding load. Final rupture was by bolt shear. The predicted ultimate net section resistance of the connection was exceeded when failure of the bolts occurred. The specimen however had suffered significant necking at the net sections prior to failure. The bolt holes were considerably ovalised, indicating that bearing also contributed to the overall deformations of the specimen. The outer plies (cover plates) showed a pronounced 'dishing' effect in the part beyond the end bolt, i.e. the plate bends out from the central ply.

It was generally observed in these tests that the actual ultimate resistance of the net section always exceeded the calculated value by a significant amount (sometimes 20% or more). This may be partially explained by the high ductility of the stainless steels used, but further investigation of the actual steel strengths may be advisable.

The test results showed that the design expressions for bearing are safe. Higher bearing resistances for drilled holes were measured than those for punched holes. It was proposed that 1,75 mm permanent deformation of a cover plate connection is acceptable at the serviceability limit state and a 5 mm permanent bearing deformation is acceptable at the ultimate limit state.

The test programme also confirmed that the design rules for austenitic and duplex stainless steel bolted joints could be applied to ferritic stainless steels.

Table C.6.2 *Summary of test specimens for the cover plate tests*[121]

Bolts	Holes	Connection Identification	e_1	p_1	d_1	e_2 mm	p_2	b	h
		A2L-12, F2L-12, D2L-12	22,5	45	55	22,5	-	45	190
M12x40	M14	A2T-12, F2T-12, D2T-12	22,5	-	55	22,5	45	90	100
		A3-12, F3-12, D3-12	22,5	45	55	22,5	45	90	190
		A4-12, F4-12, D4-12	22,5	45	55	22,5	45	90	190
		A2L-16, F2L-16, D2L-16	27,5	55	65	27,5	-	55	230
M16x50	M18	A2T-16, F2T-16, D2T-16	27,5	-	65	27,5	55	110	120
		A3-16, F3-16, D3-16	27,5	55	65	27,5	55	110	230
		A4-16, F4-16, D4-16	27,5	55	65	27,5	55	110	230
		A2L-20, F2L-20, D2L-20	35	70	80	35	-	70	290
M20x50	M22	A2T-20, F2T-20, D2T-20	35	-	80	35	70	140	150
		A3-20, F3-20, D3-20	35	70	80	35	70	140	290
		A4-20, F4-20, D4-20	35	80	80	35	70	140	290

Plate nominal thickness (mm):		Central plate	Cover plate
	Type A	10	5
	Type F	8	4
	Type D	12,5	6

Specimen numbering system:
The first letter indicates type of steel (A = austenitic, D = duplex, F = ferritic). The number following gives the number of bolts in the connection. For the 2 bolt specimens, L indicates the bolts are parallel to the load direction, T indicates they are transverse to the load. The final 2 numbers give the bolt diameter.

Table C.6.3 *Summary of test results and predicted resistances for cover plate tests on austenitic stainless steel* [121]

Steel:	Thickness.	Austenitic : nom = 5/10/5 mm ; meas. 5,2/9,85/5,2			
	Strength	Austenitic 1.4306 =304L: Measured f_y and f_u used			
	Test Config.	**A-2L**	**A-2T**	**A-3**	**A-4**
Bolt	**Element**	**Calculated Resistance in kN : Gamma = 1,0**			
	Bolt shear	174,6	174,6	261,9	349,2
	Bearing	158,3/200,6	158,3	237,5/306,1	316,7/401,1
	Gross sect.	122	243,9	243,9	243,9
12	Net section	162	322	322	322
	Serv.x1,5	126	252	252	252
	F_{u_Test}	173,9	179,4	269,6	345,6
	Failure Mode(s)	Yield<Net sect.< Bolt shear	Bearing< Bolt shear	Yield <Bearing? <Bolt shear	Yield <net sect <bolt shear
	Bolt shear	359,8	359,8	539,8	719,7
	Bearing	200,7/251,8	200,7	301/397,7	401,4/503,5
	Gross sect.	149,1	298,1	298,1	298,1
16	Net section	192,1	384,3	384,3	384,3
	Serv.x1,5	150	301	301	301
	F_{u_Test}	234,4	341,9	496	475,8
	Failure Mode(s)	Yield<Net section Central plate fail.	Bearing<yield <Bolt shear	Yield<Net.<Bear Ext. plate failure	Yield<Net sect. (not full rupture)
	Bolt shear	501	501	751,5	1002
	Bearing	261,2/330,3	261,2	391,8/507,5	522,5/660,5
	Gross sect.	189,7	379,4	379,4	379,4
20	Net section	249,3	498,5	498,5	498,5
	Serv.x1,5	195	390	390	390
	F_{u_Test}	297,1	444,5	583,6	580,9
	Failure Mode(s)	Yield<Net sect. Central plate fail.	Bear.<Yield (not full rupture)	Yield.<Bear<Net (not full rupture)	Yield<Net sect. (not full rupture)
Note:	Measured Yield and Tensile strengths used in N/mm^2				
			Yield 0,2%	Ult. Tensile	Elongation %
Steel Properties	5mm plate actual 5,2mm	Measured	271	577	63%
		Min specified	200	500	45%
	10mm plate actual 9,85 mm	Measured	288	581	62%
		Min specified	200	500	45%

Table C.6.4 *Summary of test results and predicted resistances for cover plate tests on duplex stainless steel*[121]

Steel:	Thicknesses	Duplex 1.4462 : 6/12,5/6 mm			
	Strength	Measured f_y and f_u used			
	Test Config.	D-2L	D-2T	D-3	D-4
Bolt	**Element**	**Resistance in kN :Calculated with gamma of unity**			
	Bolt shear	174,6	174,6	261,9	349,2
	Bearing	357,7	282,4	568,8	715,5
	Gross sect.	297	594	594	/594
12	Net section	255,1	364,3/510,2	510,2	510,2
	F_{u_Test}	192,2	188,7	280,3	373,9
	Failure Mode	Bolt shear	Bolt shear	Bolt shear	Bolt shear
	Bolt shear	359,8	359,8	539,8	719,7
	Bearing	449,1	358	739	898,2
	Gross sect.	363	726	726	726
16	Net section	304,5	609	609	434,8/609
	F_{u_Test}	357,9	361,7	534,3	581,3*
	Failure Mode	Net sect. <Bolt shear	Bear. <Bolt shear	Bolt shear	*Test ended Net section?
	Bolt shear	501	501	751,5	1002
	Bearing	589,1	465,9	943	1178,2
	Gross sect.	462	924	924	924
	Net section	395	790	790	790
20	F_{u_Test}	499	504,6	577,3*	580,9*
	Failure Mode	Net sect.<Yld Cl.Plate	Bear. <Bolt shear	*Test ended Bolt shear ?	*Test ended <Net sect.?
Note:	Measured Yield and Tensile strengths used in N/mm^2				
			Yield 0,2%	Ult. Tensile	Elongation %
Steel Properties	6mm plate actual 6,3	Measured	550	762	36%
		Min specified	460	640	25%
	12,5mm pl, Actual 12,85	Measured	590	770	not given
		Min specified	460	640	25%

The expressions for bearing resistance were modified slightly in the Third Edition of the Design Manual to align with the expressions in EN 1993-1-8.

Tension resistance

Two rules are provided for calculating the tensile resistance of connected parts in tension. The second of these limits the stress in the gross section to f_y in order to limit plastic deformation. Note that some plastic deformation (in fact 0,2%) would occur if the element were stressed to f_y.

The First Edition of the Design Manual gave the following expression for the ultimate resistance of the net cross-section at holes for fasteners:

$$N_{u,Rd} = \frac{0,9\ k_r\ A_{net}\ f_u}{\gamma_{M2}} \quad \text{where} \quad k_r = (1 - 0,9r + 3rd/s)$$

The k_r factor has long been used by the thin gauge cold formed design fraternity and relates to bolted connections with washers under both the bolt head and nut. The factor recognises the deleterious effect of the stress concentrations when the load in the member is taken out through a bolt (or bolts) as opposed to no load removal in a tension member just containing a hole. The latter situation may arise, for instance, in two diagonal bracings joined at their middle by a bolt (the bolt taking little or no load).

For a single bolt $r = 1$ and for two bolts (with the bolts aligned parallel to the direction of stress) $r = ½$, etc. The factor k_r is less than unity if d/s is 0,3 or smaller, no matter what value r may take.

The justification for using k_r for stainless steel is based on experimental data. In Table C.6.1, use of the k_r factor would change the experimental to predicted α ratios for the actual mode (eleventh column) for specimens 27, 28 and 29 from 0,86, 0,74 and 0,72 to 1,04, 0,99 and 0,96 respectively. These would then all be above unity when the 0,9 factor is introduced. Furthermore, with the 0,9 and k_r factors, specimens 27 and 28 would have been predicted to fail in the observed mode.

The net section failures of thin gauge type 1.4310 ½ hard stainless steel connections in the series of tests carried out by Errera et al[119,21] also indicate that the factor k_r is necessary, see Figure C.6.5. As can be seen, there is a substantial difference between the single shear and double shear specimens with 3 of the 4 single shear specimens falling below the characteristic design line. These low single shear results are all due to the effects of distortion in the thin gauge specimens and in fact are governed by failure mode VI (see Figure C.6.1). As such, it would be inappropriate to use these results to justify a lowering of the design curve. The double shear specimens, for which mode VI is not a possibility, corroborate the design recommendations for net failure.

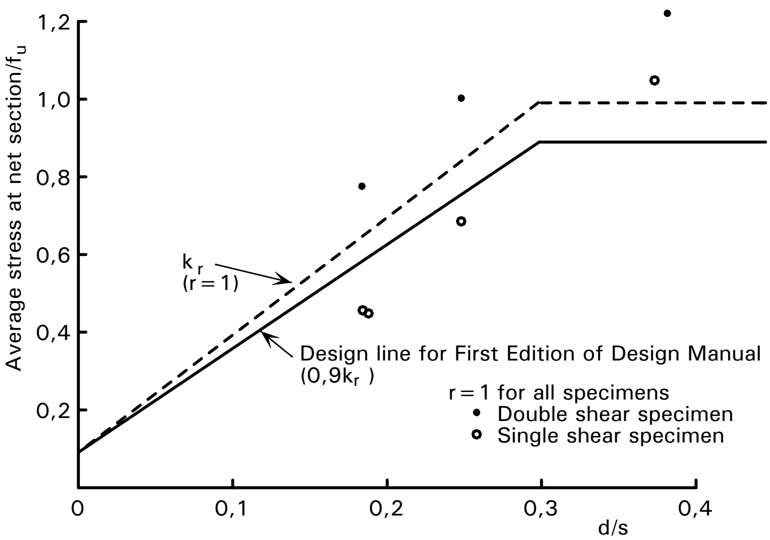


Figure C.6.5 *Cornell University data for net section failure*[21]

Figure C.6.6 shows the results for connections in ferritic stainless steel for specimens having washers under both bolt head and nut[120]. All but one specimen had two bolts, for which $r = 0,5$. Again, a reduction in load is indicated for d/s values below 0,3.

However, the recent tests reported by Ryan[121] do not justify the retention of the 0,9 factor in addition to the k_r factor in the expression for the ultimate resistance of the net cross-section. The equivalent expression in EN 1993-1-1 includes the 0,9 factor, but not the k_r term in the expression. Conversely, EN 1993-1-3 includes the k_r term but does not include the 0,9 factor. The tests reported in Reference 118 indicated that the 0,9 factor may not be needed, although it was retained to maintain compatibility with EN 1993-1-1, account for variables such as strain rate effects and limit gross deformation at the net section. It was therefore concluded during the drafting of the Second Edition of the Design Manual that there was sufficient evidence for removing the 0,9 factor from the expression for the ultimate resistance of the net cross-section. In the Second

Edition, the expression for k_r was re-arranged and modified slightly to align with the definition given in EN 1993-1-3 which includes the bolt hole diameter as opposed to the bolt diameter.

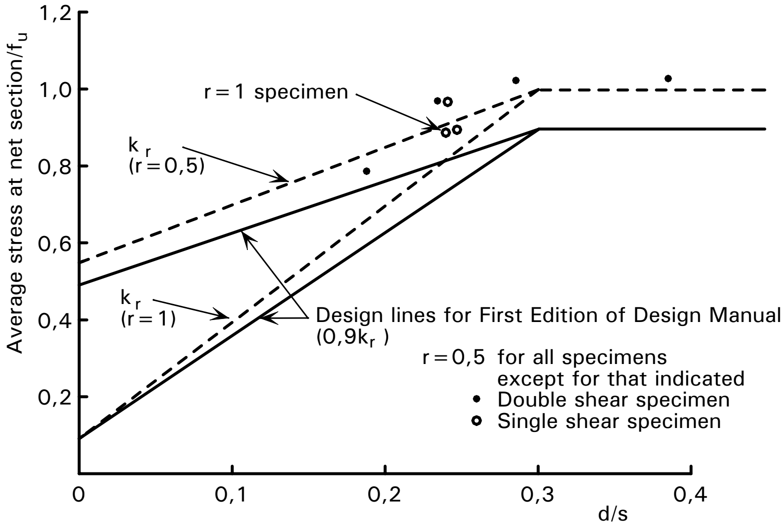


Figure C.6.6 *Net section failure data by van der Merwe [120]*

Design for block tearing

There are no specific test data for stainless steel in shear rupture. Guidance in EN 1993-1-8 is recommended.

Angles connected by one leg and other unsymmetrically connected members in tension

Ryan[121] tested bolted gusset plate connections to angle and tee sections in austenitic stainless steel. The twelve test specimens comprised tee or angle sections connected to one side of a gusset plate by bolts acting in single shear. Each specimen consisted of a 1500 mm length of section with a gusset plate bolted at each end. The specimens were loaded by applying tension to the gusset plates. The plates and sections were austenitic stainless steel grades 1.4306 and 1.4307. Some of the sections were cut from UPN type, I-sections or rectangular hollow sections. The number of bolts (A4 property class 80) in the connections varied from 4 to 8. Table C.6.5 gives a summary of the test specimens.

The test results were compared to the resistances predicted by two methods:

ENV 1993-1-4 and First Edition of the Design Manual

This approach allows the member to be treated as concentrically loaded provided the following expression for A_{net} is used to calculate the net section resistance:

A_{net} = net area of connected leg + 0,5 gross area of shorter leg

ENV 1993-1-1

This approach calculates an effective concentrically loaded section that depends on the net section of the entire angle and on the number and spacing of the bolts. The rule is given in full in Section 6.2.3, Equations 6.6, 6.7 and 6.8.

Table C.6.6 summarises the test results and predicted resistances. For the reinforced angle connections, the following modifications were made to the ENV 1993-1-1 and ENV 1993-1-4 rules to allow for the fact that the outstand leg was also attached to the gusset via a cleat:

- In the ENV 1993-1-1 method, the β factor was taken as 0,7 (the highest value proposed for angles attached by one leg).
- In the ENV 1993-1-4 method, the efficiency of the outstanding leg area was increased from 50% to 70%.

Table C.6.5 *Test specimens for gusset plate tests*[121]

Test	Section	Bolts	Remark
C1-6/20	Angle 100x100x10	6 M20-1 line	Staggered row of bolts
CR1-4/20	Angle 100x100x10	Line of 2 M20 on each leg	Reinforced by a cleat Cleat with 2M20 to gusset
CR2-6/20	Angle 100x100x10	Line of 3 M20 on each leg	Reinforced by a cleat Cleat with 3M20 to gusset
UC1-4/20	Angle 80x65	4 M20 - 1 line	Section cut from UPN 160x65 -65mm leg bolted
UCR1-4/20	Angle 80x65	Line of 2 M20 on each leg	Section cut from UPN 160x85 Cleat with 2M20 to gusset
UCR2-6/20	Angle 80x65	Line of 3 M20 on each leg	Cut from UPN 160x85 Cleat with 3M20 to gusset
RC1-3/16	Angle 110x50x4	3 M16 - 1 line	Section cut from 120x60x4 rect.tube- 110mm leg bolted
RC2-4/16	Angle 110x50x4	2 lines of 2 M16 on attached leg	Section cut from 120x60x4 rect.tube- 110mm leg bolted
T1-6/12	T 100x100x10	2 lines of 3 M12	Flange bolted
T2-8/12	T 100x100x10	2 lines of 4 M12	Flange bolted
IT-4/12	T 80x82	2 lines of2 M12	Cut from I 160x82 82 mm flange bolted
IT-8/12	T 80x82	2 lines of 4 M12	Cut from I 160x82 82 mm flange bolted

Table C.6.6 *Summary of test results and predicted resistances for gusset plate tests*[121]

	Predicted ultimate resistances kN				Comparison of ENV 1993-1-4 with tests results			
Test	Bearing	Bolt shear	Net** Section Part 1-4	Net** Section Part 1-1	Pred.*** Resist.kN Part 1-4	Test kN	Ratio of Test/Pred. for actual failure modes	Failure mode
C1	803,5	751,5	755,8	514,1	751,5	642	0,854**	Net section
CR1	559,7	501	773,0*	626,5*	501	322	0,643	Bolt shear
CR2	839,5	751,5	773,0*	626,5*	751,5	556,7	0,741	Bolt shear
UC1	629,2	501	448,7	298,2	448,7	441,8	0,985**	Net section
UCR1	629,2	501	438,1*	322,2*	438,1*	339,5	0,678	Bolt shear
UCR2	943,8	751,5	431,2*	320,4*	431,2*	514,9	1,194**	Net section
RC1	177,0	269,9	376,7	245,4	177/269,9	275	1,554/1,019	Bearing/bolt shear
RC2	236,0	359,8	317,5	274,1*	236/317,5	308,6	1,308/0,972 **	Bearing/Net sect.
T1	489,5	261,9	642,7	499,7	261,9	274,9	1,050	Bolt shear
T2	652,6	349,2	642,7	499,7	349,2	356,2	1,020	Bolt shear
IT1	328,0	174,6	620,4	400,4	174,6	162,2	0,929	Bolt shear
IT2	655,9	349,2	623,6	462,8	349,2	369,8	1,059	Bolt shear

Notes :
* By modified ENV rule
** Without 0,9 factor
*** Not necessarily for the failure mode

The test results showed that the guidance in ENV 1993-1-4 may be unsafe and would lead to excessively large permanent deformations in some cases, possibly even at the serviceability limit state. The ENV 1993-1-4 guidance always gives higher resistances than the guidance in ENV 1993-1-1, in particular for many standard angles with short connections, for which it predicts resistances up to twice those predicted by the ENV 1993-1-1 guidance. However, the guidance in ENV 1993-1-1 showed acceptable agreement with the test results, although it did not always properly account for the length of the connection. It therefore replaces the ENV 1993-1-4 rule in the Second Edition of the Design Manual.

The expression for β_2 was modified slightly in the Third Edition to ensure alignment with EN 1993-1-8.

C.6.2.4 Fasteners

Net areas

The tensile stress areas for stainless steel bolts to EN ISO 3506[8] are set out in Table C.6.7.

Table C.6.7 *Tensile stress area for bolts to EN ISO 3506*

Thread Size (Coarse Series)	Stress Area, A_s (mm^2)
M6	20,1
M8	36,6
M10	58,0
M12	84,3
M16	157,0
M20	245,0
M24	353,0
M30	561,0
M36	817,0

Shear, tension and shear/tension resistance

The recommendations given in these sections are all similar to rules given in EN 1993-1-8 for common structural bolts.

A limited test programme on the strength of stainless steel bolts was conducted to generate information for the First Edition of the Design Manual[118]. The number and type of tests are set out in Table C.6.8 and the results are summarised in Figure C.6.7.

Table C.6.8 *Number of tests on stainless steel bolts*

Fastener (set screws)	Key to Fig C.6.7	Supplier								
		A			B			C		
		T	S	T/S	T	S	T/S	T	S	T/S
M20, A4-80	□	8	8	8	5	5	5	8	3	2
M16, A4-80	□	8	7		5	5		2		
M16, A4-70	□							6	2	

Notes:
T = Tension test, S = double shear test
T/S = Combined tension and shear

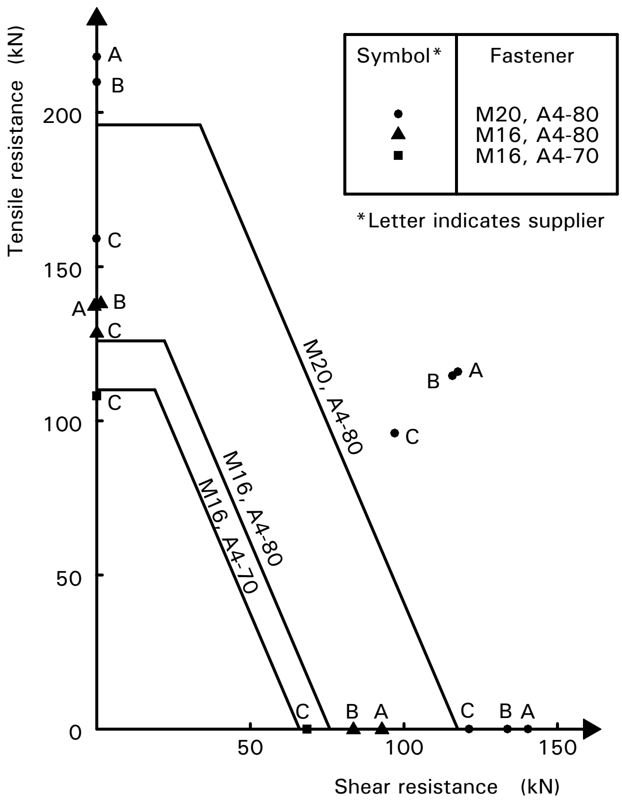


Figure C.6.7 *Results of tests on stainless steel bolts*

The lines of Figure C.6.7 correspond to the interaction formula given in Section 6.2.4 *Combined shear and tension* but using the specified tensile capacity[8] for the ordinate and 0,6 times that for the abscissa. (The provision in 6.2.4 *Tensile resistance* applies a 0,9 scaling factor to the ordinate.)

Given any particular batch and test type, the results were remarkably consistent, typically within $\pm$ 2kN for tension and $\pm$ 5kN for shear about the respective averages. Tests at loading rates differing by an order of magnitude showed little effect on failure loads.

In one instance (supplier C of M20 A4-80 set screws), the measured tensile capacity is less than the minimum specified level. This led to the recommendation in 3.1.2 to have samples independently tested. Nevertheless, even this batch was satisfactory in pure shear and in combined tension and shear (for the ratio tested, $T = S$).

More recently, Ryan[121] carried out tension and shear tests on individual bolt/nut assemblies. Some of the shear tests were carried out with the plates loaded in tension, and some in compression. Bolt diameters M12, M16 and M20 were tested; all the bolts were austenitic A4 property class 80 to EN ISO 3506. The results of the bolt tension and shear tests showed good agreement with the predicted values.

Long joints and large grip lengths

The shear flow in long joints is such that the fasteners at each end take more shear than the average shear of all the fasteners. Since stainless steel is more ductile than carbon steel, and hence permits a greater degree of force redistribution, stainless steel long joints should be at least of equal performance to those in carbon steel.

Likewise, there is no reason to think stainless steel bolts with large grip lengths behave any worse than normal structural bolts.

C.6.3 Mechanical fasteners for thin gauge material

In general, the guidance in EN 1993-1-3 has been shown to be applicable to annealed and cold worked stainless steel[22].

C.6.4 Welded connections

The Recommendations adopt the approach for determining the strength of a fillet weld for carbon steel given in EN 1993-1-8. Additionally, recommendations are given against the use of intermittent welds and partial penetration butt welds in certain circumstances, to reduce the potential for corrosion.

The provisions are primarily intended for sheet and plate of 4 mm thickness and over.

It is important that good quality welds are made using verified procedures, see Section 10.4, for the provisions to be realised.

Tests by SCI for the First Edition of the Design Manual

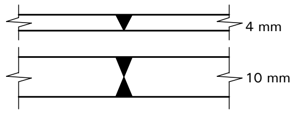
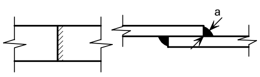
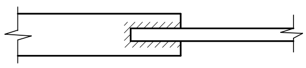
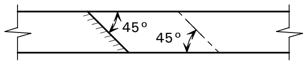
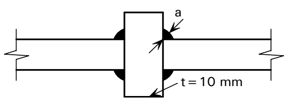
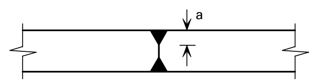
Since there were no available data on welded joints in the relevant grades of stainless steel when the First Edition of the Design Manual was being written, a limited test programme was conducted[118]. The fifteen specimens included a variety of different types of joints as shown in Table C.6.9.

Only full penetration butt weld tests were carried out on grades 1.4307 and 1.4404 material. Welds in duplex 1.4462 material were tested in all configurations. All specimens were prepared individually, without using run-off tabs, so that starting and finishing defects would be present. The results generally confirm the assertion that the strength of a weld may be considered as equal to the parent material. The lowest ratio of measured failure load to predicted failure load is 0,91 for specimen 7. Some of this discrepancy may be attributed to strain rate effects, as some specimens that failed away from the weld only reached a ratio of 0,95.

Tests on grade 1.4310 and ferritic stainless steels

References 119, 21 and 120 contain results of weld test programmes. References 119 and 21 report on a test programme on ¼ hard and ½ hard 1.4310 stainless steel and Reference 120 reports on ferritic stainless steels. The tests on the 1.4310 material show that the welding process has a partial annealing effect on the cold worked stainless steel with a consequent reduction in the cold worked strength.

Table C.6.9 *Welded connection test programme*

Specimen Number	Steel Grade	*t* (mm)	Weld Throat *a* (mm)	Measured Load / Predicted Load	Schematic
1	1.4307	4,2	Full Pen.	0,97	
2	1.4307	10,4	Full Pen.	0,95	
3	1.4404	4,2	Full Pen.	1,03	
4	1.4404	10,4	Full Pen.	0,97	
5	1.4462	2,0	1,4	1,00	
6	1.4462	10,6	7,1	0,92	
7	1.4462	2,0	1,4	0,91	
8	1.4462	10,6	7,1	-	
9	1.4462	2,0	1,4	0,99	
10	1.4462	10,6	7,1	0,96	
11	1.4462	10,6	5,0	1,08	
12	1.4462	10,6	3,9	1,02	
13	1.4462	10,6	2,6	1,09	
14	1.4462	10,6	3,5	1,06	
15	1.4462	10,6	Full Pen.	0,96	

Tests by RWTH

More recently, 46 stainless steel fillet welded connections were tested at RWTH[122]. The test programme comprised 22 single lap joints with welds parallel to the loading direction, and 24 double lap joints with welds transverse to the loading direction. Two different base material grades with two electrode material grades were tested: grade 1.4301 base material with grade 1.4316 electrodes, and grade 1.4462 base material with grade 1.4462 electrodes. Tensile coupon tests in accordance with EN 10002-1[123] were conducted on the base material (with coupons prepared both transversely and longitudinally to rolling direction), on the electrode material, and on the actual weld material.

The approach for determining the strength of a fillet weld in the Recommendations is that for carbon steel given in EN 1993-1-1, but with the correlation factor, β_w set to 1,0 for all grades of stainless steel. For the RWTH tests, the ratios of experimental failure load to predicted failure load were calculated. These ratios were much higher for welds transverse to the load direction (varying from 1,42 to 1,69) than for those parallel to the load direction (varying from 1,01 to 1,12). This behaviour is also apparent with carbon steels. In EN 1993-1-1 different β_w values are given for different grades of carbon steel and β_w is independent of the weld configuration. Statistical analysis of the SCI and RWTH test results concluded that $\beta_w = 1,0$ should be used for determining the resistance of stainless steel fillet welded connections[122]. This approach is very conservative for welds that are transverse to the direction of loading, but economic for longitudinal welds.

Welding cold worked stainless steel

References 22 and 124 describe experimental studies of the structural behaviour of welds in cold worked material and confirm the design approach given in Section 6.4.4.

C.7 FIRE RESISTANT DESIGN

C.7.1 General

Guidance in this Section follows that given in EN 1993-1-2 except where highlighted in Section C.7.4. Annex C of EN 1993-1-2 gives stainless steel properties at elevated temperatures. For the purposes of design, it is assumed that the actions are taken from EN 1991-1-2[125].

C.7.2 Mechanical properties at elevated temperatures

The derivation of the mechanical properties at elevated temperatures given in the Recommendations is fully reported by Zhao[126]. Tests were carried out on five grades of stainless steel: 1.4301, 1.4401, 1.4571, 1.4462 and 1.4003. The test programme consisted of tensile tests at room temperature, isothermal tests at elevated temperature and anisothermal tests at elevated temperatures. Based on the test results, a mathematical model was generated for describing the stress-strain relationship for stainless steel at elevated temperatures. The model is closely aligned to that for carbon steel in EN 1993-1-2 and is divided into two non-linear parts, (strains from zero to e_c, and strains from e_c to e_u), as shown in Figure C.7.1 and Table C.7.1. From the model, the strength and stiffness retention factors were generated for the grades tested (given in Table 7.1 of the Recommendations).

The advantages of this model are that it gives an accurate prediction of the stress-strain relationships of stainless steel at elevated temperatures, whilst remaining compatible with the familiar carbon steel model from EN 1993-1-2.

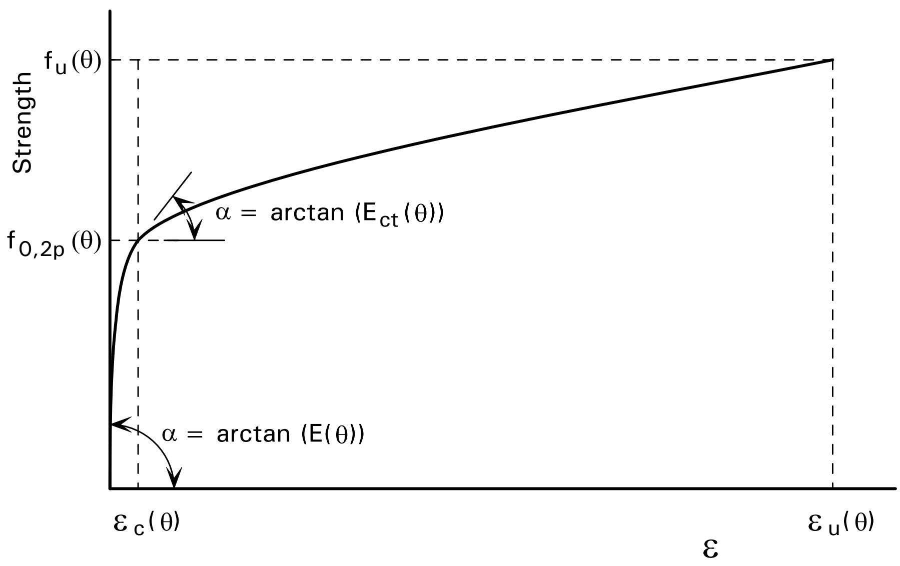


Figure C.7.1 *Definition of stress-strain parameters*

Table C.7.1 *Mathematical stress-strain model*

Strain range ε	Stress σ	Tangent Modulus E_t
$\varepsilon \le \varepsilon_c$	$\dfrac{E\ \varepsilon}{1 + a\varepsilon^b}$	$\dfrac{E\ (1 + a\varepsilon^b - ab\varepsilon^b)}{(1 + a\varepsilon^b)^2}$
$\varepsilon_c \le \varepsilon \le \varepsilon_u$	$f_{0,2p} - e + \dfrac{d}{c}\sqrt{c^2 - (\varepsilon_u - \varepsilon)^2}$	$\dfrac{d + (\varepsilon_u - \varepsilon)}{c\sqrt{c^2 - (\varepsilon_u - \varepsilon)^2}}$

where:

$$a = \frac{(E\ \varepsilon_c - f_{0,2p})}{f_{0,2p}\ \varepsilon_c{}^b} \qquad b = \frac{(1 - E_{ct}\ \varepsilon_c / f_{0,2p})}{(E\ \varepsilon_c / f_{0,2p} - 1)} \frac{E\ \varepsilon_c}{f_{0,2p}}$$

$$c^2 = (\varepsilon_u - \varepsilon_c)\left(\varepsilon_u - \varepsilon_c + \frac{e}{E_{ct}}\right) \qquad d^2 = e\ (\varepsilon_u - \varepsilon_c)\ E_{ct} + e^2$$

$$e = \frac{(f_u - f_{0,2p})^2}{(\varepsilon_u - \varepsilon_c)\ E_{ct} - 2(f_u - f_{0,2p})} \qquad \text{with } \varepsilon_c = f_{0,2p} / E + 0{,}002$$

$f_{u,\theta}$ is the tensile strength at temperature θ
$f_{0,2p,\theta}$ is the 0,2% proof strength at temperature θ
E_θ is the slope of the linear elastic range
$E_{ct,\theta}$ is the slope at the 0,2% proof strength
$\varepsilon_{c,\theta}$ is the total strain at the 0,2% proof strength
$\varepsilon_{u,\theta}$ is the ultimate strain

Tensile tests on cold formed stainless steel at elevated temperatures reported by Ala-Outinen[127] showed that the increased strength due to the cold forming process remains constant up to 600°C, after which the strength begins to decrease and the influence of cold forming totally disappears at 900°C.

Table 7.1 was extended in the Third Edition of the Design Manual to cover grade 1.4318 (annealed and C850) and grade 1.4571 (C850) based on tests carried out and reported in Reference 22.

C.7.3 Thermal properties at elevated temperatures

The thermal properties are those given in EN 1993-1-2. A comparison of the thermal properties of stainless steels with those of carbon steels is given by Ala-Outinen[127]. Note that the thermal elongation values apply to austenitic stainless steel and not to duplex grades. Data for duplex grades should be sought from a stainless steel producer.

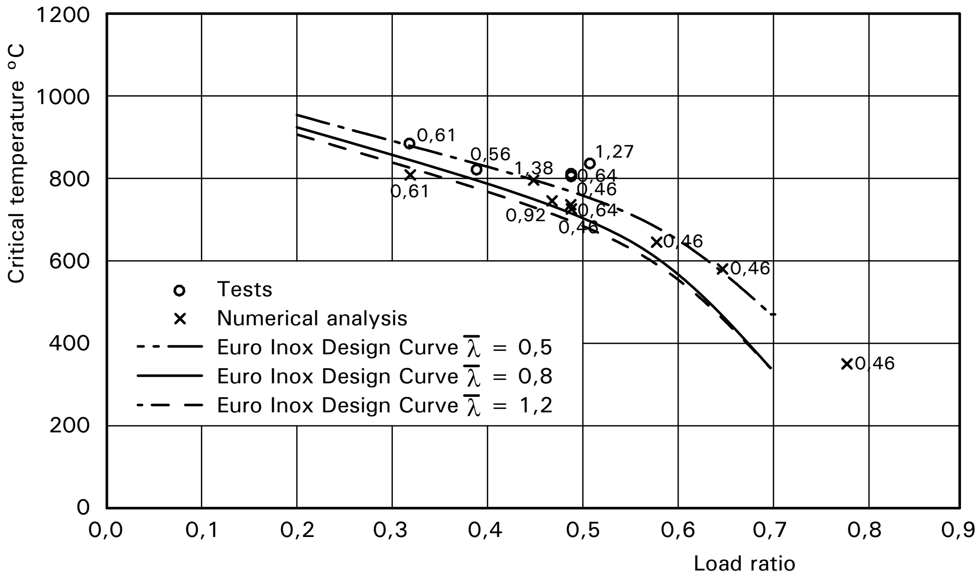
C.7.4 Determination of structural fire resistance

The behaviour of unprotected stainless steel members was first studied by Ala-Outinen and Oksanen[128]. They tested six 40x40x4 mm RHS columns in grade 1.4301 stainless steel with a buckling length of approximately 890 mm. They also studied the behaviour of butt welded joints at elevated temperatures, concluding that the joints did not have an adverse effect on the behaviour of the member in fire.

The behaviour of unprotected stainless steel beams and columns in fire was studied by Baddoo and Gardner[129]. Fire tests were carried out on six stainless steel columns and four stainless steel beams. All the members were grade 1.4301 stainless steel. Four of the columns were fixed and two were pinned.

Three of the beams were simply supported and one was continuous over two spans. The fire tests on four of the columns and two of the beams were subsequently modelled using finite element analysis. Reasonably good agreement was obtained between the test results and numerical analysis. A parametric study analysed the effects of varying the overall slenderness of columns, the load ratio (the applied load divided by the room temperature resistance) and the cross-sectional slenderness.

Using the material properties for stainless steel derived in Reference 126, design guidance for carbon steel in EN 1993-1-2 was shown to be applicable to stainless steel columns (cold formed open and hollow cross-sections only) and stainless steel beams supporting a concrete slab. Figure C.7.2 shows the column design curves against the results of the tests and numerical analyses. (A family of design curves is needed because the critical temperature is a function of both load ratio and non-dimensional slenderness, $\bar{\lambda}$.) Figure C.7.3 shows the beam design curves against the results of the tests and numerical analyses.



Note: Values of $\bar{\lambda}$ given next to each test and FE point

Figure C.7.2 *Test data, results of numerical analyses and design curve for stainless steel columns*

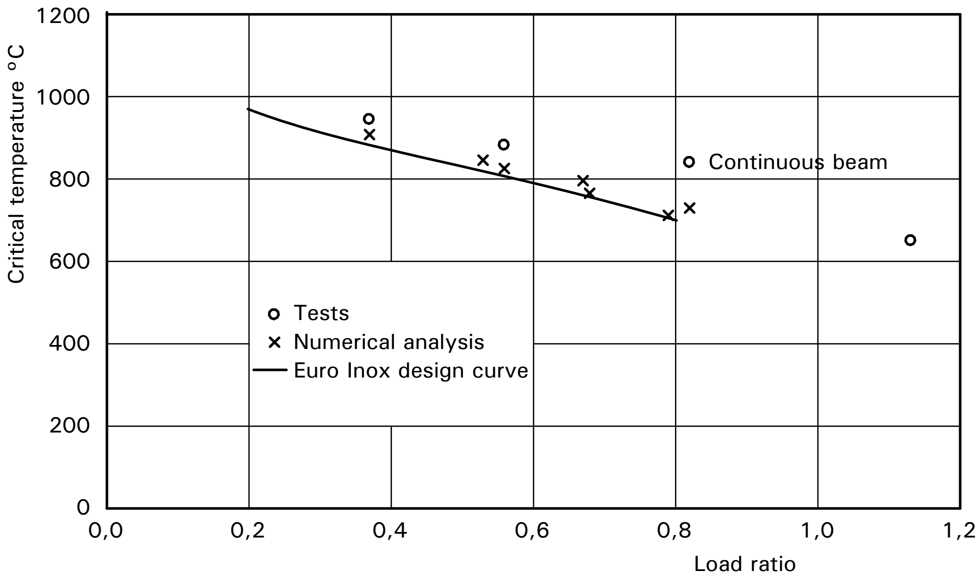


Figure C.7.3 *Test data, results of numerical analyses and design curve for stainless steel beams*

However, for the column tested with a welded, open cross-section, this design approach was found to over-estimate the measured resistance and further work is necessary before definitive design recommendations can be made for stainless steel columns with this type of cross-section.

Note that EN 1993-1-2 uses the characteristic strength at 2% total strain to determine the structural fire resistance for all modes of loading of steel members with Classes 1, 2 and 3 cross-sections. This was the approach adopted in the Second Edition of the Design Manual for stainless steel. Fire tests on structural members indicate that the strain at failure is strongly dependent on the loading mode. For example, beams supporting concrete or composite floors experience very high strains at failure (>2%). Load tests on columns show they behave rather differently in that relatively low strains are experienced at failure. It is conventional Eurocode practice to use empirical factors to adjust the applied loads (or the resistances) to obtain better agreement between tests and the basic material properties at 2% strain.

However, from the results of fire tests on members made from cold worked stainless steel, a less conservative design approach was derived based on the 0,2% proof strength for all cross-section classes and using the room temperature buckling curves rather than the fire buckling curves derived for carbon steel[22].

These two approaches to fire resistant design were compared against all available test data from stainless steel fire tests. Generally, the method in the Second Edition of the Design Manual gave slightly more conservative results than the new method, although there is not a huge difference between the design curves (Figure C.7.4 and Figure C.7.5). (Note: In these Figures the method in the Second Edition is called 'Euro Inox', and the new method is called 'CTICM'.) The Design Manual method is a little more complicated because it involves the evaluation of the stress reduction factor at a total elongation (elastic and plastic) equal to 2% ($k_{2\%,\theta}$) which implies the knowledge of the actual value of f_u, while the new method does not. The new method is not sensitive to f_u at all.

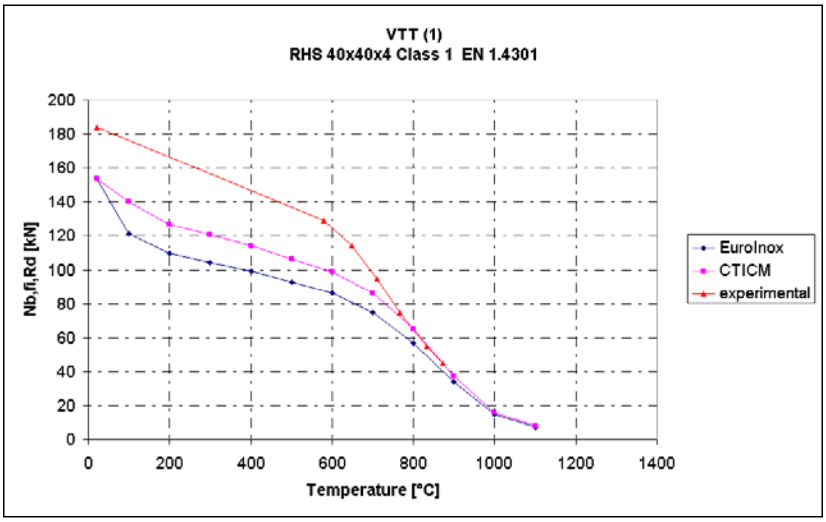


Figure C.7.4 *Column buckling tests at elevated temperature from VTT: Comparison between Euro Inox method (from 2nd Ed of Design Manual) and new approach (CTICM): grade 1.4301*

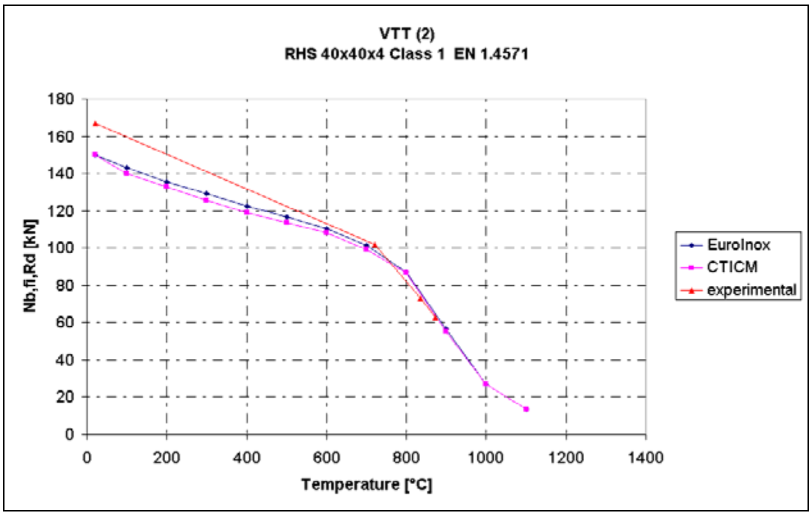


Figure C.7.5 *Column buckling tests at elevated temperature from VTT: Comparison between Euro Inox method (from 2nd Ed of Design Manual) and new approach (CTICM): grade 1.4571*

Design curves in EN 1993-1-2 and EN 1994-1-2 were derived by the relevant Project Teams from a 'mean' assessment of the predictions against the test data points with no further reliability statistical analysis. Assuming a 'mean' assessment gives an acceptable level of safety, the new approach gives an adequately safe prediction of the behaviour of stainless steel columns in fire. It was therefore decided to adopt this approach in this project and the forthcoming Third Edition of the *Design Manual for Structural Stainless Steel*.

The approach is summarised in Table C.7.2. It represents advances in understanding of the behaviour of stainless steel members in fire and is less conservative than the approach in EN 1993-1-2.

Table C.7.2 *New approach for fire resistant design*

Member	Strength and buckling curve for use in design
Columns	$f_{0,2\text{proof},\theta}$ (all cross-section classes) and the appropriate room temperature buckling curve
Restrained beams	$f_{2,\theta}$ (class 1-3) and $f_{0,2\text{proof},\theta}$ (class 4)
Unrestrained beams	$f_{0,2\text{proof},\theta}$ (all cross-section classes) and the appropriate room temperature lateral torsional buckling curve
Tension members	$f_{2,\theta}$ (all cross-section classes)

This is the only part of the Design Manual which deviates from the recommendations in Eurocode 3.

Although there are no relevant test data, the Recommendations also give guidance on the:

- shear resistance,
- lateral torsional buckling resistance and
- resistance to combined axial compression and bending

of stainless steel members in fire, based on the recommendations for carbon steel in EN 1991-1-2.

Section 7.4.7 of the Recommendations is the approach given for carbon steels in EN 1993-1-2.

The heating up characteristics of a range of stainless steel sections with section factors varying from around 200 m^{-1} to 700 m^{-1} were studied in a test programme[129]. Numerical modelling agreed well with the tests. Furthermore, it was shown that for a given section factor, a stainless steel section heats up at a very similar rate to a carbon steel section. Further studies were carried out by Gardner and Ng[130] in which the recommendation was made that a value of 02 was more appropriate for the emissivity of stainless steel than the value of 0,4 given in EN 1993-1-2.

For advanced calculation methods, the guidance given in EN 1993-1-2 can be followed.

C.8 FATIGUE

C.8.1 Introduction

Austenitic and duplex stainless steels are widely used in the fabrication of structures that are subjected to repeated loading and must therefore be designed to avoid fatigue failure. Many fatigue data exist for welded joints in structural carbon steels[131]. There is also an increasing body of stainless steel data[132,133,134].

Fatigue behaviour of welded joints is dominated by joint geometry. Similar crack growth behaviour occurs in carbon and stainless steel. The test data show that welded joints in stainless steel have fatigue strengths very similar to those in carbon steels and well established design rules for carbon steels are applicable to stainless steels.

The guidance on fatigue strengths apply to structures operating under normal atmospheric conditions and with sufficient corrosion protection and regular maintenance. The effect of seawater corrosion is not covered. Microstructural damage from high temperatures (>150°C) is not covered. Furthermore, almost all the fatigue tests on stainless steel joints which were found in the literature had been performed in air. In the presence of a corrosive environment, fatigue strength is reduced, the magnitude of reduction depending on materials, environment, loading frequency etc. The effect of sea water on carbon steel, which has been most widely investigated, is to reduce fatigue life by a factor of 2 under freely corroding conditions.

The fatigue strength of welded joints is usually determined by a fatigue life-stress range curve, a so-called *S-N* curve, which is presented as a log-log graph. In Eurocode terminology, *S-N* curves are known as $\Delta\sigma_R$-N_R curves. A fatigue strength curve is applied to each detail category. Each detail category is designated by a number that represents, in N/mm^2, the stress range that corresponds to a fatigue strength of 2 million cycles. For example, a joint assigned a detail category 80 (also designated FAT 80) would have a fatigue life of 2 million cycles when subject to a constant amplitude stress range of 80 N/mm^2.

C.8.2 S-N data for stainless steels

In a recent test programme, fatigue strengths of shielded metal arc welded (SMAW) joints from stainless steel grades 1.4301, 1.4436 and 1.4462 were determined using constant amplitude loading and axial tension fatigue tests[135]. Results from more than 50 test specimens were analysed. Test specimens were longitudinal and transverse non-load carrying fillet welds. As there are no standard S-N curves for stainless steel, the test results were compared to the carbon steel fatigue class given in EN 1993-1-9 (FAT 80 for transverse fillet welds and FAT 71 for longitudinal fillet welds). The results are shown in Figure C.8.1 and Figure C.8.2. The test results were analysed to determine the characteristic fatigue class $FAT_{95\%}$ (the stress range at a 95% survival probability). The characteristic fatigue classes exceeded the carbon steel standard classes.

Other test programmes generally support this behaviour[136,137,138] although some test programmes have shown the class of austenitic stainless steel longitudinal fillet welds to be slightly lower than that of carbon steel[139,140]. However, more recent studies have not confirmed this (Figure C.8.3), throwing some doubt on the earlier sets of results, both of which happened to be obtained at the same laboratory in the 1970s. Thus the general trend is to apply fatigue design rules for carbon steels to welded stainless steels (excluding environmental considerations)[137]. This is the approach adopted in EN 1993-1-9.

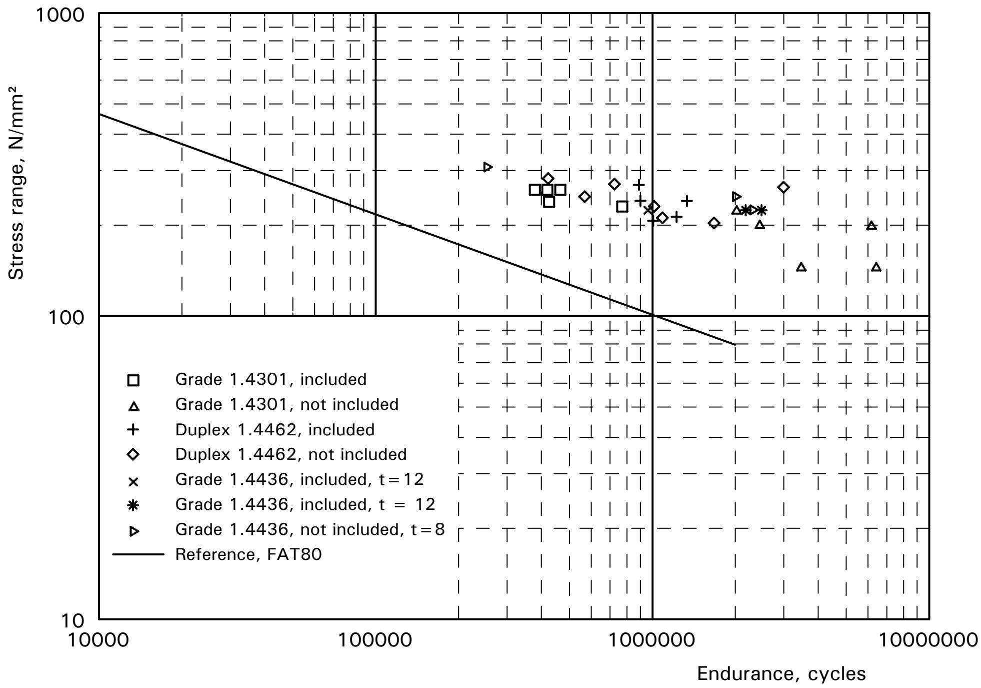


Figure C.8.1 *Fatigue endurance data for transverse fillet welds (grades 1.4301, 1.4436 and 1.4462)*

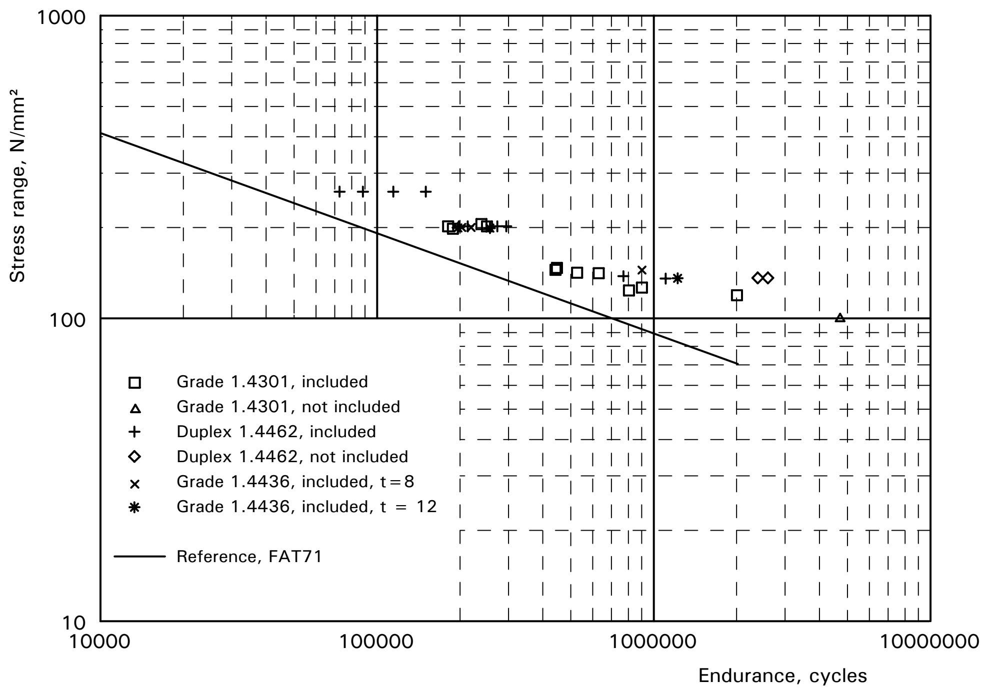
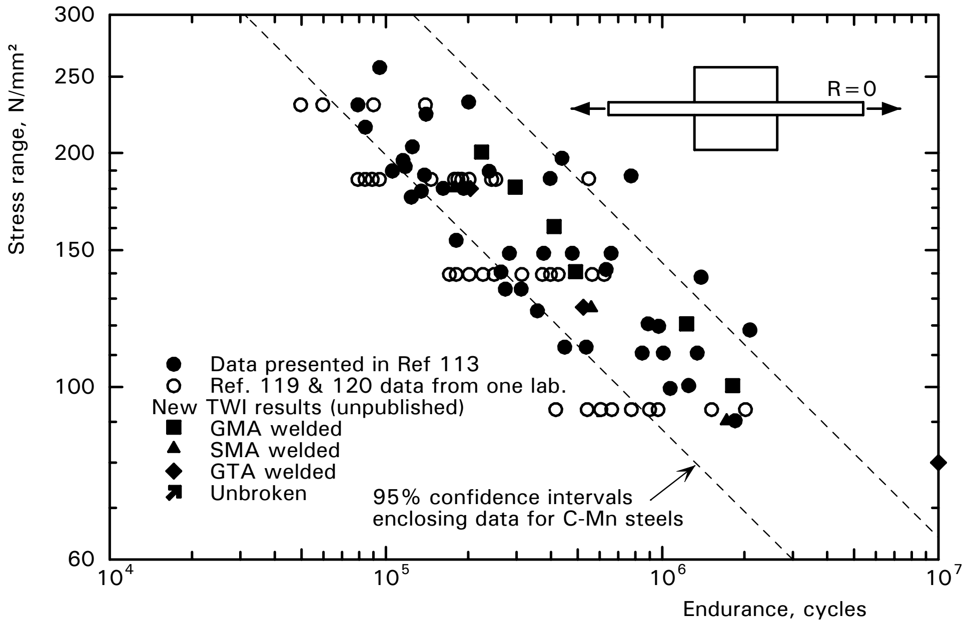


Figure C.8.2 *Fatigue endurance data for longitudinal fillet welds (grades 1.4301, 1.4436 and 1.4462)*



Note: 95% confidence intervals are taken from Reference 141

Figure C.8.3 *Fatigue test results for austenitic stainless steel plates with longitudinal fillet welded attachments*[137]

C.8.3 S-N data for cold worked stainless steels

Reference 22 describes fatigue tests on intermittent and continuous longitudinal load-carrying fillet welds on grade 1.4318 stainless steel cold worked to strength level C850. The study concluded that the guidance in EN 1993-1-9 can safely be applied to cold worked stainless steel, in fact the resistance of the cold worked joints was considerably higher than the relevant Eurocode classifications.

C.8.4 Fatigue crack growth data for stainless steels

An alternative method, although less commonly used, for fatigue assessment is the fracture mechanics approach. It is based on the observed relationship between the range in the stress intensity factor, ΔK, and the rate of growth of fatigue cracks, da/dN. This usually takes a sigmoidal form in a log ΔK versus log da/dN plot. Below a threshold stress intensity factor range, ΔK_{th}, no crack growth occurs. For intermediate values of ΔK, growth rate is idealised by a straight line in the log/log plot such that:

$$\frac{da}{dN} = C\,(\Delta K)^n$$

For a crack at the toe of a welded joint:

$$\Delta K = M_k\, Y\, \Delta S \sqrt{\pi a}$$

where

ΔS is the applied stress range,

a is the crack depth,

Y is a correction function dependent on crack size, shape and loading

M_k is a special function that allows for the stress concentration effect of the welded joint and depends on crack size, plate thickness, joint geometry and loading.

Solutions for Y for semi-elliptical cracks of the type which occur at the toes of welds and solutions for M_k for a range of welded joint geometries are available.

Combining the above two equations and integrating gives:

$$\int_{a_i}^{a_f} \frac{da}{\left(M_k Y \sqrt{\pi a}\right)^n} = C\Delta S^n N$$

where

a_i is the initial crack depth

a_f is the final crack depth corresponding to failure

Thus, if a welded joint contains a crack or crack-like flaw, its fatigue life can be predicted assuming that the life consists of fatigue crack growth from the pre-existing crack, if the initial crack size is known.

Following a review of data pertaining to the fatigue crack growth behaviour of stainless steels, values of C and n are given in Table C.8.1. It is recommended to use a ΔK_{th} value of 63,2 N/mm$^{3/2}$ (2MN/m$^{3/2}$) for all grades of stainless steel.

Table C.8.1 *Values for C and n (in air)*

R Range	C			n
	Upper 95% confidence limit	Mean	Lower 95% confidence limit	
$0 < R \le 0{,}1$	$4{,}75 \times 10^{-15}$	$2{,}31 \times 10^{-15}$	$1{,}12 \times 10^{-15}$	3,66
$R = 0{,}5$	$1{,}60 \times 10^{-14}$	$8{,}57 \times 10^{-15}$	$4{,}53 \times 10^{-15}$	3,60

Notes:
1. R = algebraic stress ratio, f_{min} / f_{max} (tension positive)
2. $da/dN = C(\Delta K)^n$
3. ΔK in N/mm$^{3/2}$, da/dN in mm/cycle
4. Valid for $300 \le K \le 1800$ N/mm$^{3/2}$

Figure C.8.4 shows the crack propagation data obtained for stainless steels in air below 100°C. The scatter band for crack growth data obtained from carbon steel[142] is also shown for comparison. Fatigue crack growth behaviour of type 1.4301 and comparison of type 1.4301 with 1.4401[143] are illustrated in Figure C.8.5 and Figure C.8.6 respectively. Propagation data relating specifically to duplex 1.4462 [144] are shown in Figure C.8.7.

The review of data on crack growth behaviour in air below 100°C indicates that the growth rates in stainless and carbon steel are similar (as shown in Figure C.8.4). This suggests that the well established Paris Law coefficients n and C for carbon steels[142] may be used for the fracture mechanics analysis of stainless steels (Table C.8.1).

A review of threshold stress intensity factors ΔK_{th} for the stainless steel types was also carried out[145] and the results are tabulated in Table C.8.2 and illustrated in Figure C.8.8. These values are similar to those for carbon steels. The recommended value of ΔK_{th} = 2MN/m$^{3/2}$ for use with welded structures is a lower bound to the values in Table C.8.2 and Figure C.8.8 (and in particular to higher values of *R*) and is the same as that used for the assessment of crack behaviour in carbon steels.

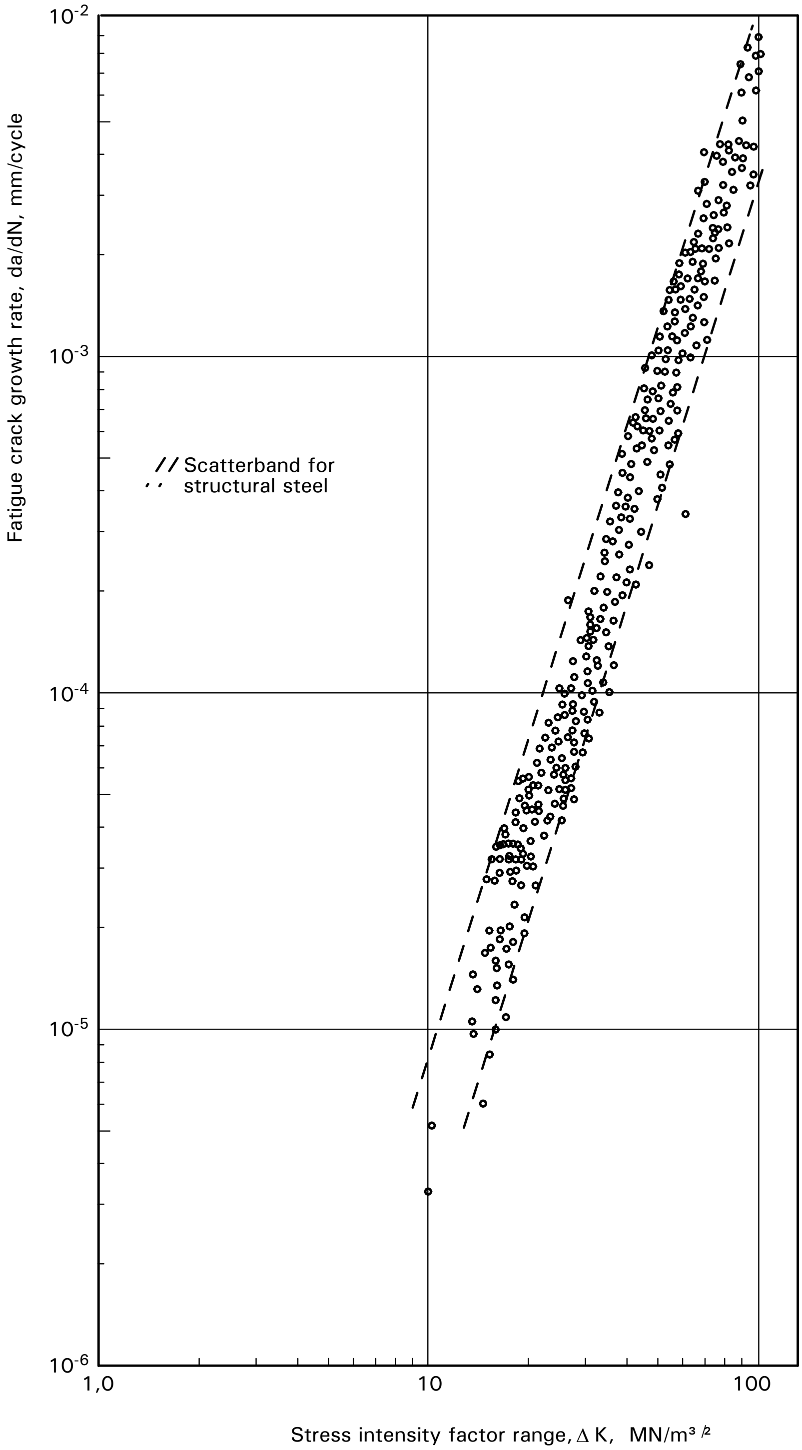


Figure C.8.4 *Crack growth rate data for stainless steels in air at temperatures less than 100°C*

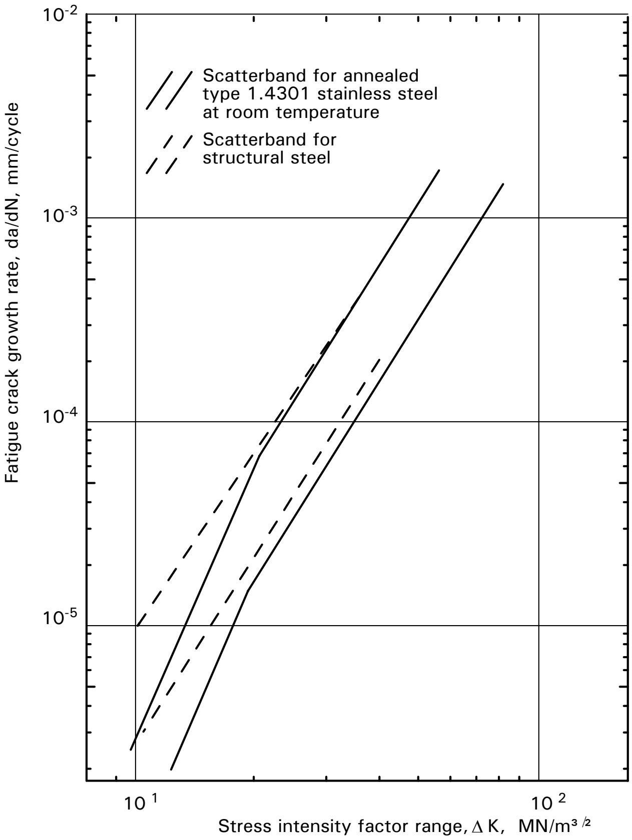


Figure C.8.5 *Fatigue crack growth behaviour of stainless steel grade 1.4301 at room temperature*

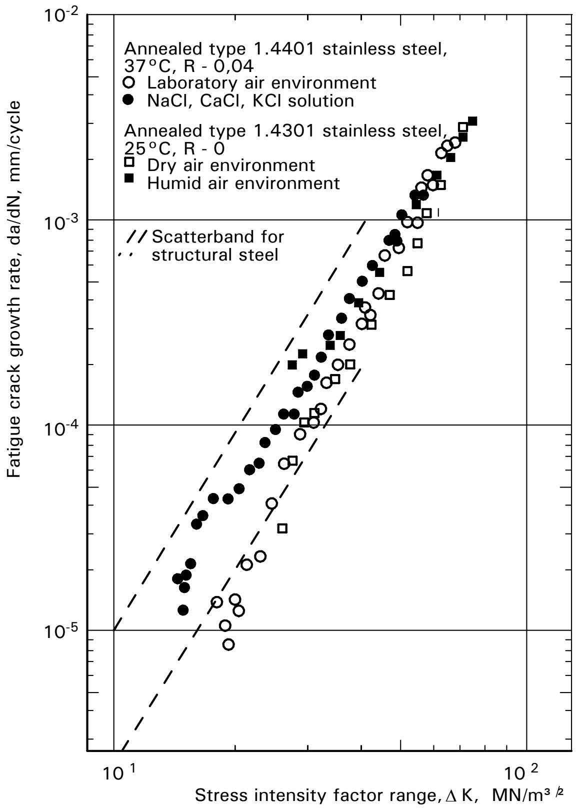


Figure C.8.6 *Comparison of fatigue growth behaviour of austenitic stainless steels*

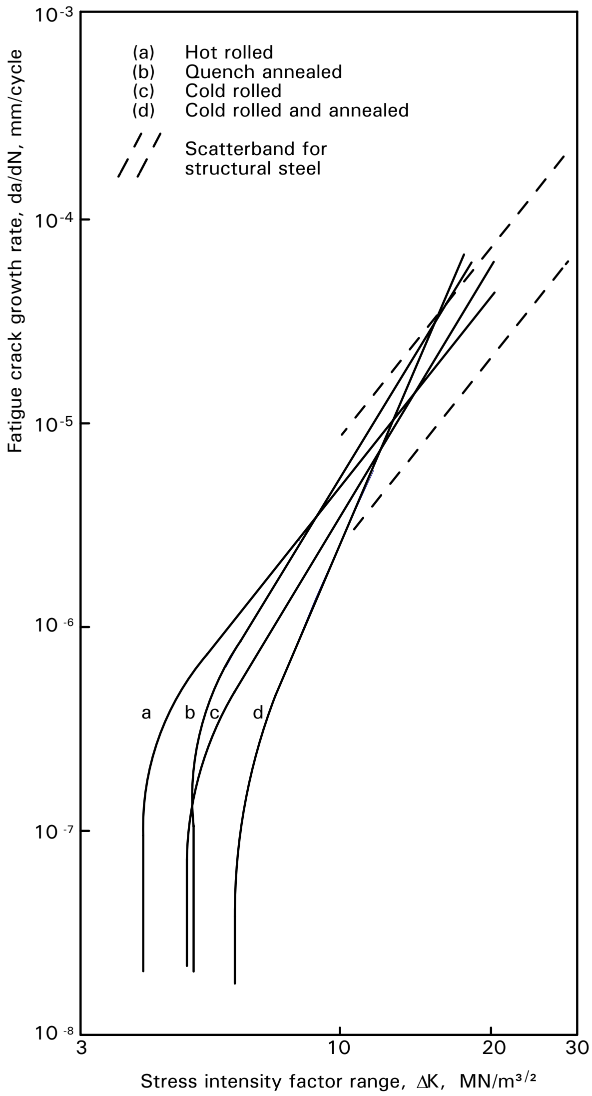


Figure C.8.7 *Fatigue crack growth behaviour of duplex grade 1.4462*

Table C.8.2 *Fatigue threshold values for stainless steel in air at room temperature*

Material	Yield Strength (N/mm²)	Stress Ratio *R*	ΔK_{th} (MN/m$^{3/2}$)
1.4401 (18Cr, 12Ni)	268	0,08	5,2
		0,1	5,0
		0,2	4,3
		0,38	3,2
		0,5	3,3
1.4401, as previous; aged	292	0,12	3,7
		0,33	3,4
		0,55	2,7
		0,68	2,7
1.4401 (18Cr, 12Ni)	255	0,05	6,8
		0,05	6,1
		0,2	5,3
		0,35	4,5
		0,6	3,0
1.4401 (18Cr, 12Ni)	198	0,02	8,1
		0,2	6,9
		0,33	6,2
		0,35	5,9
		0,61	3,8
1.4301 (18,5Cr, 8,8Ni)	222	0,0	5,5
		0,5	3,1
		0,8	2,9
		0,9	2,3
1.4301 (20,2Cr, 8,5Ni)	265	0,0	3,5
		0,4	3,5
		0,8	4,0
1.4301 (19,2Cr, 10,3Ni)	221	0,0	5,6
		0,17	4,5
		0,37	4,2
		0,80	2,8

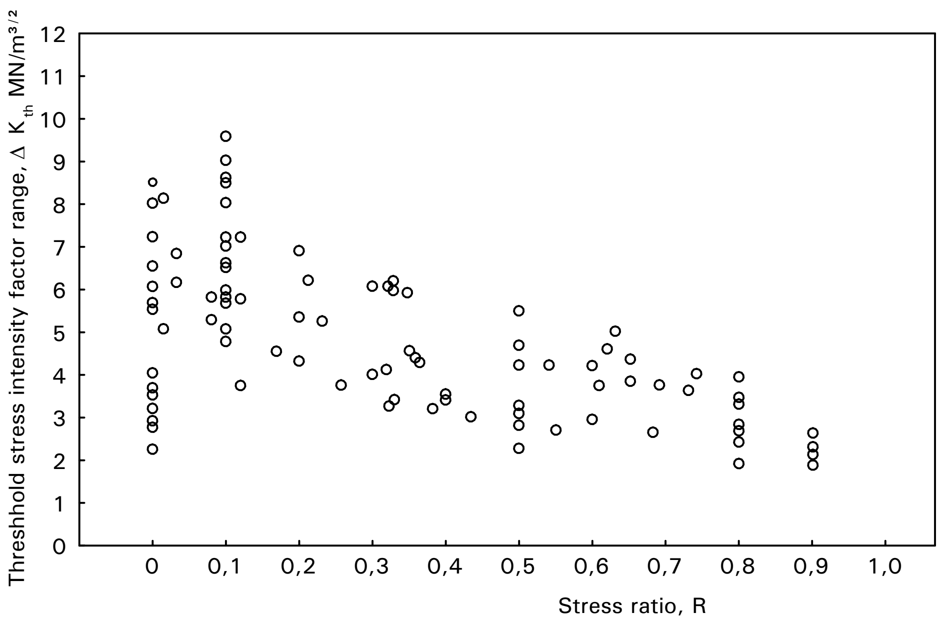


Figure C.8.8 *Variation of threshold stress intensity factor range with stress ratio for stainless steels in air*

C.9 TESTING

The guidance given in the Recommendations has been formulated with the benefit of experience gained in various test programmes that supplied background data for the First, Second and Third Editions of the Design Manual.

It should be appreciated that it is difficult to obtain an accurate stress-strain curve. In particular, it is difficult to obtain a reliable measurement of modulus using extensometry. Other techniques, such as strain gauging or acoustic resonance methods, may prove more satisfactory.

C.10 FABRICATION ASPECTS

C.10.1 Introduction

A broad overview of the precautions to be observed during fabrication is given in the Recommendations and not much further is added in this commentary.

It is emphasised that fabrication should be considered early in the design process as it may affect choice of material grade and structural form (cold formed or welded).

Advice should always be sought if in doubt. Information and literature is freely available from stainless steel producers, weld consumable manufacturers and fabricators. Indeed, much of the information presented in the Recommendations is gathered from such sources. The European specification for fabrication and erection of structural stainless steel, ENV 1090-6[146], covers materials, storage and handling, forming, cutting, joining methods, tolerances and inspection and testing. All the parts of ENV 1090 are currently being converted into EN 1090 and the requirements for the execution of steel and stainless steel structures will be contained in EN 1090-2[147] with EN 1090-1[148] covering rules for using CE marks on steel structures. Reference 149 is a handbook on the erection and installation of stainless steel components which interprets and amplifies the guidance in EN 1090-2 for stainless steel. Reference 150 gives general information about working with stainless steel.

C.10.2 Storage and handling

The use of appropriate storage and handling procedures will avoid iron contamination and surface damage, both of which may subsequently initiate corrosion. Whereas embedded iron can be relatively easily removed (by pickling), scratches may prove troublesome and costly to rectify on surfaces with fine finishes. Iron contamination is discussed in Reference 151. More information on pickling and passivation is given in Reference 152.

Advice on selecting appropriate protective coatings, and their removal, may be found in Reference 153.

C.10.3 Shaping operations

Stainless steel can be machined by all the usual techniques, though different cutting speeds and feeds to those used for carbon steel are normally required. Note that stainless steel swarf is dangerous by virtue of its length and sharpness.

Commonly, brake presses are of 3 m length capacity. However, more powerful machines which cold form longer lengths are available. Discussions with fabricators are recommended to establish plate width and thickness limits.

C.10.4 Welding

The area of stainless steel fabrication where most care is required is welding. That is not to say it is difficult, but rather that corrosion and metallurgical aspects also have to be considered. In general, fabricators who have had experience of working with stainless steels are well informed of the possible pitfalls and their advice should be heeded.

As noted above, steel suppliers and consumable manufacturers produce informative literature. This ranges from brief non-technical pamphlets, through more detailed guidance on recommended joint types and welding parameters, to very technical papers such as the effect of alloying elements on corrosion resistance in specific environments. There is also a wealth of literature in journals, conference proceedings, etc. However, it is fair to say that most of the literature in journals and conference proceedings has little immediate practical relevance to the structural applications for which this Design Manual has been prepared.

EN 1011-3[154] contains much useful information about arc welding stainless steels. EN ISO 15609-1[155] covers welding procedures and EN 287-1[156] covers approval testing of welders. Reference 157 gives general information about welding stainless steel. A comparison of the performance of common manual welding processes for stainless steel is given in Reference 158. As well as examining the technical performance, the study considered economic aspects of each process. The report also contains numerous practical comments for welders and welding engineers.

Tuthill[151] discusses various post weld treatment techniques (mechanical abrasion methods and pickling) to restore the corrosion resistance of the stainless steel. Examples are given of corrosion attacks where simple cleaning procedures were not followed. Reference 152 is also relevant.

Ultrasonics is not normally used for inspecting welds in stainless steel because the grain size in the welds is comparable to the wavelength of the beam which is thus strongly scattered. However, new techniques have been developed[159] for use in critical applications.

C.10.5 Galling and seizure

Reference 160 contains relevant information.

C.10.6 Finishing

Reference 161 describes more fully the various options for finishing a fabricated component.

APPENDIX A Correlation between stainless steel designations

Table A.1 is taken from ENV 1993-1-4.

APPENDIX B Lateral-torsional buckling slenderness, λ_{LT}

The various formulae presented in the Recommendations are taken from the June 2002 version of prEN 1993-1-1[162], which was approved by CEN's sub-committee SC3. These formulae were subsequently removed from the final version of EN 1993-1-1 to allow greater scope of sources of values.

APPENDIX C Material data for deflection calculations

The formula for estimating the secant modulus (using the constants given in Table C.1) is derived from the Ramberg-Osgood description of non-linear stress-strain curves[15]:

$$\varepsilon = \frac{\sigma}{E} + 0{,}002\left(\frac{\sigma}{f_y}\right)^n \quad \text{or} \quad \varepsilon = \frac{\sigma}{E}\left[1 + 0{,}002\left(\frac{E}{f_y}\right)\left(\frac{\sigma}{f_y}\right)^{n-1}\right]$$

The secant modulus, E_S, is thus

$$E_S = \frac{\sigma}{\varepsilon} = \frac{E}{\left[1 + k(\sigma / f_y)^{n-1}\right]}$$

where $k = 0{,}002\ E/f_y$

The k factors in Table C.1 of the Recommendations are derived from the minimum 0,2% proof strengths given in EN 10088-2 for hot rolled strip/plate and E=200 000 N/mm^2 as given in EN 10088-1 (Sections 3.1.2 and 3.2.4 of the Recommendations). The secant moduli in Table C.2 of the Recommendations were derived from the given formula and the constants in Table C.1.

The n factors are the exponents of fitted Ramberg-Osgood curves to experimental data[14]. It may be noted that a range of n factors can be found depending on how the curve fitting is carried out. For instance, fitting a curve to a number of points on the experimental stress-strain curve up to and beyond the 0,2% proof strength results in relatively high n values. However, forcing the fitted curve to simulate the observed departure from linearity (conventionally taken at the 0,01% proof stress) results in rather lower n values. The latter method was adopted for reasons of conservatism as deflection calculations are carried out at stresses below the 0,2% proof stress (for greater stresses, higher n factors are more conservative).

The constants E, k and n are necessarily derived from short term stress-strain curves and thus do not allow for the effects of room temperature creep. This need only be of concern when there is long term loading at a high level of stress.

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