A COMPARISON OF STRUCTURAL STAINLESS STEEL DESIGN STANDARDS

N R Baddoo
The Steel Construction Institute

Abstract

Design standards for structural stainless steel are compared. The European (Eurocode 3 ENV 1993-1-4), American (SEI/ASCE 8-02) and Australian/New Zealand (AS/NZS 4673) standards are reviewed in some detail. ENV 1993-1-4 can be used for designing hot rolled, fabricated and cold-formed sections, whereas the scope of the SEI/ASCE and AS/NZS Specifications is confined to the design of cold-formed sections. The grades and mechanical properties covered by the standards are compared. The design guidance for cross-sections and members is explained and compared. One important difference is that SEI/ASCE and AS/NZS Specifications adopt the tangent modulus method for calculating the buckling strength of members, which generally requires iteration to find a solution. By comparison, the Eurocode buckling curves are based on the initial modulus of elasticity and avoid the need for iteration; they were derived by calibration against experimental data. The buckling curves in the SEI/ASCE Specification are generally more conservative than the European curves. The SEI/ASCE Specification gives more conservative guidance on the calculation of the moment capacity of restrained beams, particularly for circular hollow sections. In general, the SEI/ASCE Specification requires a significantly greater calculation effort than that required by ENV 1993-1-4 and the AS/NZS Specification.

1   INTRODUCTION

This paper is a comparison of structural stainless steel design standards, with particular emphasis on the design of cross-sections and members. Structural design standards for carbon steel should not be applied to stainless steel because stainless steel has different strength and stiffness properties. The major difference between the mechanical properties of carbon and stainless steel is the stress-strain relationship - stainless steel has a continuous, but non-linear, relationship between stress and strain, whereas carbon steel has a clearly defined yield point.

The following sections 1.1 to 1.5 introduce five design standards. However, subsequent sections of the paper compare the design provisions contained only in the first three of these standards.

1.1   Europe
In 1988, a joint industry project was undertaken by The Steel Construction Institute to develop design guidance for European stainless steel structures. Euro Inox published the design recommendations arising out of this project in 1994 as the *Design Manual for Structural Stainless Steel*[1]. Some European countries also published national design guidance documents, for example those issued by the Finnish Constructional Steelwork Association[2]. In 1996, the European Standards organisation CEN issued the 'pre-standard' Eurocode ENV 1993-1-4 *Design of steel structures, Supplementary rules for stainless steels*[3]. This standard is closely aligned with the guidance in the First Edition of the Euro Inox Design Manual, with some changes arising from the need to align to the provisions for carbon steel in ENV 1993-1-1[4] as much as possible. In 2002, Euro Inox published the Second Edition of the *Design Manual for Structural Stainless Steel* which updates and extends the guidance to cover circular hollow sections and fire resistant design. ENV 1993-1-4 is currently being converted to a full EN European Standard. It is anticipated that the contents of the EN will be very closely aligned to the Second Edition of the Design Manual.

1.2   US
The first American specification dealing with the design of structural stainless steel members was published in 1968 by the AISI[5]. Following an extensive research project at Cornell University, in 1974 the specification was revised and published as the *Specification for the Design of Stainless Steel Cold-Formed Structural Members*[6]. The range of grades covered was extended in the 1991 Edition of this design specification, which included both the load and resistance factor design (LRFD) method and the allowable

stress design (ASD) method. The current edition of the design specification, SEI/ASCE 8-02, was published in 2002, with a wider range of grades covered and some minor modifications to a few of the expressions in the 1991 Edition.

1.3   Australia and New Zealand

In 2001, Standards Australia and Standards New Zealand jointly published AS/NZS 4673:2001 *Cold-formed Stainless Steel Structures*[7]. It is based on the SEI/ASCE Specification, partly because most structural applications of stainless steel in Australia use cold-formed members. In addition, the recent Australia/New Zealand Standard for cold-formed steel structures was based on the AISI *Specification for the Design of Cold-formed Steel Structural Members*[8]. As the layout and notation of ASCE and AISI Specifications are similar, using the SEI/ASCE Specification as a basis for the stainless code resulted in similar and easily comparable Australian specifications for cold-formed carbon steel and stainless steel. The testing provisions contained in Section 6 of the SEI/ASCE Specification are replaced by the testing provisions for the Australian/New Zealand Standard for cold-formed steel structures with minor modifications.

1.4   South Africa

In 1997 the South African Board of Standards published SANS 10162-4 / SABS 0162-4:1997 - *Structural use of steel* Part 4: *The design of cold-formed stainless steel structural members*[9]. It is based on the South African and Canadian design specifications for carbon steel cold-formed structural members.

1.5   Japan

In 1995, the *Design and Construction Specifications for Stainless Steel Structures* were published by the Stainless Steel Building Association of Japan[10]. These specifications are only available in Japanese and cover the design of welded, fabricated sections from relatively thick plate.

2   SCOPE OF THE STANDARDS

2.1   Europe

ENV 1993-1-4 gives supplementary provisions for the design of buildings and civil engineering works. It extends the application of ENV 1993-1-1[4] (which covers general rules for the structural design of hot rolled and welded carbon steel sections) and ENV 1993-1-3[11] (which covers the structural design of cold-formed light gauge carbon steel sections) to austenitic and duplex stainless steels. Unlike the US, Australian/New Zealand and South African standards, its scope is not confined to cold-formed cross-sections, and it includes specific guidance for sections made up of welded plate.

As well as the standard austenitic grades 304 (1.4301) and 316 (1.4401), it covers the low carbon grades and stabilised grades, e.g. 321 (1.4541) and 320 (1.4571) that are popular in some European countries. The duplex grade 1.4462 is also included. An annex gives a conservative approach for designing ferritic grades of stainless steel.

ENV 1993-1-4 includes guidance on fatigue design, making reference to the guidance for carbon steels, and also gives guidance on assessing the resistance of structures in fire.

2.2   US

This standard provides design criteria for the determination of the strength of stainless steel structural members and connections for use in buildings and other statically loaded structures. The members may be cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steel material. A Commentary describes the reasoning behind, and the justification for, various provisions of the specification.

Four grades of austenitic stainless steels (201, 301, 304 and 316) are covered, in the annealed condition and also cold rolled to 1/16, 1/4 and 1/2 Hard temper levels. Three grades of ferritic stainless steels are included (409 ($\leq$ 3.8 mm), 430 ($\leq$ 3.2 mm) and 439 ($\leq$ 3.2 mm), only in the annealed condition. It is assumed that the forming process is carried out at or near room temperature by the use of bending brakes, press brakes or roll-forming machines.

The scope of the 2002 Edition of the SEI/ASCE Specification was extended to cover grade UNS S20400, also known as Nitronic 30, which is an austenitic nitrogen strengthened grade with a 0.2% proof strength between 50 and 100% higher than grades 304 and 316. It has similar corrosion resistance to grade 304,

with suitable applications including sewage treatment plant structures and bulk solids handling equipment. It is not thought to be widely available in Europe.

No rules are provided for fabricated members, which may contain significant levels of residual stress, nor are there any rules for fatigue design or assessing fire resistance. Currently there is no US design standard for hot rolled or fabricated stainless steel members, and neither is one planned. It is presumed that designers seek guidance from stainless steel producers for the design of these stainless steel sections.

2.3   Australia and New Zealand
The AS/NZS Specification sets out minimum requirements for the design of stainless steel structural members cold formed to shape from annealed or temper-rolled sheet, strip, plate or flat bar stainless steels used for load-carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for dynamic effects.

Mechanical properties are given for the austenitic grades 304, 316, 304L, 316L and the ferritic grades 409 and 430. The specification also includes the duplex alloy S31803 (1.4462) and the 12% chromium weldable structural steel 1.4003, often referred to as 3CR12. This alloy is becoming increasingly popular in Australia, as well as other countries. The mechanical properties for these grades are given in the annealed condition only.

On the basis of tests on welded connections in circular hollow sections (CHS) and square hollow sections (SHS), an appendix is included which allows the nominal strength of tubular joints in CHS, SHS and rectangular hollow sections (RHS) to be calculated using the provisions of Annex K of ENV 1993-1-1. Further appendices provide non-normative information about fatigue and fire.

3   BASIS OF DESIGN

The European, US and Australian/New Zealand Specifications follow the limit state design concept (also called load and resistance factor design, LRFD). This is a method of proportioning structural components such that any applicable limit state is not exceeded when the structure is subjected to any appropriate factored load combination. Separate load and resistance factors (partial safety factors) are applied to specified loads and nominal resistances to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. Two types of limit state are considered in these specifications:

(1)   The limit state of strength required to resist the extreme loads during the intended life of the structure (Ultimate Limit State),

(2)   The limit state of the ability of the structure to perform its intended function under normal service conditions during its life (Serviceability Limit State).

The SEI/ASCE Specification also permits the use of the allowable stress design method (ASD), giving the relevant safety factors in an appendix.

A quantitative comparison of values of the safety index β used in the specifications is outside the scope of this paper.

4   MATERIAL PROPERTIES

4.1   General
Figure 1 shows the stress-strain characteristics for the main grades covered in the standards under consideration. Figure 2 defines the key parameters used to describe the stress-strain curve of stainless steel. Figure 3 shows the variation of tangent and secant modulus with stress, as given in the SEI/ASCE Specification.
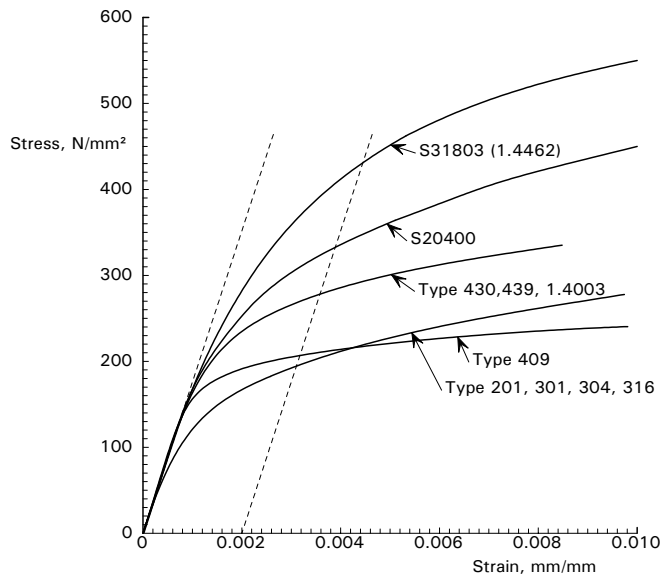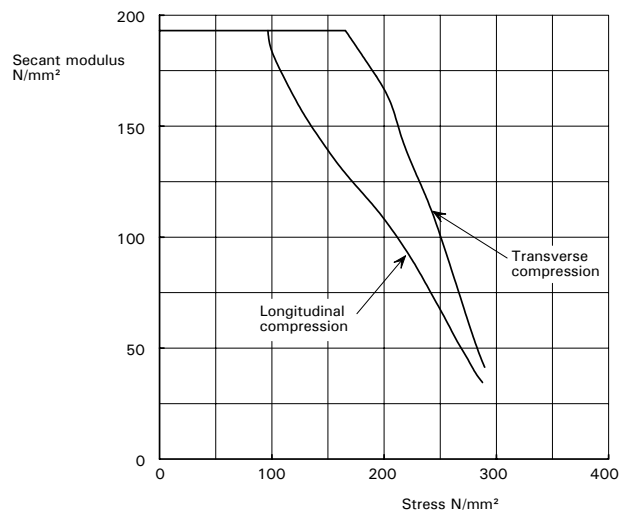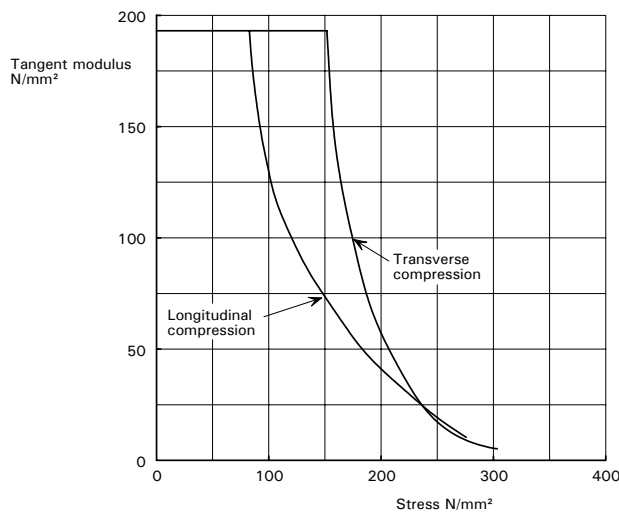
Figure 1 Stress-strain curves for the grades of stainless steel covered in the standards
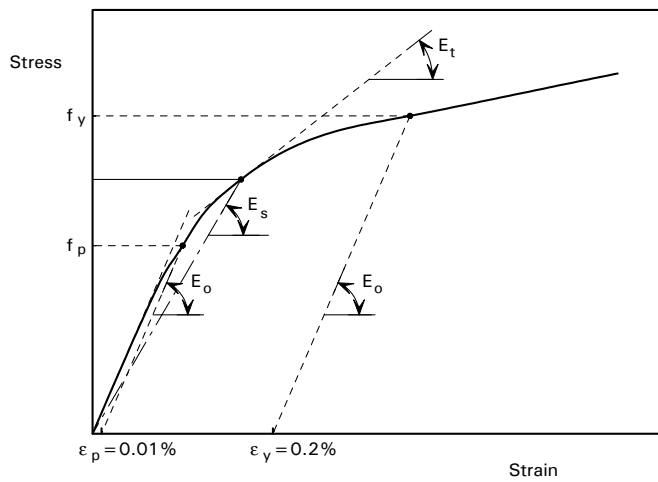
E_0 = initial modulus of elasticity

E_t = tangent modulus

E_s = secant modulus

f_y = yield strength=0.2% proof strength

f_p = proportional limit

Figure 2 Definition of parameters describing the stress-strain curve of stainless steel

Figure 3 Variation of tangent and secant modulus with stress

4.2 Europe

The mechanical and physical properties for use in designing stainless steel structural members are given in EN 10088[12]. The properties are generally only given in the annealed condition. The 0.2% proof strength values are given for the transverse direction only and hence no account is taken of the anisotropy of the material. The properties vary with the product form (cold rolled strip, hot rolled strip, hot rolled plate). However, at the design stage the product form is not always known, so it is not often possible to take advantage of the higher strength properties of cold rolled strip. Table 1 gives the mechanical properties for grade 304 and 316 in the annealed condition in EN 10088.

EN 10088 also gives five tensile strength levels for material in the cold worked condition (Table 2). The design provisions in ENV 1993-1-4 are only applicable to material with yield strength of up to 480 N/mm^2.

ENV 1993-1-4 gives values for the Ramberg Osgood parameter, n which is required for calculating the secant modulus of elasticity for use in deflection calculations. These are compared with the values in the other standards in Table 5.

EN 10088 gives a value of 200,000 N/mm^2 for the modulus of elasticity in the annealed condition.

Table 1   Minimum specified mechanical properties for grades 304 and 316 in the annealed condition

Grade	Product form[1]	Maximum thickness (mm)	Minimum 0.2% proof strength[2] (N/mm^2)	Minimum ultimate tensile strength (N/mm^2)
304 (1.4301)	C	6	230	540
	H	12	210	520
	P	75	210	520
316 (1.4401)	C	6	240	530
	H	12	220	530
	P	75	220	520
Notes:				
(1) C=cold rolled strip, H=hot rolled strip, P=hot rolled plate				
(2) Transverse properties				

Table 2   Mechanical properties for grades 304 and 316 in the cold worked condition

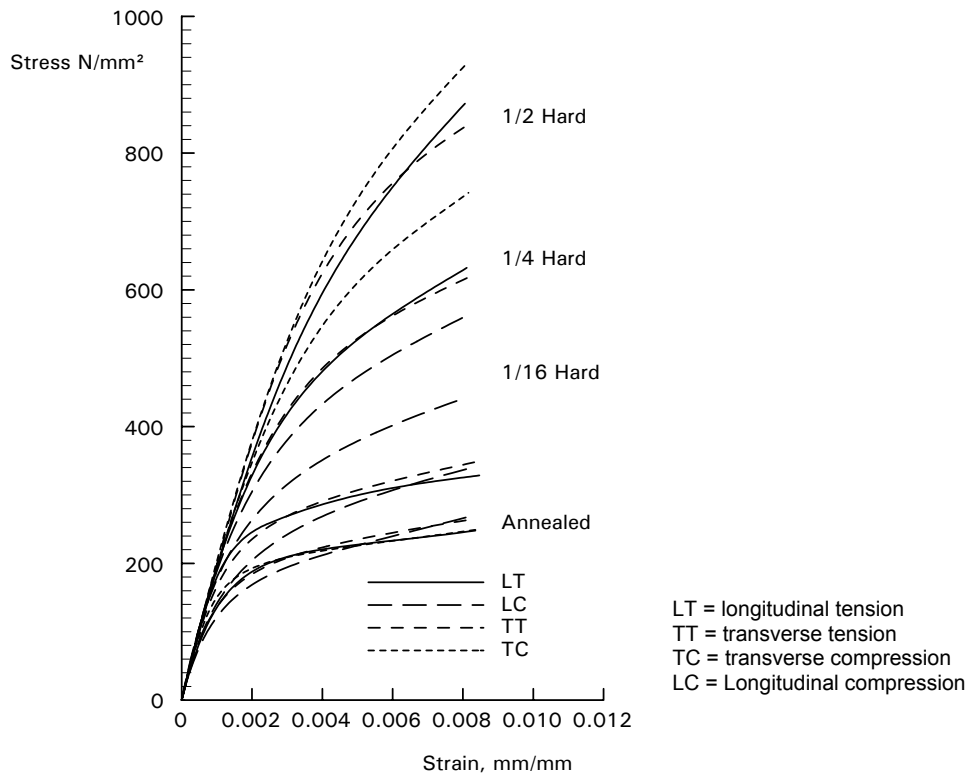| Designation	Minimum yield strength (N/mm^2)	Minimum tensile strength (N/mm^2)
C700	350	700
C850	530	850
C1000	750	1000
C1150	900	1150
C1300	-	1300
Note:		
Tensile strengths are given in EN 10088-2 and yield strengths in ENV 1993-1-4		

4.3 US

The mechanical properties given in the SEI/ASCE Specification take account of the anisotropy and asymmetry (i.e. different behaviour in compression and tension) of stainless steel, which becomes increasingly important as the level of cold working increases (Figure 4).

The SEI/ASCE Specification gives values for the Ramberg Osgood parameter, n, which is required for calculating the secant and tangent modulus of elasticity for use in buckling curves, deflection calculations etc.

A value of 193,100 N/mm^2 is given for the modulus of elasticity for all directions in the annealed condition. For material in the 1/4 and 1/2 tempered condition, a slightly lower value of 186,200 N/mm^2 applies for the longitudinal direction only.

Figure 4   Stress-strain curves for grade 304 in the annealed and cold worked conditions

Table 3   Mechanical properties for grades 304 and 316 in the SEI/ASCE Specification

Temper	Minimum yield strength (N/mm^2)			Minimum tensile strength (N/mm^2)
	Longitudinal tension and Transverse tension	Transverse compression	Longitudinal compression	
Annealed	206.9	206.9	193.1	517
1/16 hard	310.3	310.3	282.7	551.6 (grade 304) 586.1 (grade 316)
1/4 hard	517.1	621	344.8	862
1/2 hard	758.5	827.4	448.2	1034

4.4   Australia and New Zealand

The AS/NZS Specification only gives properties in the annealed condition. The mechanical properties for grades 304, 316, 409 and 430 are the same as those given in the SEI/ASCE Specification, and thus take anisotropy and asymmetry effects into account. The highest strength values are in the transverse tension direction. The mechanical properties for grades 1.4003 and S31803 (1.4462) also take account of the anisotropy of stainless steel. Grade 1.4003 shows particularly strong anisotropy and asymmetry with a maximum value of 0.2% proof strength of 320 N/mm^2 (transverse tension) and minimum value of 260 N/mm^2 (longitudinal compression).

4.5   Comparison of standards

Table 4 compares the mechanical properties for grades 304 and 316 in the annealed condition given in the standards. The values are very similar.

Table 5 compares the values for the Ramberg Osgood parameter, *n* given in the standards; the variation is surprisingly large, particularly in the transverse tension direction. This may be partially due to the fact that

n is derived from the stress at the limit of proportionality and there is no accepted standard on how this limit should be determined. It is becoming more common to assume that the limit of proportionality is equal to the 0.01% proof strength, but more subjective methods have been used in the past. The limit of proportionality is also notoriously difficult to measure accurately, unless strain gauges are used. In practical design situations, if the direction of rolling is not known, it is conservative to take the lowest value of *n* (i.e. the most non-linear stress-strain curve), which corresponds to the longitudinal compression direction.

Table 4 Comparison of mechanical properties for grades 304 and 316 in the annealed condition

Country	Grade	Minimum yield strength (N/mm^2)	Minimum ultimate tensile strength (N/mm^2)
Europe	304	210[1]	520
	316	220[1]	520
US	304 and 316	206.9 (LT, TT, TC)[2] 193.1 (LC)	517
Australia	304 and 316	205 (LT, TT, TC)[2] 195 (LC)	520

Notes:
(1)   For hot rolled plate, transverse properties
(2)   LT=longitudinal tension, TT=transverse tension, TC=transverse compression,   LC=Longitudinal compression

Table 5 Comparison of values for the Ramberg Osgood parameter *n* in the annealed condition for grades 304 and 316

Country		Longitudinal tension	Transverse tension	Transverse compression	Longitudinal compression
Europe	Grade 304	6.5	8.5	8.5	6.5
	Grade 316	7.0	9.0	9.0	7.0
US		8.31	7.78	8.63	4.10
Australia		7.5	5.5	7.0	4.0

5   EFFECTIVE WIDTHS OF SLENDER ELEMENTS

All three standards adopt the effective width concept for taking the post-buckling strength of stainless steel compression elements into account. They use the initial elastic modulus, E_0 and so do not account for the effect of gradual yielding.

5.1   Europe

ENV 1993-1-4

ENV 1993-1-4 follows the Eurocode guidelines for carbon steel in ENV 1993-1-1[4] and ENV 1993-1-3[11] with respect to calculating the effective width of a compression element. The slenderness of an element is defined in terms of the non-dimensional plate slenderness $\bar{\lambda}_p$. The reduction factor for calculating the effective width is ρ, where:

$$\bar{\lambda}_p = 1.052 \frac{b_p}{t} \sqrt{\frac{\gamma_{M1} \, \sigma_{com,Ed}}{E_0 \, k_\sigma}}$$

if $\bar{\lambda}_p \leq 0.673$, $\rho = 1.0$

if $\bar{\lambda}_p > 0.673$, $\rho = \dfrac{1.0 - \dfrac{0.22}{\bar{\lambda}_p}}{\bar{\lambda}_p}$

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in which:

b_p is the width of the element – the precise definition depending on whether the member is cold formed (in which case it is defined in ENV 1993-1-3), or welded or hot rolled (in which case it is defined in ENV 1993-1-1)

t is the thickness of the element

γ_{M1} is a partial safety factor applied to the cross-section resistance

$\sigma_{com,Ed}$ is the largest compressive stress in the relevant element

k_σ is the relevant buckling factor

The provisions in ENV 1993-1-1 and 1-3 for determining the position and distribution of the effective width and the value for k_σ are adopted in ENV 1993-1-4.

For the determination of deflections, it is permissible to use an effective cross-section based on the effective widths of compression elements determined using the stresses due to serviceability loading.

Euro Inox Design Manual
The Euro Inox *Design Manual for Structural Stainless* Steel recommends a more conservative approach for calculating effective widths which takes into account whether the element is internal or external, and whether the element is welded or cold-formed. It is considered likely that these curves will be adopted in the EN 1993-1-4.

The reduction factor ρ is calculated as follows:

Cold formed or welded internal elements: $\rho = \dfrac{0{,}772}{\overline{\lambda}_p} - \dfrac{0{,}125}{\overline{\lambda}_p^2}$ but ≤ 1

Cold formed outstand elements: $\rho = \dfrac{1}{\overline{\lambda}_p} - \dfrac{0{,}231}{\overline{\lambda}_p^2}$ but ≤ 1

Welded outstand elements: $\rho = \dfrac{1}{\overline{\lambda}_p} - \dfrac{0{,}242}{\overline{\lambda}_p^2}$ but ≤ 1

5.2   US
The SEI/ASCE Specification adopts the approach taken in ENV 1993-1-4 for calculating the effective width for uniformly compressed stiffened (i.e. internal) elements and webs and stiffened elements with stress gradients. The parameter b_p is defined as the flat width, which is slightly smaller than the notional flat width used in ENV 1993-1-3. The same approach is followed in the SEI/ASCE Specification for unstiffened compression (i.e. outstand) elements, although a less conservative value of k_σ =0.5 is taken, as compared to a value of 0.43 used in ENV 1993-1-4. Unlike the Eurocode, there is no guidance on calculating less conservative values for k_σ for unstiffened elements subject to a stress gradient.

The SEI/ASCE Specification adopts the provisions in the AISI Specification for cold-formed carbon steel[8] for determining the position and distribution of the effective width and the value for k_σ for elements with edge stiffeners or one intermediate stiffener.

For determining deflections, the effective widths are calculated using the compression stress arising from the loads for which deflections are determined. In addition, a reduced modulus of elasticity E_r is substituted for E_0 in the expression for the plate slenderness. E_r is taken as the average of the secant moduli corresponding to the stresses in the tension and compression flanges.

5.3   Australia and New Zealand
The approach in the SEI/ASCE Specification is adopted. As with the Eurocodes, there is guidance on calculating more accurate values for k_σ for unstiffened elements subject to a stress gradient.

5.4 Comparison of standards

Figures 5, 6 and 7 show the reduction factor ρ versus $\overline{\lambda}_p$ given in ENV 1993-1-4 and the SEI/ASCE Specification and the more conservative curves given in the Euro Inox Design Manual against test data. The Euro Inox curves for both cold-formed and welded internal elements lie significantly below those in ENV 1993-1-4 and the SEI/ASCE Specification, whereas there is a negligible difference in the case of cold-formed outstands.
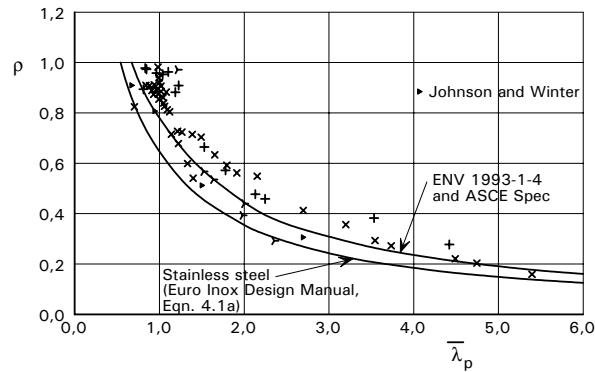
Figure 5   Reduction factor ρ versus plate slenderness $\overline{\lambda}_p$ for cold formed internal elements
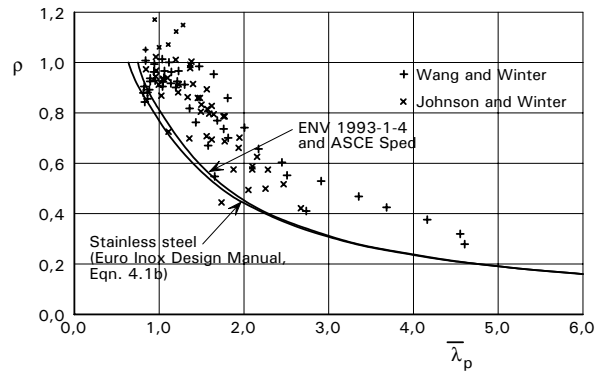
Figure 6   Reduction factor ρ versus plate slenderness $\overline{\lambda}_p$ for cold formed outstand elements
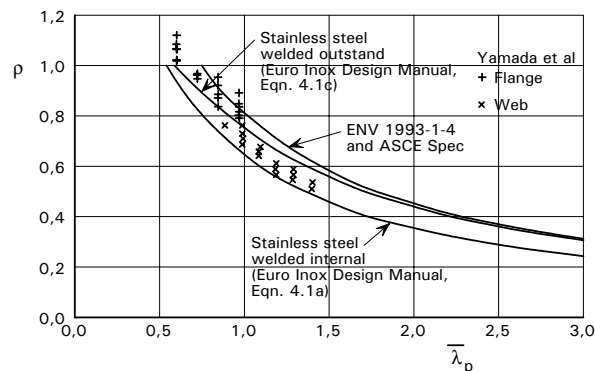
Figure 7   Reduction factor ρ versus plate slenderness $\overline{\lambda}_p$ for welded elements

6   MEMBERS SUBJECT TO AXIAL TENSILE LOAD

6.1   Europe

In ENV 1993-1-4, the resistance of cross-sections subject only to uniform tensile stresses is taken as the smaller of the plastic resistance of the gross cross-section and the ultimate resistance of the net cross-section at holes for fasteners. The net cross-section at fastener holes is calculated by multiplying the net area by a factor k_r given by:

$$k_r = [1 + 3r(\frac{d_0}{u} - 0.3)] \quad \leq \quad 1.0$$

where:

r = [number of bolts at the cross-section]/[total number of bolts in the connection]

d_0 is the nominal bolt hole diameter

u = $2e_2$ but $u \leq p_2$

e_2 is the edge distance

p_2 is the minimum spacing normal to the direction of stress

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6.2 US

The approach taken in the SEI/ASCE Specification is the same as that taken in ENV 1993-1-4, except that the expression for k_r includes the bolt diameter instead of the bolt hole diameter. Note that it is general Eurocode practice to use the bolt hole diameter (as opposed to the bolt diameter) in expressions used for calculating the strength of connections.

6.3 Australia and New Zealand

The approach in the SEI/ASCE Specification is adopted.

7 MEMBERS SUBJECT TO AXIAL COMPRESSIVE LOAD

7.1 Europe

When considering the buckling of stainless steel, it is necessary to take into account the effect of the low proportional limit, residual stresses and the gradual yielding behaviour of stainless steel. The buckling curves in ENV 1993-1-4 were derived by calibration against experimental data as it was considered preferable to have an explicit design solution as opposed to one requiring an iterative solution.

In accordance with ENV 1993-1-1, the design approach for members subject to axial compressive forces is based on the Perry-Robertson buckling curve and uses a linear expression for the imperfection parameter $\alpha(\lambda - \lambda_0)$. The values for α and λ_0 are different from those for carbon steel columns in recognition of the differences in mechanical properties and magnitudes of residual stresses.

The reduction factor to be applied to the squash load to account for flexural buckling, χ is given by:

$$\chi = \frac{1}{\varphi + \left[\varphi^2 - \bar{\lambda}^2\right]^{0,5}} \leq 1$$

in which

$$\varphi = 0,5 \left(1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right) \quad \text{and} \quad \bar{\lambda} = \frac{l}{i} \frac{1}{\pi} \sqrt{\frac{f_y \beta_A}{E_0}}$$

where:
f_y is the yield strength
l is the buckling length
i is the radius of gyration of the gross cross-section
β_A is the ratio of the effective cross-section area to the gross cross-section area
α is the imperfection factor
$\bar{\lambda}_0$ is the limiting slenderness

Figure 8 shows the buckling curves; the values for α and $\bar{\lambda}_0$ depend on the mode of buckling and the type of member. The figure shows different buckling curves for welded open sections subject to major and minor axis bending, which were recommended in the Second Edition of the Euro Inox Design Manual.

For CHS in compression, no design guidance is given for slender cross-sections, i.e. where the ratio of the outside diameter to thickness, $d/t > 0.101 E_0 / f_y$.
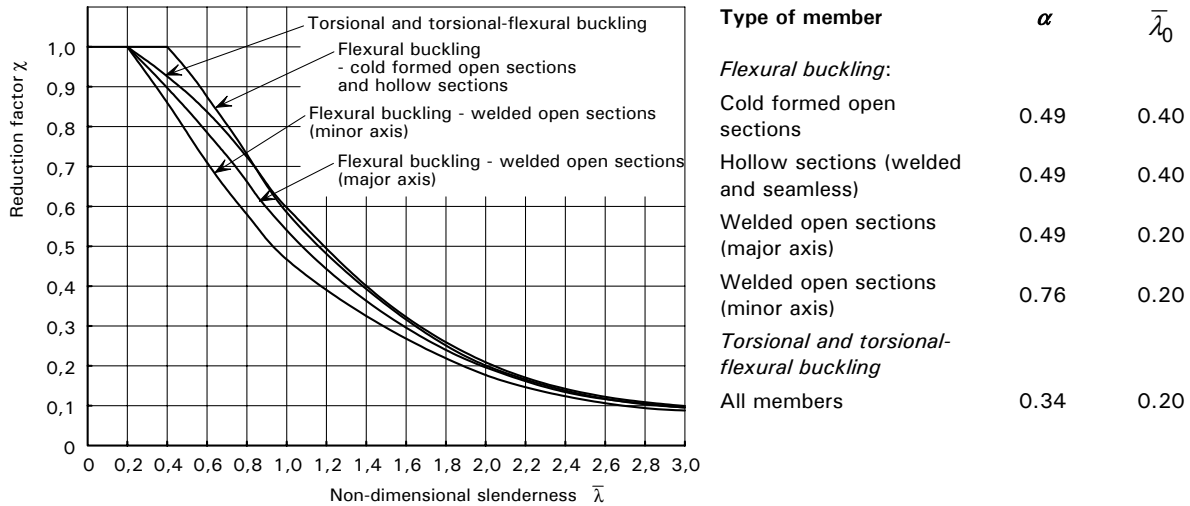
Figure 8   Buckling curves for flexural, torsional and torsional-flexural buckling

7.2   US

In order to take account of the non-linear stress-strain curve of stainless steel, the SEI/ASCE Specification replaces the initial elastic modulus with the tangent modulus, E_t corresponding to the buckling stress, which involves an iterative design procedure.

For doubly symmetric sections, closed cross-sections and any other sections which can be shown not to be subject to torsional or torsional-flexural buckling, the flexural buckling stress F_n is determined from:

$$F_n = \frac{\pi^2 E_t}{(kL/r)^2} \leq F_y$$

E_t is the tangent modulus in compression corresponding to the buckling stress
k is the effective length factor
L is the unbraced length of the member
r is the radius of gyration of the full, unreduced cross-section
F_y is the yield strength

E_t is obtained from:

$$E_t = \frac{F_y E_0}{F_y + 0.002 n E_0 \left(\dfrac{F}{F_y}\right)^{n-1}}$$

where:
n is the Ramberg Osgood parameter
F is the stress (F_n) in the member

Similarly, the tangent modulus of elasticity and tangent shear modulus are also used in the calculation of the torsional buckling stress σ_t:

$$\sigma_t = \left(\frac{1}{Ar_0^2}\right)\left(G_0 J + \frac{\pi^2 E_0 C_w}{(K_t L_t)^2}\right)\left(\frac{E_t}{E_0}\right)$$

For doubly or point symmetric sections which may be subject to torsional buckling, F_n is taken as the smaller of F_n calculated above or σ_t.

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Figure 9 compares flexural buckling curves for two materials with identical values of yield stress and modulus of elasticity, but with different n values, a low value ($n=4$), describing a material with a highly non-linear stress-strain curve, and a high value ($n=10$) describing a material with a more linear elastic-plastic stress-strain curve. At high slendernesses, the curves approximate to the Euler buckling curve. Generally, the buckling strength of the material with the low n value is smaller than that for the material with the high n value, and the difference is very significant at intermediate values of slenderness. The SEI/ASCE Specification gives an n value of 4.1 in the longitudinal compression direction for grades 304 and 316.
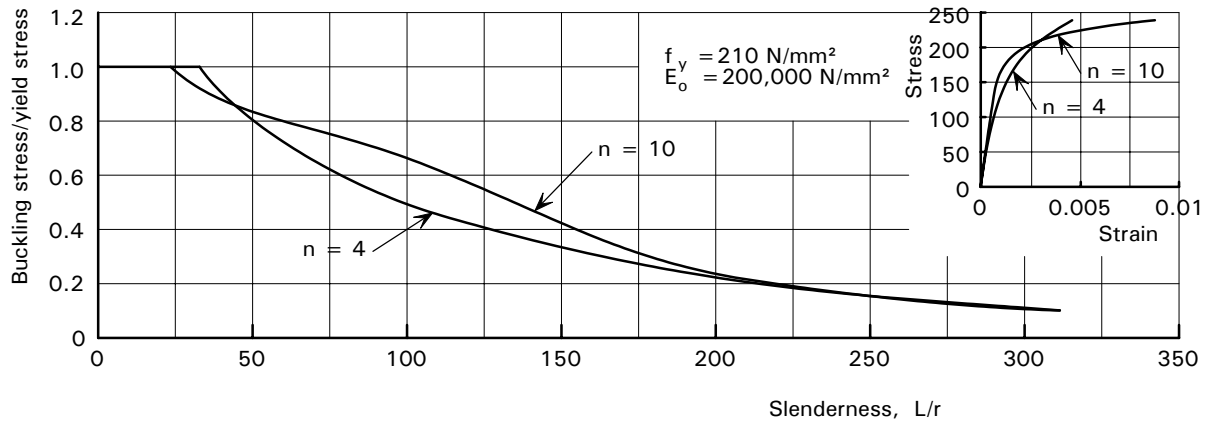
Figure 9   Comparison of buckling curve with a low and high Ramberg Osgood parameter, n

For CHS, guidance is given for calculating the buckling strength of slender sections where $0.112E_0/F_y \leq d/t \leq 0.881E_0/F_y$. The buckling strength is multiplied by a non-linear reduction factor K_c which reduces with increasing d/t ratio and also depends on the ratio of the effective proportional limit to yield strength.

7.3   Australia and New Zealand

In addition to the iterative method from the SEI/ASCE Specification, an explicit design procedure is given in the AS/NZS Specification. The explicit design procedure gives the following expression for the flexural buckling stress f_n:

$$f_n = \frac{f_y}{\varphi + \left[\varphi^2 - \lambda^2\right]^{0,5}} \quad \leq \quad f_y$$

where $\varphi = 0,5\left(1 + \eta + \lambda^2\right)$, $\quad \eta = \alpha\left((\lambda - \lambda_1)^\beta - \lambda_0\right)$ and $\lambda = \left(\frac{kl}{r}\right)\sqrt{\frac{f_y}{\pi^2 E_0}}$

in which:
k is the effective length factor
l is the unbraced length of the member
r is the radius of gyration of the full, unreduced cross-section
f_y is the yield strength

The above expressions were derived by fitting Perry curves to column strength curves which were generated by finite element analyses of geometrically imperfect columns, thus explaining why the formulation generally leads to lower strength curves than the tangent modulus approach. However, it should be noticed that the column strength obtained using the generalised Perry curve is associated with a higher resistance factor than the strength obtained using the tangent modulus approach. The parameters α, β, λ_0, λ_1 are obtained from stub column or compression coupon tests. For grades 304 and 316, the Specification gives $\alpha = 1.59$, $\beta = 0.28$, $\lambda_0 = 0.55$ and $\lambda_1 = 0.20$, and these correspond to an n value of approximately 4.

For CHS, guidance is given for calculating the capacity of slender sections where $0.313E_0/f_y \leq d/t \leq 0.881E_0/f_y$. The buckling strength is multiplied by a non-linear reduction factor K_c which reduces with increasing d/t ratio and also depends on the ratio of the effective proportional limit to yield strength.

7.4 Comparison of standards

Figure 10 compares the flexural buckling curves in ENV 1993-1-4, the SEI/ASCE Specification (using $n = 4$ and $n = 10$) and the explicit method in the AS/NZS Specification. The range of n from 4 to 10 very approximately reflects the range of n values for grade 304 given in the Specifications when the different directions are considered (see Table 5). The most conservative buckling curve is the ENV 1993-1-4 curve for welded sections.
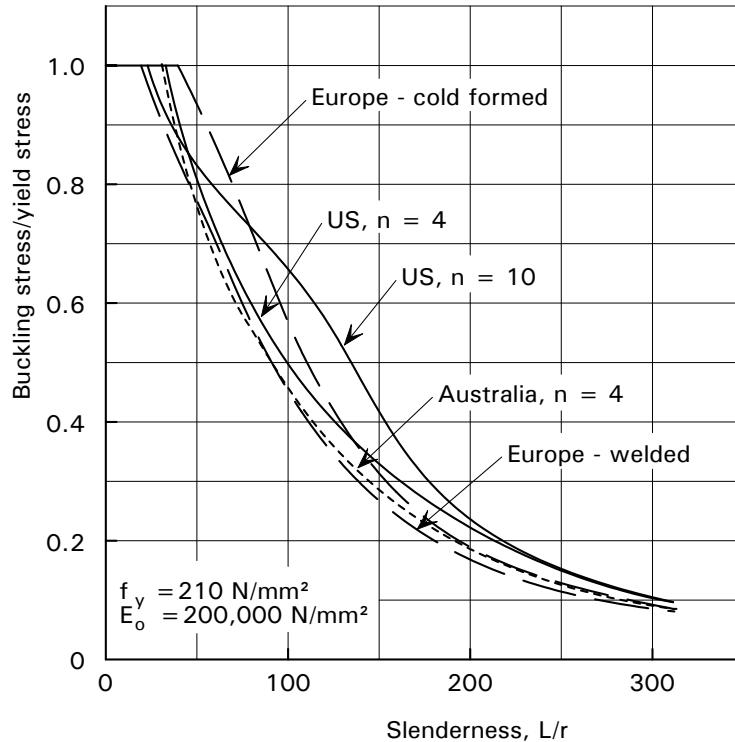
Figure 10 Comparison of flexural buckling curves given in ENV 1993-1-4, SEI/ASCE and AS/NZS Specifications

8   RESTRAINED MEMBERS SUBJECT TO BENDING

8.1   Europe

Elements and cross-sections are classified as Class 1, 2, 3 or 4 depending on the susceptibility to local buckling and their rotation capacity (Class 1 and 2). These Classes determine the maximum allowable moment that can be calculated for a section without the occurrence of local buckling. The Classes are dependent on the width-to-thickness ratios of the elements of the cross-section that are partly or wholly in compression.

The moment capacity of a cross-section subject to a uniaxial moment, $M_{c,Rd}$, is given by:

$M_{c,Rd}$ = $W_{pl}\, f_y/\gamma_{M0}$ for Class 1 or 2 cross-sections
$M_{c,Rd}$ = $W_{el}\, f_y/\gamma_{M0}$ for Class 3 cross-sections
$M_{c,Rd}$ = $W_{eff}\, f_y/\gamma_{M1}$ for Class 4 cross-sections

where:
W_{pl} is the plastic section modulus
W_{el} is the elastic section modulus
W_{eff} is the elastic modulus of the effective section.
γ_{M1}, γ_{M0} are partial safety factors applied to the cross-section resistance

In ENV 1993-1-4, a CHS subject to bending is classified as slender when $d/t > 0.101E_0/f_y$. Following subsequent research[13], this limit was increased to $d/t > 0.313E_0/f_y$ in the Second Edition of the Euro Inox Design Manual. For CHS, no design guidance is given for slender cross-sections, i.e. where $d/t > 0.313E_0/f_y$.

8.2   US
The SEI/ASCE Specification calculates the moment capacity either on the basis of initiation of yielding (Procedure 1) or on the basis of the inelastic reserve capacity (Procedure 2).
Procedure 1
The nominal flexural strength M_n is given by $M_n = S_e F_y$ where S_e is the elastic section modulus of the effective section.

Procedure 2
The nominal flexural strength M_n shall not exceed $1.25 S_e F_y$ or that causing a maximum compressive strain of $C_y e_y$ where:

e_y is the yield strain $= F_y/E_0$
C_y is the compression strain factor

For stiffened (i.e. internal) compression elements without intermediate stiffeners:

$$C_y = 3 \qquad \text{for } \frac{w}{t} \le \lambda_1$$

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right) \quad \text{for } \lambda_1 < \frac{w}{t} < \lambda_2$$

$$C_y = 1 \qquad \text{for } \frac{w}{t} \ge \lambda_2$$

where $\lambda_1 = \dfrac{1.1}{\sqrt{\dfrac{F_{yc}}{E_0}}}$ and $\lambda_2 = \dfrac{1.28}{\sqrt{\dfrac{F_{yc}}{E_0}}}$

and
w is the flat width of the compression element
F_{yc} is the yield strength in compression

For unstiffened (i.e. outstand) compression elements, multiple stiffened compression elements and compression elements with edge stiffeners, C_y=1.

In addition, the ratio of the depth of the compressed portion of the web to its thickness must not exceed λ_1.

Note that it is only cross-sections such as RHS and top hat sections where the compression element is stiffened that can take advantage of the inelastic reserve capacity. In order to calculate the moment causing a strain of $C_y e_y$, it is necessary to calculate the stress distribution in the cross-section arising from a maximum strain in the compression flange of $C_y e_y$. The moment is a function of the section width and depth, t, F_{yc} and C_y.

For CHS, the SEI/ASCE Specification limits the moment capacity to the elastic moment capacity, providing that $d/t \le 0.112E_0/F_y$. Further guidance is given for calculating the moment capacity of more slender CHS sections where $0.112E_0/F_y \le d/t \le 0.881E_0/F_y$. The moment capacity is multiplied by a non-linear reduction factor K_c that varies with the d/t ratio and also depends on the ratio of the effective proportional limit to yield strength.

8.3   Australia and New Zealand
The approach in the SEI/ASCE Specification is adopted. In addition, this specification permits the plastic section modulus to be used in calculating the moment capacity in the design of RHS provided that the slenderness of the flange, b/t and that of the compressed portion of the web, d_c/t satisfy:

$$\frac{b}{t} \le \frac{1.11}{\sqrt{f_y / E_0}}, \qquad \frac{d_c}{t} \le \frac{1.11}{\sqrt{f_y / E_0}}$$

where b is the flat width of the flange and d_c is the compressed depth of the web.

For CHS the specification permits the plastic section modulus to be used in calculating the moment capacity for cross-section sections satisfying $d/t \le 0.078E_0/f_y$, and the elastic modulus for sections satisfying $0.078E_0/f_y \le d/t \le 0.31E_0/f_y$. Further guidance is given for calculating the moment capacity of more slender sections where $0.31E_0/f_y \le d/t \le 0.881E_0/f_y$. The moment capacity is multiplied by a non-linear reduction factor K_c that varies with the d/t ratio and also depends on the ratio of the effective proportional limit to yield strength.

8.4 Comparison of standards

Table 6 compares the limiting width-to-thickness ratios in the standards for RHS subject to bending in order for the moment capacity to be based on the plastic section modulus (i.e. to be a Class 1 or 2 cross-section). Using the plastic section modulus leads to higher moment capacities than allowed in the SEI/ASCE Specification, which only incorporates some effect of yielding in the design of RHS (the upper limit on the moment capacity is $1.25S_eF_y$, whereas typical ratios of plastic to elastic section moduli for RHS vary from 1.24 to 1.33). The SEI/ASCE Specification method for calculating the inelastic reserve capacity requires more calculation effort than the other standards. The limiting width-to-thickness ratios in ENV 1993-1-4 are more conservative than those in the AS/NZS specification.

Furthermore, ENV 1993-1-4 permits the moment capacity of open sections such as channels with stocky flange and web elements to be based on the plastic section modulus.

Table 6 Limiting width-to-thickness ratios for RHS subject to bending in order for the moment capacity to be based on the plastic section modulus

	Flange limiting width-to-thickness ratio (Element subject to compression)		Web limiting width-to-thickness ratio (Element subject to bending)	
	General expression	E_0= 200000 N/mm^2 f_y= 210 N/mm^2	General expression	E_0= 200000 N/mm^2 f_y= 210 N/mm^2
Europe	$26.7 \sqrt{\dfrac{235}{f_y}\dfrac{E_0}{210000}}$	27.6	$58.2 \sqrt{\dfrac{235}{f_y}\dfrac{E_0}{210000}}$	59.9
US[1]	Function of flange width, web depth, t, F_{yc} and C_y		$\dfrac{2.22}{\sqrt{\dfrac{f_y}{E_0}}}$ [2]	68.5
Australia	$\dfrac{1.11}{\sqrt{\dfrac{f_y}{E_0}}}$	34.3	$\dfrac{2.22}{\sqrt{\dfrac{f_y}{E_0}}}$ [2]	68.5
Note: (1) Moment capacity is 1.25 x elastic section modulus (2) Assuming that half of the web is in compression (i.e. the neutral axis is at the midpoint of the web)				

Table 7 compares the limiting diameter-to-thickness ratios in the different specifications for CHS subject to bending in order for the moment capacity to be based on the plastic section modulus (i.e. to be a Class 1 or 2 cross-section) and the elastic section modulus (i.e. to be a Class 3 cross-section). The European guidance and that in the AS/NZS Specification are identical, and permit much less conservative designs than the SEI/ASCE Specification (typically the ratio of plastic to elastic section modulus for a CHS is 1.3).

Table 7 Limiting diameter-to-thickness ratios for CHS subject to bending in order for the moment capacity to be based on the plastic and elastic section modulus

	Limiting diameter-to-thickness ratio to permit use of plastic moment capacity		Limiting diameter-to-thickness ratio to permit use of elastic moment capacity	
	General expression	E_0= 200000 N/mm^2 f_y= 210 N/mm^2	General expression	E_0= 200000 N/mm^2 f_y= 210 N/mm^2
Europe[1]	$0.078\dfrac{E_0}{f_y}$	74.3	$0.313\dfrac{E_0}{f_y}$	298
US	-	-	$0.112\dfrac{E_0}{f_y}$	107
Australia	$0.078\dfrac{E_0}{f_y}$	74.3	$0.313\dfrac{E_0}{f_y}$	298
Note (1) Guidance in the Second Edition of the Euro Inox Design Manual				

9   UNRESTRAINED MEMBERS SUBJECT TO BENDING

9.1   Europe

As with flexural buckling, the design approach for unrestrained members subject to bending about the major axis is based on the Perry-Robertson buckling curve, with different values for α and λ_0 to those applicable for flexural buckling. The reduction factor to be applied to the moment capacity to account for lateral torsional buckling, χ_{LT} is given by:

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \left[\varphi_{LT}{}^2 - \overline{\lambda}_{LT}{}^2\right]^{0.5}} \quad \leq \quad 1$$

in which

$$\varphi_{LT} = 0{,}5\left(1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - \overline{\lambda}_{0\ LT}\right) + \overline{\lambda}_{LT}{}^2\right) \text{ and } \quad \overline{\lambda}_{LT} = \lambda_{LT}\frac{1}{\pi}\sqrt{\frac{f_y\beta_{W,y}}{E}}$$

where:

$$\lambda_{LT} = \left[\pi^2 E_0 W_{pl,y}/M_{cr}\right]^{0.5}$$

in which:
α_{LT} is the imperfection factor
      = 0,34 for cold formed sections and hollow sections
      = 0,76 for welded open sections
$\overline{\lambda}_{0\ LT}$ is the limiting slenderness = 0.2 in ENV 1993-1-4, but subsequent research[13] led to a value of
      0.4 being recommended in the Second Edition of the Euro Inox Design Manual
$\beta_{W,y}$ = 1 for Class 1 or 2 cross-sections
      = $W_{el,y}/W_{pl,y}$ for Class 3 cross-sections
      = $W_{eff,y}/W_{pl,y}$ for Class 4 cross-sections
$W_{pl,y}$, $W_{el,y}$, $W_{eff,y}$ are defined in Section 8.1
M_{cr} is the elastic critical moment for lateral torsional buckling.

For a beam of uniform symmetrical cross-section with equal flanges subject to end moment loading and transverse loads applied at the shear centre and normal conditions of restraint at each end, M_{cr} can be calculated as:

$$M_{\mathrm{cr}} = C_1 \frac{\pi^2 E_0 I_z}{L^2} \left[\frac{I_w}{I_z} + \frac{L^2 G_0 I_t}{\pi^2 E_0 I_z} \right]^{0.5}$$

where:

C_1 is a factor depending on loading and end restraint conditions
L is the length of beam between points of lateral restraint
I_z is the moment of inertia about the minor axis
I_w is the warping constant
I_t is the torsional constant
G_0 is the initial shear modulus

9.2   US

In the SEI/ASCE Specification, the lateral torsional buckling capacity of a beam is calculated directly from the expression for the elastic critical moment for lateral torsional buckling, M_{cr} given in Section 9.1. However, to take into account the non-linear behaviour of stainless steel in the inelastic stress range, the initial elastic modulus E_0 and initial shear modulus G_0 are replaced by the tangent modulus E_t and tangent shear modulus G_t. An iterative approach is therefore required to determine the elastic critical moment.

9.3   Australia and New Zealand

The approach in the SEI/ASCE Specification is adopted.

9.4   Comparison of standards

The buckling curves for lateral torsional buckling in ENV 1993-1-4 (but with $\overline{\lambda}_{0\,LT} = 0.4$) are compared with those in the SEI/ASCE Specification for two values of n in Figure 11. At low and intermediate lengths the SEI/ASCE curve for n=4 is more conservative than the ENV 1993-1-4 cold-formed curve.
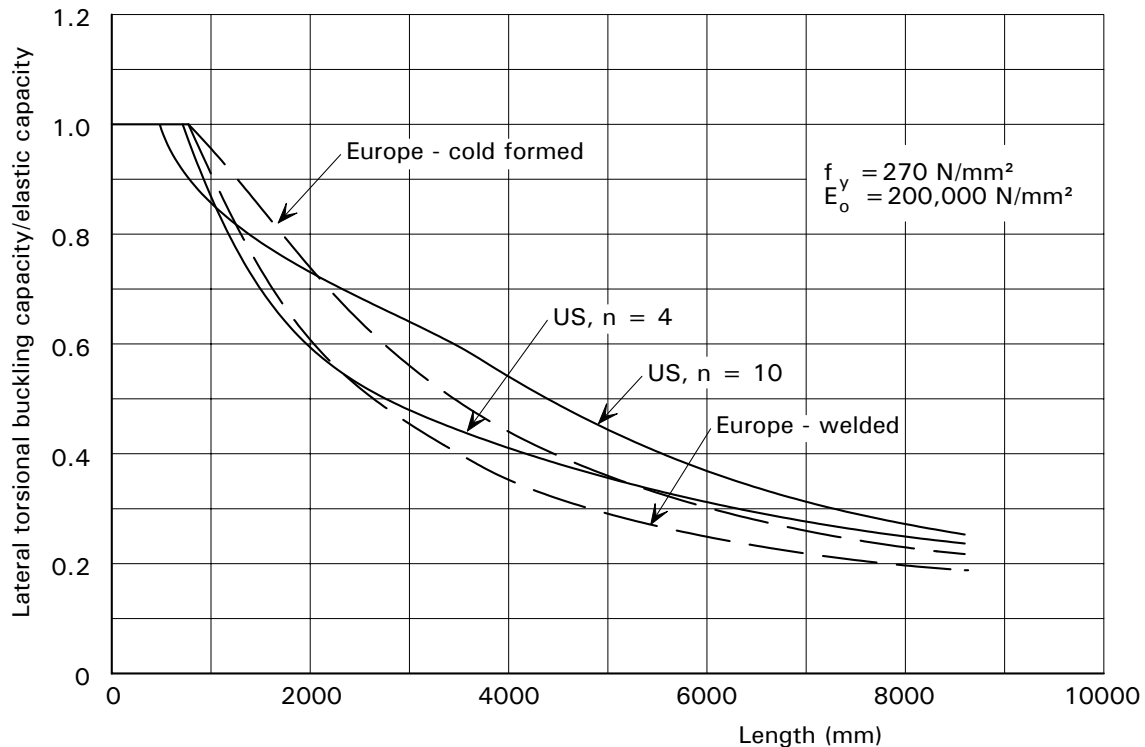
Figure 11   European buckling curves for lateral torsional buckling compared to SEI/ASCE buckling curves with Ramberg Osgood parameter n=4 and n=10

10   MEMBERS SUBJECT TO COMBINED LOADING

10.1   Europe
The approach taken for carbon steel in ENV 1993-1-1 is adopted.

10.2   US
The approach taken for carbon steel in the AISI Specification for cold-formed carbon steel[8] is adopted without modification.

10.3   Australia and New Zealand
The approach in the SEI/ASCE Specification is adopted.

11   FIRE RESISTANT DESIGN

11.1   Europe
ENV 1993-1-4 makes reference to the Eurocode covering the fire resistant design of steel structures, ENV 1993-1-2[14] and refers to the material specification EN 10088 for (very conservative) properties at elevated temperatures. Since ENV 1993-1-4 was issued, further research was carried out looking at the performance of stainless steel beams and columns in fire[13]. The Second Edition of the Euro Inox Design Manual includes the results of this research and gives mechanical and physical properties and a design approach for assessing the fire resistance of stainless steel beams and columns. During the conversion of ENV 1993-1-2 into full EN status, its scope has been extended to cover structural stainless steel members also, and the recently derived material properties are included in an Annex to this standard.

11.2   US
The SEI/ASCE Specification does not cover fire resistant design.

11.3   Australia and New Zealand
The AS/NZS Specification does not cover fire resistant design, but an Informative Appendix describes what guidance is available in ENV 1993-1-4.

12   FATIGUE RESISTANT DESIGN

12.1   Europe
The First Edition of the Euro Inox Design Manual recommended that all details in stainless steel be derated one fatigue detail classification from the equivalent detail in carbon steel. However, ENV 1993-1-4 states that the design approach for determining the fatigue strength for carbon steel in ENV 1993-1-1 (soon to be superseded by EN 1993-1-9[15]) is applicable. Subsequent research work confirmed this[13].

12.2   US
The SEI/ASCE Specification does not give any guidance on the assessment of the fatigue strength of stainless steel structures.

12.3   Australia and New Zealand
The AS/NZS Specification contains an Informative Appendix that describes what guidance is available in the First Edition of the Euro Inox Design Manual and ENV 1993-1-1. Since the fatigue provisions for carbon steel are similar to those in the Australian standard AS4100[16], the AS/NZS Specification recommends that AS4100 be used for fatigue design, within certain limits of applicability.

13   CONCLUSIONS

This paper is a comparison of structural stainless steel design standards. The European (Eurocode 3 ENV 1993-1-4), American (SEI/ASCE 8-02) and Australian/New Zealand (AS/NZS 4673) standards are reviewed in some detail. These standards use limit state principles as their design basis. ENV 1993-1-4 can be used for designing hot rolled, fabricated and cold-formed sections, whereas the scope of the SEI/ASCE and AS/NZS Specifications is confined to the design of cold-formed sections.

The grades of stainless steel covered by the standards, and their mechanical properties are compared. The SEI/ASCE and AS/NZS specifications give mechanical properties for the longitudinal direction (tension and compression) and the transverse direction (tension and compression). The mechanical properties in ENV 1993-1-4 are independent of the direction of rolling or stress. The SEI/ASCE Specification gives mechanical properties for material at four temper (cold work) levels. It is presumed that in the US, stainless steel material is used for structural applications at these temper levels. However, ENV 1993-1-4 and the AS/NZS specification focus on material in the annealed condition; reflecting the (current) minimal demand for cold worked material in structural applications in these regions. There is a wide variation in the magnitude of the Ramberg-Osgood parameter given in the standards.

The design guidance for cross-sections and members is explained and compared. The standards adopt the same approach for calculating the effective widths of slender elements in compression, although the European plate buckling curves are more conservative. The SEI/ASCE Specification gives more conservative guidance on the calculation of the moment capacity of restrained beams, particularly for CHS and its method for calculating the inelastic reserve capacity requires more calculation effort than the other standards.

One important difference is that SEI/ASCE and AS/NZS specifications adopt the tangent modulus method for calculating the buckling strength of members, which generally requires iteration to find a solution. The shape of the buckling curve varies with the magnitude of the Ramberg-Osgood parameter, n, becoming more conservative as the value of n reduces (i.e. as the stress-strain curve becomes more non-linear). By comparison, the Eurocode design curves were derived by calibration against experimental data; they use the initial modulus of elasticity and avoid the need for iteration.

All the standards adopt guidance developed for carbon steel for verifying the resistance of a member to combined axial loading and bending.

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