Steel Design Guide

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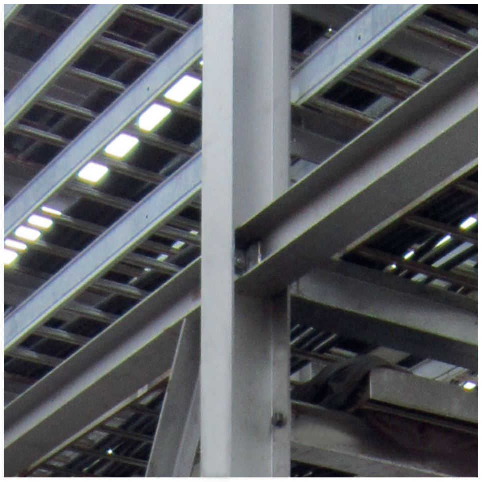
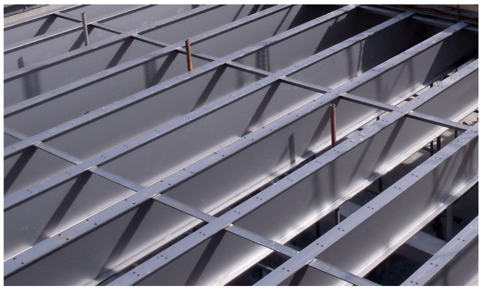
# Structural Stainless Steel





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Steel Design Guide

# Structural Stainless Steel

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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**Philippe Richard (ICDA)**

The International Chromium Development Association seeks to promote and increase end-use applications of chromium, an essential alloy in all stainless steels.

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# Chapter 1 Introduction

## 1.1 WHAT IS STAINLESS STEEL?

Stainless steel is the name given to a family of corrosion and heat resistant steels containing a minimum of 10.5% chromium. Just as there are various structural and engineering carbon steels meeting different strength, weldability and toughness requirements, there is also a wide range of stainless steels with varying levels of corrosion resistance and strength. This array of stainless steel properties is the result of controlled alloying element additions, each affecting specific attributes of strength and ability to resist different corrosive environments. To achieve the optimal economic benefit, it is important to select a stainless steel which is adequate for the application without being unnecessarily highly alloyed and costly.

With a combination of the chromium content above 10.5%, a clean surface and exposure to air or any other oxidizing environment, a transparent and tightly adherent layer of chromium-rich oxide forms spontaneously on the surface of stainless steel. If scratching or cutting damages the film, it reforms immediately in the presence of oxygen. Although the film is very thin, about $0.2 \times 10^{-6}$ in. ($5 \times 10^{-6}$ mm), it is both stable and nonporous and, as long as the type of stainless steel is corrosion resistant enough for the service environment, it will not react further with the atmosphere. For this reason, it is called a passive film. The stability of this passive layer depends on the composition of the stainless steel, its surface treatment, and the corrosiveness of its environment. Its stability increases as the chromium content increases and is further enhanced by alloying additions of molybdenum and nitrogen.

Stainless steels can be classified into the following five basic groups, with each group providing unique properties and a range of different corrosion resistance levels.

### Austenitic stainless steels

The most widely used types of austenitic stainless steel are based on 17 to 18% chromium and 8 to 11% nickel additions. In comparison to structural carbon steels, which have a body-centered cubic atomic (crystal) structure, austenitic stainless steels have a different, face-centered cubic atomic structure. As a result, austenitic stainless steels, in addition to their corrosion resistance, have high ductility, are easily cold-formed, and are readily weldable. Relative to structural carbon steels, they also have significantly better toughness over a wide range of temperatures. They can be strengthened by cold working, but not by heat treatment. Their corrosion performance can be further enhanced by higher levels of chromium and additions of molybdenum and nitrogen.

### Ferritic stainless steels

The chromium content of the most popular ferritic stainless steels is between 10.5% and 18%. Ferritic stainless steels contain either no or very small nickel additions and their body-centered atomic structure is the same as that of structural carbon steels. They are generally less ductile, less formable and less weldable than austenitic stainless steels. They can be strengthened by cold working, but to a more limited degree than the austenitic stainless steels. Like the austenitics, they cannot be strengthened by heat treatment and can be used in a broad range of corrosive environments. They have good resistance to stress corrosion cracking and their corrosion performance can be further enhanced by additions of molybdenum.

### Duplex stainless steels

Duplex stainless steels have a mixed microstructure of austenite and ferrite, and so are sometimes called austenitic-ferritic steels. They typically contain 20 to 26% chromium, 1 to 8% nickel, 0.05 to 5% molybdenum, and 0.05 to 0.3% nitrogen. They provide higher strength levels than austenitic steels and are suitable for a broad range of corrosive environments. Although duplex stainless steels have good ductility, their higher strength results in more restricted formability compared to the austenitics. They can also be strengthened by cold working, but not by heat treatment. They have good weldability and good resistance to stress corrosion cracking.

### Martensitic stainless steels

Martensitic stainless steels have a similar body-centered cubic structure as ferritic stainless steel and structural carbon steels, but due to their higher carbon content, they can be strengthened by heat treatment. Martensitic stainless steels are generally used in a hardened and tempered condition, which gives them high strength and provides moderate corrosion resistance. They are used for applications that take advantage of their wear and abrasion resistance and hardness, like cutlery, surgical instruments, industrial knives, wear plates and turbine blades. They are less ductile and more notch sensitive than the ferritic, austenitic and duplex stainless steels. Although most martensitic stainless steels can be welded, this may require preheat and postweld heat treatment, which can limit their use in welded components.

**Precipitation hardening stainless steels**

Precipitation hardening steels can be strengthened by heat treatment to very high strengths and fall into three microstructure families depending on the type: martensitic, semi-austenitic and austenitic. These steels are not normally used in welded fabrication. Their corrosion resistance is generally better than the martensitic stainless steels and similar to the 18% chromium, 8% nickel austenitic types. Although they are mostly used in the aerospace industry, they are also used for tension bars, shafts, bolts and other applications requiring high strength and moderate corrosion resistance.

## 1.2 APPLICATIONS OF STAINLESS STEELS IN THE CONSTRUCTION INDUSTRY

Stainless steels have been used in construction ever since they were invented over 100 years ago. They are attractive and highly corrosion resistant, while having good strength, toughness and fatigue properties in combination with low maintenance requirements. Stainless steels can be fabricated using a wide range of commonly available engineering techniques and are fully recyclable at the end of their useful life. They are also hygienic and easily cleaned.

Stainless steel is the material of choice in applications situated in aggressive environments; for example, structures in proximity to saltwater, exposed to deicing salts, or in very heavily polluted locations. The high ductility of stainless steels is a useful property where resistance to seismic loading is required since greater energy dissipation is possible; however, seismic applications are outside the scope of this Design Guide. They are commonly used in industrial structures for the water treatment, pulp and paper, nuclear, biomass, chemical, pharmaceutical, and food and beverage industries. The industrial structural applications include platforms, barriers/gates and equipment supports.

Stainless steel is also used for pedestrian and vehicular bridge components exposed to aggressive environments. The number of pedestrian bridges where stainless steel is a primary structural component is steadily increasing. There are vehicular bridges where stainless steel is the primary structural component, but the most common applications are concrete reinforcing bar, seismic components or retrofits, cable sheathing, expansion joints, pins, bumper structural supports, and railings and stair components. Seawalls, piers, parking garages and other structures exposed to high levels of coastal or deicing salts are increasingly making use of stainless steel structural components.

In aesthetic buildings and structure exteriors, stainless steel structural components are a popular choice for supporting low profile and other glass curtain wall designs, roofs, canopy supports, seismic components, security barriers and other applications that take advantage of the material's corrosion resistance and strength to reduce maintenance requirements and improve safety. They are widely used for hand railing and street furniture for the same reasons. The good corrosion resistance of stainless steels makes them ideal materials for wood and masonry fasteners, anchoring systems and support angles because wood and masonry can be inherently corrosive to other metals and moisture and corrosive chemical absorption over time is likely. Additionally, these types of components are often inaccessible or difficult to replace. Excellent corrosion resistance and good strength means stainless steels are also suitable for applications in soil or stone, such as tunnel linings, security and other fencing, and retaining walls.

In swimming pools, stainless steels are used both for architectural and structural applications such as pool liners, handrails, ladders, structural components, fasteners, furniture, diving structures, decorative items, and water treatment and ventilation systems. Special precautions should however be taken for structural components in swimming pools due to the risk of stress corrosion cracking in areas where condensates may form (see Section 2.6.2).

Stainless steels can absorb considerable impact without fracturing due to their high strength, ductility, and strain hardening characteristics. This makes them suitable for explosion and impact resistant structures such as blast and security walls, gates, and bollards.

The greater corrosion resistance, heat resistance, and strength of some highly alloyed austenitic, ferritic and duplex stainless steels make them suitable for demanding industrial and saltwater spray, splashing and immersion applications—like offshore platforms and for down-hole oil flow applications. On offshore platforms, stainless steel offers a low maintenance, lightweight, fire- and explosion-resistant solution for blast walls, cable ladders and walkways. In these applications, the life cycle cost savings are an important benefit, while any weight saved in the structure and equipment is an important advantage in the overall project cost.

Figure 1-1 to Figure 1-9 show examples of structural applications.

The 630 ft (192 m) high Gateway Arch in St. Louis, Missouri (Figure 1-1), inspired a great amount of research into the structural performance of stainless steel in the United States in the early 1960s. It was the first very large structural application of stainless steel, using 804 tons of ¼-in.-thick (6 mm) Type S30400. The cross section is a hollow, equilateral triangle. The exterior structural skin is stainless steel plate and the interior is carbon steel plate.

The Gatineau Preservation Centre was designed for 500 year service with minimal material replacement (Figure 1-2). The structural support for the outer building, which carries the roof structure, consists of 34 80-ft-tall (24-m) stainless steel towers connected by curved beams. The vaults that form the inner building and house the archived materials are reinforced concrete. The choice of materials and design provides added protection from the environment, terrorism, fire, vermin and water. In total, 1,320 tons of S30403 and S31603 stainless steel were used. After construction, the components were glass bead blasted to make their appearance more consistent.

Figure 1-3 shows the Schubert Club Band Shell, constructed using a saddle-shaped Type S31600 stainless steel lattice grid to resist high winds, deicing salt from a nearby highway bridge, and seasonal flooding.

Figure 1-4 shows a staircase in a brewery using Type S30400 stainless steel hot rolled I-shaped members as the staircase stringers. In food, beverage and pharmaceutical production, there has been a large investment in stainless steel for staircases, platforms and supports over the past 10 to 15 years in an effort to obtain long-term cost reduction, particularly in corrosive manufacturing environments. The use of stainless steel also avoids secondary contamination concerns from peeling paint and carbon steel corrosion.

Figure 1-5 shows some of the 80 stainless steel I-shaped members made from duplex stainless steel which support the

*Fig. 1-1. Gateway Arch, St. Louis, Missouri. (Photo courtesy of Catherine Houska.)*

lamella clarifers in the first water desalination plant in the UK, at the Thames Gateway Water Treatment Works. The beams were initially specified to be carbon steel with an epoxy coating. However, there was a high risk of damage to the epoxy coating during service and maintenance, and the subsequent carbon steel corrosion would have damaged the expensive desalination membranes. Duplex S32205 stainless steel was specified instead, because it is approved for contact with drinking water, requires little maintenance, and is corrosion resistant in brackish water without any applied coating. The higher installed cost was offset by the long-term cost savings, including low maintenance requirements and greater assurance of water quality throughout the plant's design life of at least 60 years.

The stainless steel Air Force Memorial sculpture reaches a height of 270 ft (82 m) and is one of the world's largest stainless steel structures in terms of both height and tonnage (Figure 1-6). The structure is made from Type S31603 plate

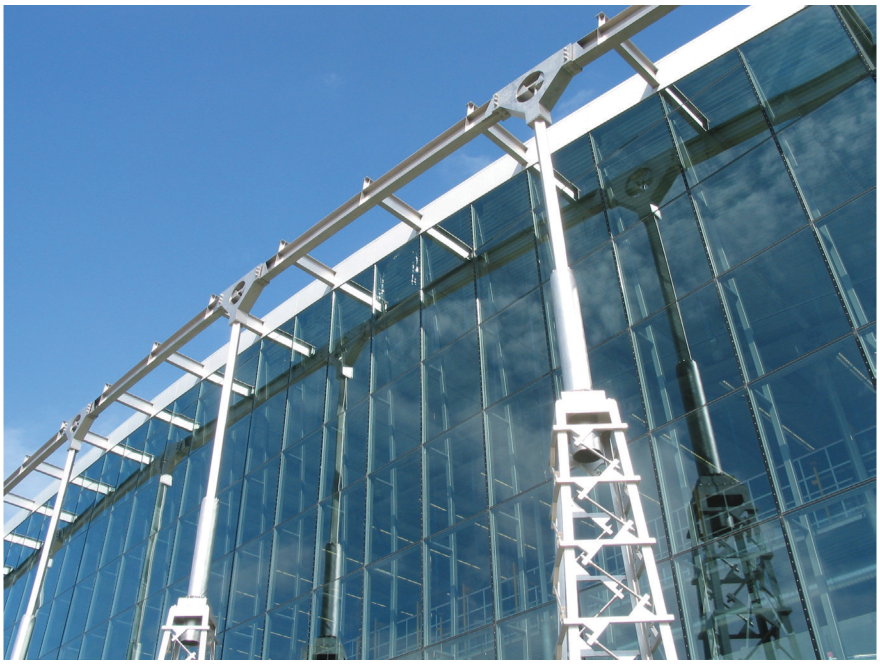


*Fig. 1-2. Gatineau Preservation Centre, Canadian National Archive.*
*(Photo courtesy of Library and Archives Canada.)*

*Fig. 1-3. Schubert Club Band Shell, Raspberry Island, St. Paul, Minnesota.*
*[Photo courtesy of Skidmore, Owings & Merrill LLP (left), James Carpenter Design Associates and Shane McCormick (right).]*

of ¾ in. (19 mm) thickness. This type of stainless steel was chosen because the memorial is subjected to deicing salts from the adjacent highways and the chosen polished, pickled and glass-bead blasted surface finish was relatively rough. This surface finish was required in order to achieve the right level of reflectivity—low enough in daytime to avoid dazzling pilots landing aircraft nearby and high enough at night to provide illumination. (A rough finish retains more salt, dust and pollutants, necessitating a higher level of corrosion resistance.)

The glass roof of the entrance to Brooklyn Museum of Art is supported by interior twin-armed structural castings made from precipitation hardening stainless steel Type S17400, in the H1150 heat treatment condition (Figure 1-7). The tension bars are cold drawn Type S30400 and the main beams are painted carbon steel.

Figure 1-8 shows a typical small-scale architectural structural application in which a large sign is supported from a building by a stainless steel system comprising Type S31600 plate and Type S30400 cold drawn tension rods. The tension rods are drawn to a yield strength of 110 ksi (760 MPa) and ultimate tensile strength of 130 ksi (900 MPa).

Figure 1-9 shows the stainless steel tension bar system at the Corning Museum of Glass.

*Fig. 1-4. Stainless steel staircase in brewery. (Photo courtesy of Stainless Structurals LLC.)*

*Fig. 1-5. Thames Gateway Water Treatment Works, UK. (Photo courtesy of Interserve.)*

*Fig. 1-6. Air Force Memorial. (Photo courtesy of Patrick McCafferty, Arup.)*

## 1.3 SCOPE OF THIS DESIGN GUIDE

This Design Guide is written for engineers experienced in the design of carbon steel structural components but not necessarily in the design of stainless steel structures. It is aligned with the design provisions in the 2010 AISC *Specification for Structural Steel Buildings* (AISC, 2010c), hereafter referred to as the AISC *Specification.* The major difference between the mechanical properties of carbon and stainless steel is the stress-strain relationship—stainless steel has a continuous, but nonlinear, relationship between stress and strain, while carbon steel has a clearly defined yield point. The nonlinear stress-strain curve means that in some cases different design expressions are applicable to stainless steel; for example, buckling curves for columns and unrestrained beams.

The guidance is based on the provisions in the European *Design Manual for Structural Stainless Steel* (Euro Inox and SCI, 2006a) and *Eurocode 3: Design of Steel Structures, Supplementary Rules for Stainless Steels,* Part 1-4 (CEN, 2006a). This Design Guide follows the guidance in the AISC *Specification* as closely as possible and the recommendations have, as much as possible, been harmonized with it.

This guide applies to the design of structural hot-rolled or welded open sections, such as I-shaped members, channels, and equal-leg angles. It also applies to rectangular and round hollow structural sections (HSS). Guidance on the design of cold-formed structural stainless steel members is available from ASCE/SEI 8, *Specification for the Design of Stainless Steel Cold-Formed Structural Members* (ASCE, 2002), hereafter referred to as ASCE/SEI 8, which covers austenitic and ferritic stainless steels but does not preclude its application to other stainless steels.

The guidance provided by this document is applicable to austenitic, duplex and precipitation hardening stainless steel structural sections with thickness ⅛ in. (3 mm) and greater. Reference should be made to ASCE/SEI 8 for the design of thinner stainless steel structural sections.

The guide is intended for the design of primary and secondary structural components. It covers aspects of material behavior and selection, cross section design, member design, connections, and fabrication. For special structures, such as those in nuclear installations or pressure vessels, additional requirements may need to be considered. The design of certain structural sections under unusual loading scenarios has been omitted; for example, angles and tees in flexure, sections in flexure where the web is classified as slender, unequal leg angles, equal leg angles with a slender cross section, and round HSS with a slender cross section.

The guidance in this publication applies to austenitic and duplex stainless steels; these are the most appropriate stainless steels for welded, hot-rolled or extruded structural shapes. In addition to this, some guidance is given on the use of a precipitation hardening stainless steel for tension members, fittings and fasteners.

Specific guidance on the use of stainless steel in the water industry and in architecture is available at **www.stainlesswater.org**, **www.stainlessarchitecture.org** and **www.imoa.info**.

Guidance on loads, load combinations, system limitations, and general design requirements is given in ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010).

*Fig. 1-7. Brooklyn Museum of Art. (Photo courtesy of TriPyramid Structures, Inc.)*

*Fig. 1-8. Stainless steel sign support, Tampa (Photo courtesy of TriPyramid Structures, Inc.)*

*Fig. 1-9. Corning Museum of Glass, Corning, New York. (Photo courtesy of TMR Consulting.)*

# Chapter 2
# Materials: Properties, Selection and Durability

## 2.1 BASIC STRESS-STRAIN BEHAVIOR

The stress-strain behavior of stainless steels differs from that of carbon steels in a number of aspects. The most important difference is in the shape of the stress-strain curve. While carbon steel typically exhibits linear elastic behavior up to the yield stress and a plateau before strain hardening is encountered, stainless steel has a more rounded response with no well-defined yield stress. Therefore, stainless steel "yield" strengths are generally defined for a particular offset permanent strain (conventionally the 0.2% strain), as indicated in Figure 2-1 which shows typical experimental stress-strain curves for common austenitic and duplex stainless steels. The curves shown are representative of the range of material likely to be supplied and should not be used in design. The proportional limit of stainless steels ranges from 40 to 70% of the 0.2% offset yield strength. Figure 2-2 shows typical stress-strain curves to failure.

Strength levels of austenitic and duplex stainless steels are enhanced by cold work (such as imparted during cold-forming operations, including roller leveling/flattening and fabrication). As strength increases with cold work, there is a reduction in ductility. Since the initial ductility is so high, this normally has only a minor influence on design, especially for the austenitic stainless steels. During the fabrication of an HSS, the 0.2% offset yield strength increases by about 50% in the cold-formed corners of cross sections.

As well as nonlinearity, the stress-strain characteristics of stainless steels also display nonsymmetry of tensile and compressive behavior and anisotropy (differences in behavior of coupons aligned parallel and transverse to the rolling direction). Tests on hot rolled material indicate higher strengths transverse to the rolling direction than in the direction of rolling. In general, anisotropy and nonsymmetry increase with cold work. For the structural sections covered by this Design Guide, which are not made from heavily cold worked material, the differences in the stress-strain behavior due to nonsymmetry and anisotropy are not large; the nonlinearity has a more significant effect. Anisotropy and nonsymmetry is more significant in the design of lighter gage, heavily worked sections, which are covered by ASCE/SEI 8.

The design strengths recommended in this Design Guide are the minimum values specified in the relevant ASTM product specifications. The direction(s) in which tensile testing is required by ASTM varies with the product form and, in some cases, the stainless steel alloy family. It is most commonly in the rolling direction. Specifiers may require additional testing but this must be agreed in advance with the supplier and additional testing costs expected.

The high tensile strengths (and hardnesses) of precipitation hardening stainless steels come from a heat treatment process which can lead to strength levels several times that of the austenitics. The most common heat treatment conditions are H900, H1025 and H1150, where 900 °F (482 °C), 1,025 °F (552 °C) and 1,150 °F (621 °C) are the suggested temperatures at which hardening and/or aging treatment are carried out. The material strength reduces and impact toughness increases with the aging temperature; values are given in ASTM A564/A564M (ASTM, 2010b).

Stainless steels can absorb considerable impact without fracturing due to their excellent ductility (especially the austenitic stainless steels) and their strain hardening characteristics.

## 2.2 SUITABLE STAINLESS STEELS FOR STRUCTURAL APPLICATIONS

Austenitic stainless steels are generally selected for structural applications, which require a combination of good strength, corrosion resistance, formability (including the ability to make tighter bends), excellent field and shop weldability and, for seismic applications, excellent elongation prior to fracture. Where high strength, corrosion resistance, and/or higher levels of crevice and stress corrosion cracking resistance are required, duplex stainless steels are most suitable. In many cases, the high strength of duplex stainless steel can make section size reduction possible.

This Design Guide applies to the austenitic, duplex and precipitation hardening stainless steels that are most commonly encountered in structural applications (see Table 2-1 for applicable ASTM standards). Only the rolled versions, as opposed to the cast versions, are considered. [Note that ASTM A351/A351M (ASTM, 2012d) covers austenitic castings and ASTM A890/A890M (ASTM, 2012g) covers duplex stainless steel castings. A different naming system is used for castings. The properties of castings may be different from their rolled versions, e.g., austenitic stainless steel castings may be slightly magnetic.] The design rules in this Design Guide may also be applied to other austenitic, duplex and precipitation hardening stainless steels in the ASTM standards given in Table 2-1. The austenitic and duplex alloys considered should have a minimum elongation of 20%. However, the advice of a stainless steel producer or consultant should be sought regarding the durability,

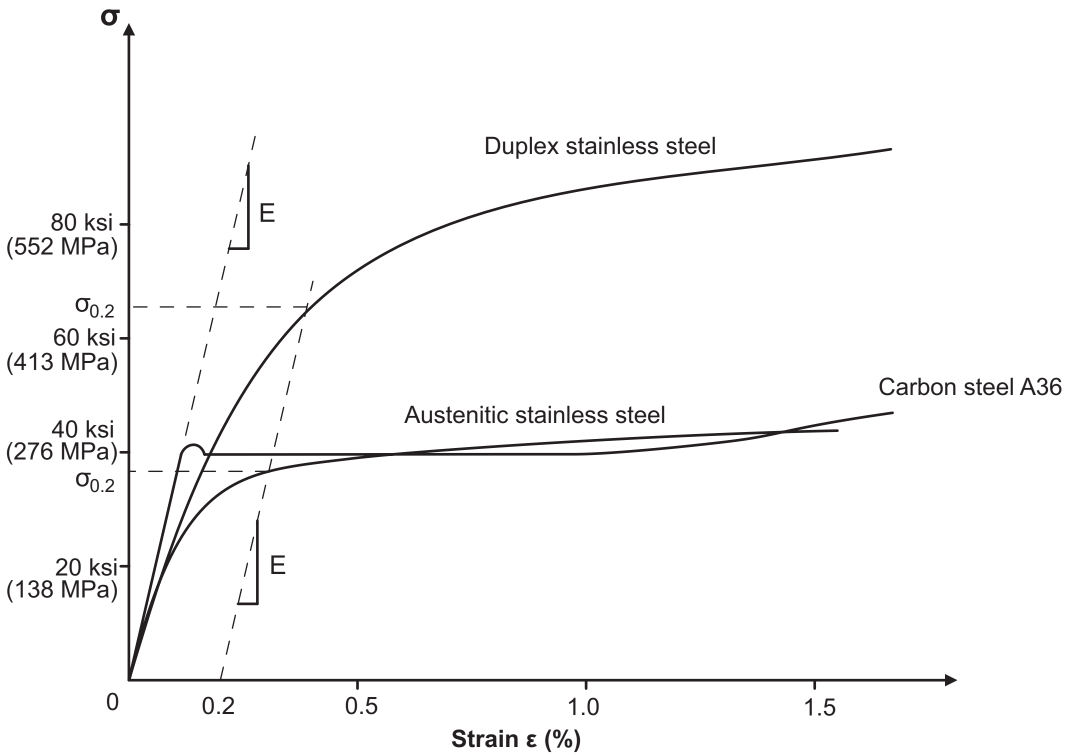



*Fig. 2-1. Typical stress-strain curves for stainless and carbon steel in the annealed (softened) condition.*

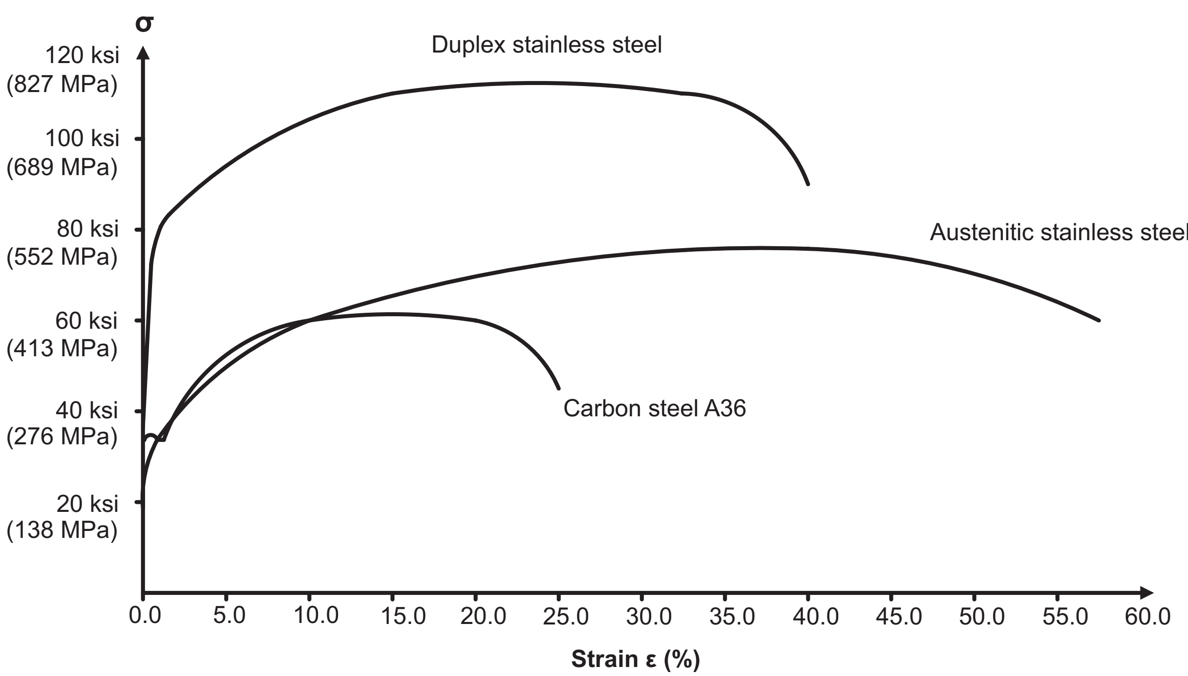



*Fig. 2-2. Typical stress-strain curves for stainless and carbon steel to failure.*

fabrication, weldability, fatigue resistance, and high temperature performance of other alloys. Guidance on selection of stainless steels for particular applications is given in Section 2.5.

The following stainless steels are addressed in this Design Guide.

**Austenitic stainless steels**

UNS S30400 (304) and UNS S30403 (304L)

UNS S31600 (316) and UNS S31603 (316L)

Types S30400 and S30403 are the most commonly used standard austenitic stainless steels and contain 18 to 20% chromium and 8 to 11% nickel. Types S31600 and S31603 contain about 16 to 18% chromium, 10 to 14% nickel and the addition of 2 to 3% molybdenum, which improves corrosion resistance.

Note: The 'L' in the designation indicates a low carbon version with reduced risk of sensitization (of chromium carbide precipitation) and of intergranular corrosion in heat-affected zones of welds; they should be specified for the welded sections covered by this document. Low carbon does not affect corrosion performance beyond the weld areas. When producers use state-of-the-art production methods, commercially produced stainless steels are often low carbon and dual certified to both designations (e.g., S30400/S30403, with the higher strength of S30400 and the lower carbon content of S30403). When less modern technology is used, this cannot be assumed and there may be a price premium for the low carbon, L, specification. Therefore, the low carbon version should be explicitly specified in the documents of projects in which welding is involved.

**Duplex stainless steels**

UNS S32101 (LDX 2101®), representative of proprietary lean duplexes*

UNS S32304 (2304)

UNS S32205 (2205)

**Precipitation hardening stainless steels**

UNS17400 (630), also known as 17-4

This martensitic precipitation hardening stainless steel is the most commonly used within this family. Due to the high strength of precipitation hardening stainless steels, most applications are in the aerospace and other high technology industries. However, they are occasionally used in construction for tension members and fasteners, where very high strength with good corrosion resistance is required.

## 2.3 MECHANICAL PROPERTIES

### 2.3.1 Standards for Flat and Long Products

Table 2-1 gives the relevant ASTM Specifications for the stainless steels listed in Section 2.2 which are covered by this Design Guide.

Table 2-2 gives minimum specified mechanical properties of the stainless steels covered in this Design Guide according to the relevant ASTM Specifications (see also the Notes at the bottom of the table). The chemical compositions are given in Table 2-3.

Austenitic stainless steels are not susceptible to brittle fracture, even at low temperatures; they are widely used for cryogenic applications and demonstrate impact toughness well above 74 ft-lbf (100 J) at –320 °F (–196 °C).

Duplex stainless steels also have adequate toughness for most low temperature applications, e.g., lean duplex S32101 shows an impact toughness of at least 30 ft-lbf (40 J) in base and weld metal at –58 °F (–50 °C) for 1.2 in. (30 mm) material. The more highly alloyed duplexes show even better toughness.

### 2.3.2 Standards for Bolts

The most common standards giving the chemical compositions and mechanical properties of smaller diameter (up to 1½ in. or 36 mm) austenitic, ferritic and precipitation hardening stainless steel bolts for general corrosion resistance service applications (excluding low or high temperature, high strength, pressure rated equipment, or other special service) are as follows:

**ASTM F593, *Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs* (ASTM, 2008a)**

This standard includes austenitic, ferritic, martensitic and precipitation hardening stainless steels for general corrosion resistance. Group numbers indicate that fasteners are chemically equivalent for general purpose use. S30400/S30403 bolts are classified as Alloy Group 1 and S31600/S31603 are Alloy Group 2 bolts. The standard covers bolts up to 1½ in. diameter. [The corresponding standard for nuts is ASTM F594, *Standard Specification for Stainless Steel Nuts* (ASTM, 2009c).]

* LDX 2101® is a proprietary lean duplex stainless steel. Lean duplexes contain less nickel and molybdenum relative to other alloys of similar corrosion resistance, which reduces alloy costs and improves price stability. They possess the high strength characteristic of duplex stainless steels. Their corrosion resistance lies between that of S30400 and S31600 austenitic stainless steels. In this Design Guide, the properties of S32101 are considered to be representative of this group.

| Table 2-1. Specifications for Stainless Steel Flat and Long Products | | | |
|---|---|---|---|
| **Shape** | **ASTM Specification No.** | **Title** | **Description** |
| Plate, sheet, strip | ASTM A240/ A240M | Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications | Chemical composition and mechanical properties for plate, sheet and strip. |
| | ASTM A480/ A480M | Standard Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip | Sheet, strip and plate finishes, dimensional tolerance, flatness, and shipping requirements |
| Hollow sections | ASTM A312/ A312M | Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes | Chemical composition, mechanical properties and dimensional tolerance requirements for round austenitic stainless steel pipe. (The added testing requirements make this product more expensive than mechanical tube produced to ASTM A554.) |
| | ASTM A554 | Standard Specification for Welded Stainless Steel Mechanical Tubing | Chemical composition, dimensional, straightness and other tolerances for round, square, and rectangular austenitic and ferritic stainless steel tubing. [This is the most commonly used standard for hollow structural applications. It covers sizes up to 16 in. (406 mm) OD and wall thicknesses of 0.020 in. (0.51 mm) and over.] |
| | ASTM A789/ A789M | Standard Specification for Seamless and Welded Ferritic/ Austenitic Stainless Steel Tubing for General Service | Chemical composition, mechanical properties and dimensional tolerance requirements for duplex stainless steel tubing (duplexes are not currently covered by ASTM A554). |
| | ASTM A790/ A790M | Standard Specification for Seamless and Welded Ferritic/ Austenitic Stainless Steel Pipe | Chemical composition, mechanical properties, and dimensional tolerance requirements for round duplex stainless steel pipe. |
| Bars and shapes | ASTM A276 | Standard Specification for Stainless Steel Bars and Shapes | Chemical composition and mechanical properties for bars, including rounds, squares, and hot-rolled or extruded shapes such as angles, tees and channels. |
| | ASTM A479/479M | Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels | Chemical composition and mechanical properties for hot- and cold-finished bars of stainless steel, including rounds, squares and hexagons, and hot-rolled and extruded shapes such as angles, tees and channels for use in boiler and pressure vessel construction. |
| | ASTM A484/ A484M | Standard Specification for General Requirements for Stainless Steel Bars, Billets, and Forgings | Dimensional tolerance, straightness, and finish descriptions for hot- or cold-finished bar, squares, angles, channels, tees and other shapes. The finish descriptions are very general. |
| | ASTM A564/564M | Standard Specification for Hot-Rolled and Cold-Finished Age-Hardening Stainless Steel Bars and Shapes | Chemical composition and mechanical properties for hot- or cold-finished rounds, squares, hexagons, bar shapes, angles, tees and channels. |
| | ASTM A1069/ A1069M | Standard Specification for Laser-Fused Stainless Steel Bars, Plates, and Shapes | Ordering information, manufacture, materials etc. relating to laser-fused stainless steel bars, plates, and shapes of structural quality for use in bolted or welded structural applications. (Note: Laser fusion is a laser welding process without the use of filler material.) |

Notes:
1. The current version of the standard should be used at the time the project specification is submitted to bid.
2. ASTM A240/A240M (ASTM, 2012c), ASTM A276 (ASTM, 2010a), or ASTM A479/A479M (ASTM, 2012e) should be referenced when specifying the chemical composition and mechanical property requirements for all laser fused fabrications [ASTM A1069/A1069M (ASTM, 2011b)].

| Table 2-2. Minimum Specified Mechanical Properties of Common Stainless Steels | | | | | | | |
|---|---|---|---|---|---|---|---|
| Group of Steels | Type | Heat Treatment Condition | $F_u$ | | $F_y$ | | Minimum Elongation in 2 in. (50 mm) |
| | | | ksi | MPa | ksi | MPa | % |
| Basic chromium-nickel austenitic stainless steels | S30400 | — | 75 | 515 | 30 | 205 | 40 |
| | S30403 | — | 70 | 485 | 25 | 170 | 40 |
| Molybdenum-chromium-nickel austenitic stainless steels | S31600 | — | 75 | 515 | 30 | 205 | 40 |
| | S31603 | — | 70 | 485 | 25 | 170 | 40 |
| Duplex stainless steels | S32101 | — | 94[a] | 650[a] | 65[a] | 450[a] | 30 |
| | S32304 | — | 87 | 600 | 58 | 400 | 25 |
| | S32205 | — | 95 | 655 | 65 | 450 | 25 |
| Precipitation hardening stainless steels | S17400 | H900 | 190 | 1310 | 170 | 1170 | 10 |
| | | H1025 | 155 | 1070 | 145 | 1000 | 12 |
| | | H1150 | 135 | 930 | 105 | 725 | 16 |

[a] These values apply to material of thickness > 0.187 in. (5 mm). For material of thickness ≤ 0.187 in. (5 mm), the min. tensile strength is 101 ksi (700 MPa) and the minimum yield strength is 77 ksi (530 MPa).
Notes:
The values are taken from ASTM 240/A240M for the austenitic and duplex stainless steels. ASTM A276 gives identical values for the stainless steels included in this table. The values are taken from ASTM A564/A564M (ASTM, 2010b) for the precipitation hardening stainless steel.

| Table 2-3. Chemical Composition | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Group of Steels | Type | Content of Alloying Element (Maximum or Range) Weight, % | | | | | | | | | |
| | | Carbon | Manganese | Phosphorus | Sulfur | Silicon | Chromium | Nickel | Molybdenum | Nitrogen | Copper |
| Austenitic stainless steels | S30400 | 0.070 | 2.00 | 0.045 | 0.030 | 0.75 | 17.5 – 19.5 | 8.0 – 10.5 | — | 0.10 | — |
| | S30403 | 0.030 | 2.00 | 0.045 | 0.030 | 0.75 | 17.5 – 19.5 | 8.0 – 12.0 | — | 0.10 | — |
| | S31600 | 0.080 | 2.00 | 0.045 | 0.030 | 0.75 | 16.0 – 18.0 | 10.0 – 14.0 | 2.00 – 3.00 | 0.10 | — |
| | S31603 | 0.030 | 2.00 | 0.045 | 0.030 | 0.75 | 16.0 – 18.0 | 10.0 – 14.0 | 2.00 – 3.00 | 0.10 | — |
| Duplex stainless steels | S32101 | 0.04 | 4.0 – 6.0 | 0.04 | 0.03 | 1.00 | 21.0 – 22.0 | 1.35 – 1.70 | 0.10 – 0.80 | 0.20 – 0.25 | 0.10 – 0.80 |
| | S32304 | 0.030 | 2.50 | 0.040 | 0.030 | 1.00 | 21.5 – 24.5 | 3.0 – 5.5 | 0.05 – 0.60 | 0.05 – 0.20 | 0.05 – 0.60 |
| | S32205 | 0.030 | 2.00 | 0.030 | 0.020 | 1.00 | 22.0 – 23.0 | 4.5 – 6.5 | 3.0 – 3.5 | 0.14 – 0.20 | — |
| Precipitation hardening stainless steels | S17400 | 0.070 | 1.00 | 0.040 | 0.030 | 1.00 | 15.0 – 17.5 | 3.0 – 5.0 | — | — | 3.0 – 5.0 |

Notes:
The values are taken from ASTM 240/A240M for the austenitic and duplex stainless steels. ASTM A276 gives very similar values for the stainless steels described in this table. The values are taken from ASTM A564/A564M for the precipitation hardening stainless steel. Type S17400 additionally contains columbium (niobium) and tantalum for a total of 0.15 to 0.45%.

**ASTM F738M,** ***Standard Specification for Stainless Steel Metric Bolts, Screws, and Studs*** **(ASTM, 2008b)**

This is the metric equivalent of ASTM F593 for general corrosion resistance. S30400/S30403 bolts are designated property Class A1 and S31600/S31603 bolts as A4. The standard covers bolts up to 36 mm diameter. [The corresponding standard for nuts is ASTM F836M, *Standard Specification for Style 1 Stainless Steel Metric Nuts* (ASTM, 2010c).]

The following standards are used for specialized applications like low or high temperature, high strength, high pressure, and other specialized applications and include austenitic, ferritic, duplex and precipitation hardening stainless steels. They are also used when specifying bolt diameters over 1½ in. or 36 mm.

**ASTM A320/A320M,** ***Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service*** **(ASTM, 2011a)**

This standard covers austenitic and ferritic stainless steels and is intended specifically for low temperature service, whether the application is structural or a piece of equipment. Type S30400 bolts are designated as B8 and B8A and Type S31600 as B8M and B8MA. The standard covers bolts up to 1½ in. (38 mm) diameter. [The corresponding standard for nuts is ASTM A962/A962M, *Standard Specification for Common Requirements for Bolting Intended for Use at Any Temperature from Cryogenic to the Creep Range* (ASTM, 2012h).]

**ASTM A193/A193M,** ***Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications*** **(ASTM, 2012a)**

This standard covers austenitic and ferritic stainless steel bolting for high temperature or high pressure service, or other special purpose applications. It includes both metric and U.S. customary system units. This is the only standard which can be used for ordering stainless steel bolts in larger diameters. [The corresponding standard for nuts is ASTM A194/A194M, *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both* (ASTM, 2012b).]

**ASTM A1082/A1082M** ***Standard Specification for High Strength Precipitation Hardening and Duplex Stainless Steel Bolting for Special Purpose Applications*** **(ASTM, 2012i)**

This standard covers high-strength duplex and precipitation hardening stainless steels for special purpose applications such as pressure vessels. Nuts are to be made from the stainless steels listed in the standard and tested to its requirements. This is the only ASTM standard that covers duplex stainless steel bolts and it has no minimum or maximum size limit. The size limit for precipitation hardening stainless steels varies with the heat treatment condition but is generally 8 in. (200 mm). (ASTM F593 and F738M also cover precipitation hardening stainless steels but limit sizes to 1½ in. or 36 mm respectively and are for general corrosion resistance, not high-strength applications.)

There are no ASTM standards for stainless steel washers so purchasers should require that the washer raw material has a chemical composition and mechanical properties that meet the requirements of ASTM A240/A240M (ASTM, 2012c). The bolts, washers and nuts should all be of equivalent corrosion resistance.

Table 2-4, Table 2-5, Table 2-6, Table 2-7 and Table 2-8 give the minimum specified mechanical properties for the austenitic, precipitation hardening and duplex stainless steel bolts covered in ASTM F593, ASTM F738M, ASTM A320/A320M, ASTM A193/A193M, and ASTM A1082/1082M, respectively.

### 2.3.3 Mechanical Properties Used in Design

*Flat and long products*

It is recommended that the specified minimum yield stress, $F_y$, and the specified minimum tensile strength, $F_u$, be taken as the minimum values specified in the relevant ASTM standard (Section 2.3.1).

It should be noted that the measured yield strength of austenitic stainless steels may exceed the specified minimum values by a margin varying from 25 to 40%, for plate thicknesses of 1 in. (25 mm) or less. The margin for duplex stainless steels is lower, perhaps up to 20%. There is an inverse relationship between thickness or diameter, and yield stress; lighter gauges typically have yield stresses that are significantly higher than the minimum requirement whereas at thicknesses of 1 in. (25 mm) and above, the values are usually fairly close to the ASTM specified minimum yield stress.

For external, exposed structures in very hot climates, due consideration should be taken of the maximum temperature the stainless steel is likely to reach. While smaller and sheltered components may remain at ambient temperatures, large surface areas of bare stainless steel that are exposed to direct sun can reach temperatures that are about 50% higher than ambient temperature. Resources like **www.weatherbase.com** can be used to determine historic weather patterns. If the maximum temperature of the stainless steel is likely to reach 140 °F (60 °C), then a 5% reduction should be made to the room temperature yield strength; greater reductions will be necessary for higher temperatures.

| Table 2-4. Minimum Specified Mechanical Properties of Austenitic and Precipitation Hardening Stainless Steel Bolts to ASTM F593 | | | | | |
|---|---|---|---|---|---|
| | Alloy Mechanical Property Marking | | Nominal Diameter | Tensile Strength | Yield Strength[b] |
| Condition[a] | Group 1 (S30400/S30403) | Group 2 (S31600/S31603) | in. | ksi | ksi |
| AF | F593A | F593E | ¼ to 1½ incl | 65 – 85 | 20 |
| A | F593B | F593F | ¼ to 1½ incl | 75 – 100 | 30 |
| CW1 | F593C | F593G | ¼ to ⅝ incl | 100 – 150 | 65 |
| CW2 | F593D | F593H | ¾ to 1½ incl | 85 – 140 | 45 |
| SH1 | F593A | F593E | ¼ to ⅝ incl | 120 – 160 | 95 |
| SH2 | F593B | F593F | ¾ to 1 incl | 110 – 150 | 75 |
| SH3 | F593C | F593G | 1⅛ to 1¼ incl | 100 – 140 | 60 |
| SH4 | F593D | F593H | 1⅜ to 1½ incl | 95 – 130 | 45 |
| | Group 7 (S17400) | | | | |
| AH | F593U | | ¼ to 1½ incl | 135 – 170 | 105 |

[a] Explanation of conditions:
A—Machined from annealed or solution-annealed stock thus retaining the properties of the original material, or hot-formed and solution-annealed.
AF—Headed and rolled from annealed stock and then re-annealed.
CW—Headed and rolled from annealed stock thus acquiring a degree of cold work; sizes ¾ in. (19 mm) and larger may be hot worked and solution-annealed.
SH—Machined from strain hardened stock or cold worked to develop the specified properties.
AH—Solution annealed and age-hardened after forming.
[b] Yield strength is the stress at which an offset of 0.2% gage length occurs.

| Table 2-5. Minimum Specified Mechanical Properties of Austenitic and Precipitation Hardening Stainless Steel Bolts to ASTM F738M | | | | | |
|---|---|---|---|---|---|
| | | | | Tensile Strength | Yield Strength[c] |
| Condition[a] | Property Class[b] | Alloy Mechanical Property Marking | Nominal Thread Diameter | MPa | MPa |
| AF | A1-50 | F738A | M1.6 – M5 | 500 | — |
| | A4-50 | F738C | M6 – M36 | 500 | 210 |
| CW | A1-70 | F738D | M1.6 – M5 | 700 | — |
| | A4-70 | F738F | M6 – M20 | 700 | 450 |
| | | | Over M20 – M36 | 550 | 300 |
| SH | A1-80 | F738G | M1.6 – M5 | 800 | — |
| | | | M6 – M20 | 800 | 600 |
| | A4-80 | F738J | Over M20 – M24 | 700 | 500 |
| | | | Over M24 – M30 | 650 | 400 |
| | | | Over M30 – M36 | 600 | 300 |
| AH | P1-90 | F738W | M1.6 – M5 | 900 | — |
| | | | M6 – M36 | 900 | 700 |

[a] Explanation of conditions:
AF—Headed and rolled from annealed stock and then re-annealed.
CW—Headed and rolled from annealed stock thus acquiring a degree of cold work; sizes ¾ in. (19 mm) and larger may be hot worked and solution-annealed.
SH—Machined from strain hardened stock or cold worked to develop the specified properties.
AH—Solution annealed and age-hardened after forming
[b] Property Class A1 is equivalent to S30400/S30403, A4 to S31600/S31603, and P1 to S17400.
[c] Yield strength is the stress at which an offset of 0.2% gage length occurs.

| Table 2-6. Minimum Specified Mechanical Properties of Austenitic Stainless Steel Bolts to ASTM A320/A320M | | | | |
|---|---|---|---|---|
| Class | Type | Diameter in. (mm) | Tensile Strength ksi (MPa) | Yield Strength 0.2% offset ksi (MPa) |
| Class 1 | B8 and B8M | All diameters | 75 (515) | 30 (205) |
| Class 1A | B8A and B8MA | All diameters | 75 (515) | 30 (205) |
| Class 2 | B8 | ¾ (20) and under | 125 (860) | 100 (690) |
| | | Over ¾ to 1 (20 to 25) incl | 115 (795) | 80 (550) |
| | | Over 1 to 1¼ (25 to 32) incl | 105 (725) | 65 (450) |
| | | Over 1¼ to 1½ (32 to 40) incl | 100 (690) | 50 (345) |
| Class 2 | B8M | ¾ (20) and under | 110 (760) | 95 (655) |
| | | Over ¾ to 1 (20 to 25) incl | 100 (690) | 80 (550) |
| | | Over 1 to 1¼ (25 to 32) incl | 95 (655) | 65 (450) |
| | | Over 1¼ to 1½ (32 to 40) incl | 90 (620) | 50 (345) |

Notes:
1. Explanation of classes:
   Class 1 products are made from solution-treated material. Class 1A products are solution-treated in the finished condition.
   Class 2 products are solution-treated and strain-hardened.
2. Designations B8 and B8A are equivalent to S30400 and B8M and B8MA to S31600.

| Table 2-7. Minimum Specified Mechanical Properties of Austenitic Stainless Steel Bolts to ASTM A193/A193M | | | | |
|---|---|---|---|---|
| Class | Type | Diameter[a] in. (mm) | Tensile Strength ksi (MPa) | Yield Strength 0.2% Offset ksi (MPa) |
| For Class1, 1A and 2 the data given is the same as that given in ASTM A320 shown in Table 2-6. | | | | |
| Class 2B | B8, B8M2 | 2 and under (M48 and under) | 95 (655) | 75 (515) |
| | | Over 2 to 2½ incl (over M48 to M64 incl) | 90 (620) | 65 (450) |
| | | Over 2½ to 3 incl (over M64 to M72 incl) | 80 (550) | 55 (380) |
| Class 2C | B8M3 | 2 and under (M48 and under) | 85 (585) | 65 (450) |
| | | Over 2 (over M48) | 85 (585) | 60 (415) |

[a] For diameters 1½ in. (M38) and over, center (core) properties may be lower than indicated by test reports, which are based on values determined at ½ in. (13 mm) radius.

Notes:
1. Explanation of classes
   Class 2 products are solution-treated and strain-hardened.
2. Designation B8 is equivalent to Type S30400 and B8M2 and B8M3 to S31600

| Table 2-8. Minimum Specified Mechanical Properties of Some Duplex Stainless Steel Bolts to ASTM A1082/A1082M | | | |
|---|---|---|---|
| UNS Designation | Marking | Tensile Strength ksi (MPa) | Yield Strength 0.2% Offset ksi (MPa) |
| S32101 | 32101 | 94 (650) | 65 (450) |
| S32304 | 32304 | 90 (620) | 65 (450) |
| S32205 | 32205 | 95 (655) | 65 (450) |

Note:
Higher strengths are available in other duplex stainless steels in this specification.

**Table 2-9. Room Temperature Physical Properties, Annealed Condition**

| Type | Initial Modulus of Elasticity | | Density | | Thermal Conductivity at 68 °F (20 °C) | | Specific Thermal Capacity at 68 °F (20 °C) | |
|---|---|---|---|---|---|---|---|---|
| | ksi | MPa | $lb/ft^3$ | $kg/m^3$ | BTU/ (hr-ft-°F) | W/(m-K) | BTU/ (lb-°F) | J/(kg-K) |
| S30400 S30403 | 28,000 | 193,000 | 490 | 7900 | 8.7 | 15 | 0.12 | 500 |
| S31600 S31603 | 28,000 | 193,000 | 500 | 8000 | | | | |
| S32101 S32304 S32205 | 29,000 | 200,000 | 485 | 7800 | | | | |
| S17400 | 28,500 | 197,000 | 485 | 7800 | 9.2 | 16 | | |

Note:
The data are taken from EN 10088, *Stainless Steels—Part 1: List of Stainless Steels* (CEN, 2005d) apart from the values for the initial modulus of elasticity which are taken from *Boiler and Pressure Vessel Code, Section II: Materials—Part D: Properties (Customary)* (ASME, 2010), with the value for the austenitic stainless steels rounded down to 28,000 ksi (193,000 MPa).
Poisson's ratio can be taken as 0.3 and the shear modulus of elasticity, $G$, as $0.385E$.

*Bolts*

For a bolt under tension or shear, or combined tension and shear, the available strength should be based on the specified minimum values given in the relevant ASTM standard.

## 2.4 PHYSICAL PROPERTIES

Table 2-9 gives the room temperature physical properties in the annealed condition of the stainless steels covered in this Design Guide. Physical properties may vary slightly with product form and size but such variations are usually not of critical importance to the application.

The coefficients of thermal expansion are given in Section 10.2.1. Note that the coefficient of thermal expansion for austenitic stainless steels is about 30% higher than that for carbon steel. Where carbon steel and austenitic stainless steel are used together, the effects of differential thermal expansion coefficients should be considered in design.

Both duplex and precipitation hardening stainless steels are magnetic. Where the nonmagnetic properties of the austenitic stainless steels are important to the application, care must be exercised in selecting appropriate filler metals for welding to minimize the ferrite content in the weldment. Heavy cold working, particularly of the lean alloyed austenitic steels, can also increase magnetic permeability; subsequent annealing would restore the nonmagnetic properties. For nonmagnetic applications, it is recommended that further advice be obtained from a steel producer.

Austenitic stainless steels are used for cryogenic applications. At the other end of the temperature scale, austenitic stainless steels retain a higher proportion of their strength above approximately 1,020 °F (550 °C) than carbon steel. However, the design of structures subject to long-term exposure at cryogenic temperatures or to long-term exposure at high temperatures is outside the scope of this Design Guide. Other stainless steels than those selected here are in most cases better suited for high temperature applications and further advice should be sought.

Precautions should be taken to ensure that in the event of a fire, molten zinc from galvanized steel cannot drip or run onto the stainless steel and cause embrittlement.

Duplex stainless steels should not be used for long periods at temperatures above approximately 570 °F (300 °C), due to the possibility of embrittlement.

There is no evidence that suggests through-thickness lamellar tearing occurs in stainless steels.

## 2.5 SELECTION OF MATERIALS

### 2.5.1 Stainless Steel Selection

In the great majority of structural applications utilizing stainless steel, it is the corrosion resistance of the metal that is being exploited, whether this is for aesthetic reasons, minimal maintenance, or long-term durability. Corrosion resistance must therefore be the primary factor in choosing a suitable stainless steel.

Stainless steels derive their corrosion resistance from the presence of a passive surface oxide film which, given adequate access to oxygen or suitable oxidizing agents, tends to be self-healing if damaged or removed by machining or finishing. This oxide film is primarily a consequence of the chromium content of the stainless steel, although small additions of molybdenum and nitrogen also improve resistance to certain corrosion mechanisms, e.g., pitting (see Section 2.6.2).

There are many different stainless steel alloys offering a wide range of corrosion resistance. Corrosion can initiate when environmental conditions are too corrosive for the particular stainless steel specified. This could include exposure to corrosive chemicals, fumes, particulate and chlorides (i.e., chloramines, hydrochloric acid, food additives, coastal and deicing salts, water processing) in applications like industrial plants, building exteriors, swimming pools and infrastructure. Contamination with iron, carbon steel, residual adhesive from protective films, and other substances can also lead to surface corrosion.

Careful design and stainless steel selection should ensure trouble free performance, but designers should be aware that even stainless steels may be subject to various forms of corrosion under certain circumstances. It is possible to employ stainless steels effectively, provided that a few elementary principles are kept in mind.

The selection of an appropriate stainless steel must take into account the service environment, fabrication requirements like bend radii and welding, surface finish, and the maintenance of the structure. Additionally, the designer must determine the criteria for corrosion failure. If the component must remain structurally sound for a defined period of time and appearance is not important, acceptable corrosion rates are considered during selection and a less corrosion-resistant stainless steel may be satisfactory. If however a pristine corrosion-free appearance is important, then a more corrosion-resistant stainless steel, a smoother surface finish, or more frequent cleaning may be required. It should be noted that the maintenance requirement is minimal; merely washing down the stainless steel, even naturally by rain, can maintain or improve the initial appearance and assist in extending the service life.

The first step is to characterize the service environment, including reasonably anticipated deviations from the design conditions. In addition to exposure to corrosive substances, operational, climate and design details that can influence performance must be considered as well as the expected service life. For example, in industrial applications, corrosive chemical combinations and concentrations, exposure times, surface deposit accumulations, acidity, and maintenance cleaning can all influence performance. In exterior applications, exposure to heavy cleaning rain (or degree of sheltering), moisture levels (e.g., humidity, rain heaviness, fog), airborne particulate levels, salt spray (e.g., a rocky coast or roadway), splashing or immersion in chloride (salt) water, and similar factors must be considered. In all applications, design details like unsealed crevices, contact with other metals, and finish specification can influence performance. Possible future developments or change of use should also be considered. It should also be noted that installations can be in close proximity but have very different exposure levels.

Candidate types of stainless steel can then be chosen to give overall satisfactory corrosion resistance in the anticipated environment. The selection of a suitable stainless steel should consider which possible forms of corrosion might occur. Section 2.6 describes these forms of corrosion, illustrates appropriate design, and discusses the circumstances where caution may be necessary and specialist advice should be sought. Consideration should then be given to mechanical properties, ease of fabrication, availability of product forms, surface finish, and costs. Duplex stainless steels have a significantly higher yield strength than conventional structural carbon steel and austenitic stainless steels. Therefore, for members that are not controlled by buckling or deflection, the use of duplex stainless steel leads to considerable weight savings, which offsets to some degree the higher material cost.

Assessing the suitability of a type of stainless steel is best approached by referring to experience with stainless steels in similar applications and environments. Table 2-10 gives very general guidance for selecting suitable stainless steels for external applications where surface corrosion is not desirable. It only considers pollution and chloride (coastal and deicing salts), and not other factors that can influence performance, such as crevices created by fasteners, high surface particulate levels, indoor swimming pool environments, or an environment where both pollution and chlorides are

| Table 2-10. Suggested Stainless Steels for External Non-Immersed Applications That Are Not Sheltered nor Exposed to Heavy Rain on a Regular Basis | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Location | | | | | | | | | | | |
| | Rural | | | Urban | | | Industrial | | | Coastal/ Deicing salt | | |
| Stainless Steel | L | M | H | L | M | H | L | M | H | L | M | H |
| Basic chromium-nickel austenitic steels (e.g., S30400 and S30403) | √ | √ | √ | √ | √ | (√) | (√) | (√) | × | √ | (√) | × |
| Molybdenum-chromium-nickel austenitic steels (e.g., S31600 and S31603) and duplexes S32101 and S32304 | O | O | O | O | √ | √ | √ | √ | (√) | √ | √ | (√) |
| Duplex S32205 | O | O | O | O | O | O | O | O | √ | O | O | √ |

L Least corrosive conditions within that category, e.g., tempered by low humidity, low temperatures.
M Fairly typical of that category.
H Corrosion likely to be higher than typical for that category, e.g., increased by persistent high humidity, high ambient temperatures, and particularly aggressive air pollutants. This does not include applications exposed to chloride salt spray, splashing or occasional immersion.
O Potentially more corrosion resistant than necessary but other factors such as strength or aesthetic requirements may lead to specification.
√ Probably the best choice for corrosion resistance and cost in most environments, but particularly severe locations or aesthetic application requirements may make the use of more highly alloyed stainless steel necessary.
× Likely to suffer excessive corrosion.
(√) Should only be considered if a very smooth surface is specified and if regular manual washing is carried out.

Notes:
1. These guidelines assume unsheltered exposure, exposure to regular heavy rain cleaning, no significant accumulation of particulate, and do not consider the crevice corrosion in chloride containing environments. They may not be conservative enough for aesthetic applications.
2. Precipitation hardening stainless steel Type S17400 is comparable to or slightly less corrosion resistant than S30400/S30403.

present. In the case of immersed stainless steel, see Section 2.6.3. When stainless steel comes into contact with chemicals or is used in higher chloride exposure environments, expert advice should be sought. For exterior applications that are expected to be more corrosive or that combine pollution and chloride exposure, refer to the IMOA Stainless Steel Selection System (**www.imoa.info**) which provides more detailed guidance for nonmetallurgist decision makers. Information about the corrosiveness of industrial chemicals can be obtained from stainless steel producers and industry associations.

Austenitic stainless steel fastener, bar, tubular products and plate produced for use in high-speed welding applications generally have sulfur levels above 0.005%. These higher levels of sulfur increase the likelihood of corrosion, especially in industrial and chloride (e.g., salt) environments, unless surface sulfides are removed by chemical passivation in accordance with ASTM A967, *Standard Specification for Chemical Passivation Treatments for Stainless Steel Parts* (ASTM, 2005).

### 2.5.2 Availability of Product Forms

*General Types of Product Forms*

Sheet, plate and bar products are all widely available in the stainless steels included in this Design Guide. Tubular products are available in austenitic types and Duplexes S32205 and S32101, though S32304 and proprietary duplexes are not readily stocked in tubular form in the U.S. presently. Mill deliveries of S32304 are still possible from European stocks.

A range of structural sections (I-shaped members, angles, channels, tees, rectangular hollow sections) are stocked in standard austenitic types such as S30400/S30403 and S31600/S31603 but the duplex stainless steels usually require special orders. Generally, sections may be produced by cold forming, hot rolling, extrusion, and arc or laser welding.

Regarding precipitation hardening stainless steels, fasteners are generally available from distributors whereas special runs may be required for tension bars; early discussion with the service center or steel producer is recommended.

*Cold Forming*

It is important that early discussion with potential fabricators takes place to ascertain cold-forming limits for heavier gage hot rolled stainless steel plate. Stainless steels require higher forming loads than carbon steels and have different springback properties. The length of brake pressed cold-formed sections is necessarily limited by the size of the machine or by power capability in the case of thicker or stronger materials. Duplexes require approximately twice the forming loads used for the austenitic materials and consequently the possible range of duplex sections is more limited. If down gaging is possible, the difference in forming load will also be lower. Furthermore, because of the lower ductility in the duplex material, more generous bending radii should be used. Lighter walled hollow sections are often produced by roll forming and welding. Hot rolled austenitic plate up to about 0.5 in. (13 mm) can be cold rolled to form structural sections, such as angles.

Precipitation hardening stainless steels can be cold formed by rolling, bending and hydroforming but only in the fully annealed condition. After cold working, stress corrosion resistance is improved by aging at the precipitation hardening temperature.

*Hot Rolling*

Stainless steel plates too thick for cold forming are heated and rolled into their final shape. This method is generally most cost effective for larger production runs. A wide range of plate thicknesses and widths are used to produce medium to large structural components. Angles and channels are commonly produced using this technique. This technique may be combined with welding to create structural sections. For example, welding two channels together produces I-shaped members. Heavier walled hollow structural sections are often produced by hot rolling and welding.

*Extrusion*

Hot finished stainless steel extrusions are produced from bar. If the shape required is not common, a larger production run may be necessary to justify the die cost. The maximum size varies with the producer but must fit within a 13 in. (330 mm) circle. Sections are generally provided in lengths of up to 34 ft (10 m). In addition to standard structural shapes, extrusion is capable of producing a wide range of custom shapes that might otherwise require machining or a custom welded fabrication. Suppliers should be contacted regarding minimum section thicknesses and corner radii.

*Welded Plate*

Welded plate fabrications are typically used when small quantities of a custom shape are required, sharper bends or nontapered legs are preferred, or the component is quite large. When a project requires small quantities of very large or unusually shaped structural components, experienced stainless steel fabricators often fabricate them by welding together plate using the standard approved methods in AWS D1.6/D1.6M, *Structural Welding Code—Stainless Steel* (AWS, 2010), hereafter referred to as AWS D1.6/D1.6M.

Laser welded or fused angles, beams, channels, tees and hollow sections are increasingly being stocked by service centers in common sizes. Angles, beams and channels of up to 15 in. (380 mm) may be found in austenitic stainless steels. Larger sections and duplex stainless steel sections can also be produced.

*Surface Finish*

In certain applications, surface finish roughness may be important for corrosion performance or surface cleanability. Appearance uniformity and finish options are important for aesthetic applications. Manufacturers offer a range of standard finishes, from dull mill finish through bright polishes. ASTM A480/A480M (ASTM, 2012f) describes plate, sheet and strip finishes. Cold-formed sections can be obtained in the broadest range of finishes for a variety of aesthetic effects.

The finish options for hot-formed, extruded and welded plate sections are more limited. An abrasive blasted and pickled[†] finish is the most commonly available option. Sandblasting is common but other abrasive blasting materials can be requested. Quartz and stainless steel shot can produce a more reflective surface than sandblasting. When mechanically polished ASTM A480 No. 3 or 4 finishes are required on heavier sections, the supplier and specialty polishers should be contacted to determine availability. Welded plate fabrications are generally the easiest to obtain with a smooth polished finish.

It should be noted that although the various finishes are standardized, variability in processing introduces differences in appearance between manufacturers and even from a single producer so suppliers must be made aware of finish matching requirements. Bright finishes make any surface unevenness more apparent. Duller finishes always look flatter. There is inherently a minor variation in the natural silver color of different stainless steel families (austenitic, duplex, ferritic) which should be considered during design.

More guidance on surface finishes is given in *Stainless Steels in Architecture, Building and Construction—Guidelines for Corrosion Prevention* (Houska, 2001), and an interactive guide to the standard mill and polished finishes for different stainless steel product forms can be found on the Specialty Steel Industry of North America (SSINA) website **www.ssina.com**.

*Bolts*

ASTM F593 bolts are the most widely available. Certain size and length restrictions apply and reference should be made to the relevant ASTM standards. It is possible to have special bolts made-to-order and indeed, this sometimes produces an economical solution.

Bolts can be produced by a number of techniques, e.g., machining, cold forming and forging. Machined threads should not be used in very aggressive environments (e.g.,

† Pickling is the removal of a thin layer of metal from the surface of the stainless steel, usually by applying a mixture of nitric and hydrofluoric acid. Alternative, less aggressive compounds are also available from specialized suppliers.

marine), due to potential problems with crevice corrosion. Rolled threads are also preferred because they are generally stronger than machined threads and provide greater resistance to thread galling. The bolts and other fastener materials should always be specified from stainless steels that are at least as corrosion resistant as the sections that they are joining to avoid premature failure. See Section 2.3.2 for more information on fastener specification.

### 2.5.3 Life Cycle Costing and Environmental Impact

There is increasing awareness that life cycle (or whole life) costs, not just initial costs, should be considered when selecting materials. Life cycle cost analyses consider:

- Life cycle inventory
- Initial costs
- Maintenance costs
- Diversion from landfills and recycled content
- Service life and environment

The initial raw material cost of a structural stainless steel product is considerably higher than that of an equivalent carbon steel product. However, there can be initial cost savings associated with eliminating corrosion-resistant coatings. Utilizing high-strength stainless steels may reduce material requirements by decreasing section size and overall structure weight which cuts initial costs. Additionally, eliminating the need for coating maintenance or component replacement due to corrosion can lead to significant long-term maintenance cost savings.

Stainless steel has a high residual scrap value. For this reason, pre-consumer scrap is diverted from landfills and recycled into new metal and end-of-life (EOL) recycling rates are very high. An international study of the life cycle of stainless steel, including the typical service life and EOL recapture rates by application, concluded that in the industrial equipment and the building and infrastructure segments, 92% of stainless steel is captured at the EOL for use in new stainless or carbon steel (Reck et al., 2010; ISSF and Société de Calcul Mathématique SA, 2010). The EOL recycling rates for stainless steel increased by 6% between 2000 and 2005.

Stainless steel producers use as much scrap as is available, but the material's overall average 20 to 30 year service life limits scrap availability. In 2002, the International Stainless Steel Forum (ISSF) estimated that, internationally, the typical recycled content for all types of stainless steel was about 60%. In North America, SSINA has a downloadable USGBC LEED statement indicating that the typical recycled content of the austenitic stainless steels is between 75% and 85%. Currently, in parts of the world where scrap is readily available, some producers are reporting scrap recycled content levels of up to 90%. Stainless steel is 100% recyclable and can be indefinitely recycled into new high quality stainless steel.

## 2.6 DURABILITY

### 2.6.1 Introduction

Stainless steels are generally very corrosion resistant and perform satisfactorily in most environments. The limit of corrosion resistance for a given stainless steel is predominantly dependent on its alloying elements, which means that each type has a slightly different response when exposed to a corrosive environment. Care is therefore needed to select the most appropriate stainless steel for a given application. Generally, higher levels of corrosion resistance increase the cost of the material. For example, Type S31600 stainless steel costs more than Type S30400 because of the addition of molybdenum. Duplex stainless steels can potentially offer increased corrosion resistance with less of a price premium. Furthermore, their higher strength may make it possible to reduce section sizes and, therefore, material cost; thus, potentially reducing the overall project cost. Most austenitic material in the cold-worked condition has a similar corrosion resistance to that in the annealed condition.

The most common reasons for a metal to fail to live up to corrosion performance expectations are:

(1) Incorrect assessment of the environment or exposure to unexpected conditions, e.g., unsuspected contamination by chloride ions or higher than expected surface accumulations.

(2) Inappropriate stainless steel fabrication techniques (e.g., welding, heat treating, and heating during forming), incomplete weld heat tint removal, or surface contamination may increase susceptibility to corrosion.

(3) Too rough or incorrectly orientated finish.

Even when surface staining or corrosion occur, it is unlikely that structural integrity will be compromised (see Section 2.6.2). In aggressive industrial and marine environments, tests have shown no indication of reduction in component capacity even where a small amount of weight loss occurred. The specific environmental conditions and loads have to be considered and the advice of a stainless steel corrosion specialist should be obtained for more corrosive environments. Typical corrosion rate data may be available for predicting service life. However, the user may still regard unsightly rust staining on external surfaces as an aesthetic failure. In addition to careful material selection, good detailing and workmanship can significantly reduce the likelihood of staining and corrosion; practical guidance is given in Chapter 12. Experience indicates that any serious corrosion problem is most likely to show up in the first two or three years of service. In certain aggressive environments, some stainless steels are susceptible to localized attack. Six mechanisms are described in Section 2.6.2, although the last three are very rarely encountered in buildings. It should be emphasized that the presence of moisture (including humidity and condensation) is necessary for corrosion to occur.

### 2.6.2 Types of Corrosion and Performance of Steel Types

*Pitting Corrosion*

As the name implies, pitting takes the form of localized pits. It occurs as a result of local breakdown of the passive layer, normally by chloride ions, although the other halides and other anions can have a similar effect. In a developing pit, corrosion products may create a very corrosive solution often leading to high propagation rates. In most structural applications, the extent of pitting is likely to be superficial and the reduction in section of a component is negligible. However, corrosion products can stain architectural features. A less tolerant view of pitting should be adopted for services such as ducts, piping and containment structures.

Since the chloride ion is a common cause of pitting in exterior applications, coastal areas and environments laden with deicing salts (e.g., sodium chloride, calcium chloride, magnesium chloride) are rather aggressive. In addition to chloride content, the probability of a service environment causing pitting depends on factors such as the temperature, corrosive pollutants and particulate, acidity or alkalinity, the content of oxidizing agents, and also the presence or absence of oxygen. The pitting resistance of a stainless steel is dependent on its chemical composition. Chromium, molybdenum and nitrogen all enhance the resistance of stainless steel to pitting.

The pitting resistance equivalent index (PRE) gives an approximate measure of pitting resistance and is defined as:

$$PRE = \%\ \text{wt Cr} + 3.3(\%\ \text{wt Mo}) + 30(\%\ \text{wt N}) \quad \text{for austenitics}$$

$$PRE = \%\ \text{wt Cr} + 3.3(\%\ \text{wt Mo}) + 16(\%\ \text{wt N}) \quad \text{for duplexes}$$

The PRE of a stainless steel is a useful guide to its corrosion resistance relative to other stainless steels, but should only be used as a first rough indicator. Small differences in PRE can easily be overshadowed by other factors that also influence corrosion pitting resistance. Therefore the PRE should not be the only factor in selection.

Type S30400 has the lowest PRE of the stainless steels covered in this Design Guide and exhibits surface corrosion in applications with low to moderate coastal or deicing salt exposure and is unsuitable for environments with spray/mist, splashing and immersion. Type S30400 may also show unacceptable levels of pitting in industrial atmospheres. For low to moderate exposure to industrial pollution, or coastal or deicing chloride salts, S31600 or Duplex S32304 (or a lean duplex such as LDX2101®) is preferred. When pollution or salt exposure is higher or there is direct exposure to industrial chemicals, Duplex S32205 or even more corrosion-resistant stainless steels are generally an option.

*Crevice Corrosion*

Crevice corrosion can only occur when there are tight unsealed crevices that allow infiltration of moisture and corrosive substances. It can be avoided by sealing crevices (e.g., welding, flexible inert washers, or sealant) or eliminating them. The severity of a crevice is very dependent on its geometry—the narrower and deeper the crevice, the more severe the corrosion conditions. The appropriate design will be dependent on the conditions.

Joints that are not submerged should be designed to shed moisture. Some stainless steels, including Types S30400 and S31600, are susceptible to crevice corrosion when chlorides or salts are present in the environment. More corrosion-resistant austenitics and the duplexes are less susceptible and performance will be dependent on the conditions, especially the temperature.

The severity of corrosion in submerged crevices is generally worse than in corrosive above-water atmospheric environments that have wetting and drying cycles, or are regularly slightly moist. Submerged tight crevices are more aggressive because the diffusion of oxidants necessary for maintaining the passive film is restricted.

Crevices may result from a metal-to-metal joint, a gasket, biofouling, surface deposits (e.g., particulate, leaves, food, guano, debris), and surface damage such as embedded iron. Careful attention to detailing can help to eliminate crevices, but it is often not possible.

As in pitting corrosion, the alloying elements chromium, molybdenum and nitrogen enhance the resistance to attack and thus the resistance to crevice corrosion increases from Type S17400 and S30400, through S31600 to S32205.

*Galvanic (Bi-Metal, Dissimilar Metal) Corrosion*

When two dissimilar metals are in direct electrical contact and are also bridged by an electrolyte (i.e., an electrically conducting liquid such as seawater or impure fresh water), a current flows from the anodic metal to the cathodic or nobler metal through the electrolyte. As a result the less noble metal corrodes.

Stainless steels usually form the cathode in a galvanic couple and therefore do not suffer additional corrosion. Stainless steels and copper alloys are very close in the galvanic series, and when exposed to moderate atmospheric conditions can generally be placed in direct contact without concern.

This form of corrosion is particularly relevant when considering joining stainless steel to carbon or low alloy steels, weathering steel, or aluminum. If the metals will be joined by welding, it is important to select filler metal that is at least as noble as the most corrosion-resistant material and to apply a protective corrosion-resistant coating. Likewise, if connected with fasteners, the bolting material should be equivalent to the most corrosion-resistant metal. Galvanic corrosion between different types of stainless steel is hardly ever a concern, and then, only under fully immersed conditions.

Galvanic corrosion can be prevented by eliminating current flow by:

- Insulating dissimilar metals, i.e., breaking the metallic path (see Section 9.1).
- Preventing electrolyte bridging, i.e., breaking the electrolytic path by paint or other coating. Where protection is sought by this means and it is impractical to coat both metals, then it is preferable to coat the more noble one (i.e., stainless steel in the case of a stainless/carbon steel connection).

The risk of a deep corrosion attack is greatest if the area of the more noble metal (i.e., stainless steel) is large compared with the area of the less noble metal (i.e., carbon steel). Special attention should be paid to the use of paints or other coatings on the carbon steel. If there are any small pores or pinholes in the coating, the small area of bare carbon steel provides a very large cathode/anode area ratio, and severe pitting of the carbon steel may occur. This is, of course, likely to be most severe under immersed conditions. In these situations, it is preferable to paint the stainless steel also up to a distance of about 2 in. (50 mm) away from where the metals are in contact so that any pores lead to small area ratios.

Adverse surface area ratios are likely to occur with fasteners and at joints. Carbon steel bolts in stainless steel members should be avoided because the ratio of the area of the stainless steel to the carbon steel is large and the bolts will be subject to aggressive attack. Conversely, the rate of attack on a carbon steel or aluminum member by a stainless steel bolt is negligible. It is usually helpful to draw on previous experience in similar sites because dissimilar metals can often be safely coupled under conditions of occasional condensation or dampness with no adverse effects, especially when the conductivity of the electrolyte is low.

The prediction of these effects is difficult because the corrosion rate is determined by a number of complex issues. The use of electrical potential tables ignores the presence of surface oxide films, the effects of surface area ratios, and different solution (electrolyte) chemical compositions. Therefore, uninformed use of these tables may produce erroneous results. They should be used with care and only for initial assessment.

The general behavior of metals in galvanic contact is discussed in ASTM G82 (ASTM, 2009b). When corrosive conditions are created by an electrolyte other than seawater immersion, such as industrial applications, ASTM G71 (ASTM, 2009a) should be referenced and testing conducted before material specification.

*Stress Corrosion Cracking*

The development of stress corrosion cracking (SCC) requires the simultaneous presence of tensile stresses and specific environmental factors unlikely to be encountered in normal building atmospheres. The stresses do not need to be very high in relation to the yield stress of the material and may be due to loading or to residual effects from manufacturing processes (such as welding or bending). Duplex stainless steels usually have superior resistance to stress corrosion cracking than the austenitic stainless steels covered in this Design Guide. Higher alloy austenitic stainless steels such as N08904 (904L), N08926 (25-6Mo) and S31254 (254 SMO), which are not covered in this Design Guide, have been developed for applications where SCC is a corrosion hazard. Cracking may also occur in high-strength stainless steels such as precipitation hardening steels. This type of cracking is almost always due to hydrogen embrittlement and can occur in both environments containing sulfides and environments free of sulfides.

Caution should be exercised when stainless steel members containing high residual stresses (e.g., due to cold working) are used in chloride-rich environments (e.g., indoor swimming pools, marine, offshore). Highly loaded cables in chloride-rich environments may be susceptible to SCC, depending on the stainless steel. The general advice for load-bearing members in atmospheres containing chlorides that cannot be cleaned regularly (e.g., in suspended ceilings above swimming pools), is to use Types N08926, S31254, and N08367 (AL-6XN), unless the concentration of chloride ions in the pool water is less than or equal to 250 ppm, in which case Duplex S32205 or N08904 are also suitable. Alternative stainless steels that have been shown to have equivalent resistance to stress corrosion cracking in these atmospheres may also be used.

*General (Uniform) Corrosion*

Under normal conditions typically encountered in structural applications, stainless steels do not suffer from the general loss of section that is characteristic of rusting in nonalloyed irons and steels.

*Intergranular Corrosion (Sensitization) and Weld Decay*

When austenitic stainless steels are subject to prolonged heating in the range of 842 to 1,560 °F (450 to 850 °C), the carbon in the steel diffuses to the grain boundaries and precipitates chromium carbide. This removes chromium from the solid solution and leaves a lower chromium content adjacent to the grain boundaries. Steel in this condition is termed sensitized. The grain boundaries become prone to preferential attack when subsequent exposure to a corrosive environment occurs. This phenomenon is known as weld decay when it occurs in the heat-affected zone of a weldment.

There are three ways to avoid intergranular corrosion:

- Use steel having a low carbon content.
- Use steel stabilized with titanium or columbium (niobium) (e.g., Types S32100 or S34700), because these elements combine preferentially with carbon to form stable particles, thereby reducing the risk of forming chromium carbide.
- Use heat treatment; however, this method is rarely used in practice.

A low carbon content (0.03% maximum) stainless steel should be specified when welding sections to avoid sensitization and intergranular corrosion.

### 2.6.3 Corrosion in Selected Environments

The organizations listed at the end of this Design Guide under Sources of Additional Information offer technical articles and publications that discuss the performance of stainless steels in different environments and the reasons for their use.

*Air*

Atmospheric environments vary, as do their effect on stainless steels. Rural atmospheres, uncontaminated by industrial fumes or coastal or deicing salts, are very mild in terms of corrosivity, even in areas of high humidity. Industrial, deicing salt and coastal atmospheres are considerably more severe. Table 2-10 provides very general guidance on selecting suitable types of stainless steel. Additional guidance can be obtained by evaluating the site using the IMOA Stainless Steel Selection System (**www.imoa.info** and Houska, 2009).

The most common causes of atmospheric corrosion are surface contamination with metallic iron particles, from fabrication or at the site, and chloride salts originating from the sea or deicing, industrial pollution, and chemicals (e.g., bleach and hydrochloric acid). Some deposited particles (e.g., dust and sand), vegetation and debris, although inert, create crevices and are able to absorb salts, chemicals, and weak acid solutions from acid rain. Since they also retain moisture for longer periods of time, the result can be a more corrosive local environment.

The surface finish has a significant effect on the general appearance of exposed stainless steel (e.g., dirt accumulation), the effectiveness of rain cleaning, and corrosion rates (smoother finishes have better corrosion resistance).

*Seawater*

Seawater, including brackish water, contains high levels of chlorides and, hence, is very corrosive, particularly when the water flow rate is low [under about 5.0 ft/s (1.5 m/s)]. Severe pitting of S30400 and S31600 stainless steels can occur. Also, these types can suffer attack at crevices, whether these result from design details or from fouling organisms such as barnacles. In some applications, where corrosion can be tolerated, where the expected service life is defined and components will be inspected, Duplex S32205 may be suitable. For longer term installations, "seawater" stainless steels such as 6% or higher molybdenum austenitics, super ferritics, or super duplexes should be specified.

Regular salt spray or splashing may cause as much attack as complete immersion because the surface chloride concentration is raised by the evaporation of water. It should be noted that high chloride concentration run-off water from deicing salt can cause similar corrosion problems in storm drain components.

The possibility of severe galvanic corrosion must be considered if stainless steel is used with other metals in the presence of seawater.

*Other Waters*

Standard austenitic and duplex stainless steels usually perform satisfactorily in distilled, tap and boiler waters. Where acidity is high, Type S31600 should be specified, otherwise Type S30400 usually suffices. Type S31600 is also suggested as being more suitable where there are minor amounts of chloride present to avoid possible pitting and crevice corrosion problems.

Untreated river or lake water, and water used in industrial processing, can sometimes be very corrosive. A full water chemical composition analysis should be obtained including pH level, solids content and type, and chloride level. The typical temperature range, type of biological or microbiological activity, and the concentration and nature of corrosive chemicals are also relevant.

The possibility of erosion corrosion should be considered for waters containing abrasive particles.

*Chemical Environments*

As stainless steel is resistant to many chemicals, it is often used for their containment. The range of applications for stainless steels in chemical environments is broad and specification requires an understanding of the chemical composition, pH, operating temperature range, maintenance, and potential process variability. Because the topic is complex, it is not appropriate to cover this subject in detail here. It should be noted, however, that in many applications stainless steels other than those considered in this Design Guide may be more suitable.

| Table 2-11. Stainless Steels for Use in Different Soil Conditions | | | |
|---|---|---|---|
| **Typical Location** | **Soil Condition** | | **Stainless Steel** |
| Inland | Cl | < 500 ppm | S30400 and S30403<br>S31600 and S31603 |
| | Resistivity | > 400 ohm-in. (1000 ohm-cm) | |
| | pH | > 4.5 | |
| Chorides<br>(coastal/deicing salt)<br>non-tidal zone | Cl | < 1500 ppm | S31600 and S31603 |
| | Resistivity | > 400 ohm-in. (1000 ohm-cm) | |
| | pH | > 4.5 | |
| Chlorides<br>(coastal/deicing salt)<br>tidal zone | Cl | < 6000 ppm | S32750 (2507)<br>S31254, N08926, N08367<br>(6% Mo austenitics) |
| | Resistivity | > 200 ohm-in. (500 ohm-cm) | |
| | pH | > 4.5 | |
| Note:<br>S32750 is a super duplex stainless steel and S31254, N08926 and N08367 are super-austenitic stainless steels with 6% or more molybdenum.<br>These stainless steels are not generally used in construction applications and fall outside the scope of this Design Guide. | | | |

Charts and literature published by manufacturers and industry associations showing results of corrosion tests in various chemicals are available but require careful interpretation. Users can search for specific chemical names or formulae on the online interactive *Outokumpu Corrosion Handbook* (**www.outokumpu.com**) which gives data for different chemical concentrations, operating temperatures, and for some common chemical combinations. Although this provides a guide to the resistance of a particular type of stainless steel, service conditions (temperatures, pressures, concentrations, etc.) vary and generally differ from the laboratory test conditions. Also, the effect of impurities and the degree of aeration can have a marked effect on results. If the conditions appear to be potentially corrosive, further stainless steel specification advice should be sought from a corrosion specialist and testing in the operating environment considered.

*Soils*

Soils differ in their corrosiveness depending on moisture level, pH, aeration, presence of chemical contamination, microbiological activity, and surface drainage. Stainless steels generally perform well in a variety of soils and especially well in soils with high resistivity, although some pitting has occurred in low resistivity, moist soils. The presence of aggressive chemical species, such as chloride ions, as well as types of bacteria and stray current (caused by local direct current electric transportation systems such as railways or tram systems) can cause localized corrosion. The development of stray current can be suppressed with a proper electrical shielding of the pipe (coatings or wrappings) and/or cathodic protection.

For selection of a particular stainless steel, it is recommended to first consider the corrosion resistance of buried stainless steel in relation to the presence of chloride ions, and second, according to the soil resistivity and pH, assuming poorly drained soils in all cases. Chlorides are present near bodies of saltwater (e.g., oceans, bays, lakes), roadways where deicing salts are used, and in some inland locations that were previously sub-sea. The inland locations with higher soil chloride levels are also generally characterized by high chloride concentrations in local water like the southwestern United States. Chlorides may also be present where industrial contamination has occurred.

Research has been conducted around the world to compare the performance of metals buried in a wide range of soil environments. Table 2-11 is extracted from "Corrosion Resistance of Stainless Steels in Soils and in Concrete" (Cunat, 2001). The research summarized in this paper is based on comprehensive studies by the National Institute of Standards and Technology (NIST) (formerly the U.S. National Bureau of Standards), the Japanese Stainless Steel Association (JSSA), and European researchers. If more detailed information is required, "The Underground Corrosion of Selected Type 300 Stainless Steels after 34 Years" (Adler Flitton et al., 2009) summarizes the performance of Types S30400 and S31600 buried near the New Jersey coast. *A Report on the Performance of Stainless Steel Pipe for Water Supply in Underground Soil Environments* (Kyōkai, 1988) compares the performance of stainless steel with copper and iron at 26 sites.

### 2.6.4 Design for Corrosion Control

The most important step in preventing corrosion problems is selecting an appropriately resistant stainless steel with suitable fabrication characteristics for the given environment. However, after specifying a particular steel, much can be achieved in realizing the full potential of the resistance of the

| Table 2-12. Design and Specification for Corrosion Control |
|---|
| Avoid dirt, moisture and corrosive deposit entrapment |
| • Orient angle and channel profiles to minimize the likelihood of deposit or moisture retention |
| • Provide drainage holes, ensuring they are of sufficient size to prevent blockage |
| • Avoid horizontal surfaces |
| • Specify a small slope on gusset stiffeners which nominally lie in a horizontal plane |
| • Use tubular and bar sections (seal tubes with dry gas or air where there is a risk of harmful condensates forming) |
| • Specify smooth finishes or, if rougher finishes are unavoidable, orient the grain vertically if possible |
| Avoid or seal crevices |
| • Use welded rather than bolted connections when possible |
| • Use closing welds or mastic fillers |
| • Preferably dress/profile welds to smooth the surface |
| • Prevent biofouling |
| • Use flexible inert washers or high quality sealants for above ground, nonimmersed bolted connections |
| Reduce the likelihood of stress corrosion cracking in those specific environments where it may occur (see Section 2.6.2): |
| • Minimize fabrication stresses by careful choice of welding sequence |
| • Shot peen (but avoid the use of iron/carbon steel shot to avoid surface embedment of carbon steel particles) |
| Reduce likelihood of pitting (see Chapter 12): |
| • Remove weld spatter |
| • Pickle stainless steel to remove heat tint. Strongly oxidizing chloride-containing reagents such as ferric chloride should be avoided; instead, a pickling bath or a pickling paste, both containing a mixture of nitric acid and hydrofluoric acid, should be used. Welds should always be cleaned up to restore corrosion resistance. Other means such as mechanical cleaning with abrasives or glass beads blasting, or local electrolysis may also be used to clean heat tint and welds. |
| • Avoid pick-up of carbon steel particles (e.g., use workshop area and tools dedicated to stainless steel) |
| • Follow a suitable maintenance program |
| Reduce likelihood of galvanic corrosion (see Section 2.6.2): |
| • Provide electrical insulation between bolted metals with inert materials such as neoprene |
| • Use paints appropriately |
| • Minimize periods of wetness |
| • Use metals that are close to each other in electrical potential |

steel by careful attention to detailing. Anti-corrosion actions should ideally be considered at the planning stage and during detailed design.

Table 2-12 gives a checklist for consideration. Not all points would give the best detail from a structural strength point of view and neither are the points intended to be applied to all environments. In particular, in environments of low corrosivity or where regular maintenance is carried out, many would not be required. Figure 2-3 illustrates poor and good design features for durability.

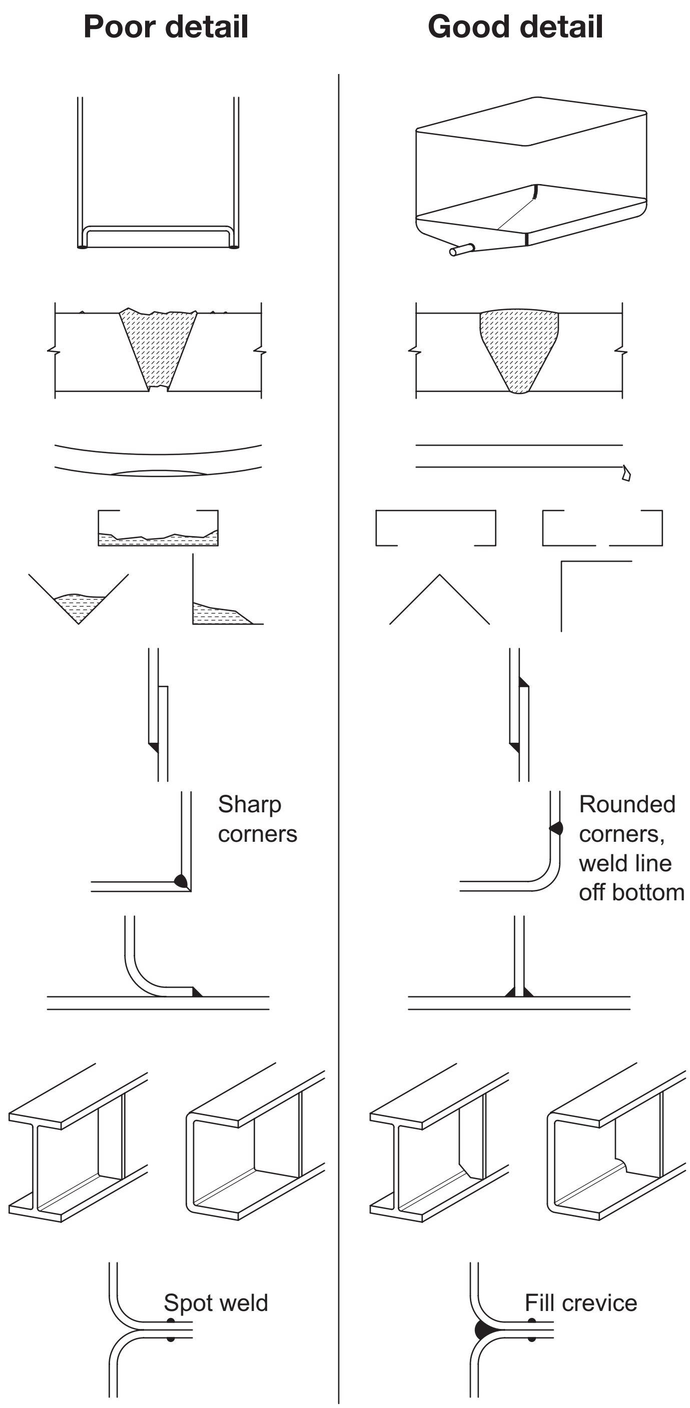



*Fig. 2-3. Poor and good design features for durability.*

# Chapter 3
# Design Requirements

## 3.1 LOADS AND LOAD COMBINATIONS

This Design Guide should be used in conjunction with the loads and load combinations given in ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010).

## 3.2 DESIGN BASIS

Designs should be carried out in accordance with the provisions for load and resistance factor design (LRFD) or the provisions for allowable strength design (ASD).

### 3.2.1 Required Strength

The required strength of structural members and connections should be determined by structural analysis for the appropriate load combinations.

### 3.2.2 Limit States

Design should be based on the principle that no applicable strength or serviceability limit state should be exceeded when the structure is subjected to all appropriate load combinations. The design recommendations are given in terms of load and resistance factor design (LRFD) and allowable strength design (ASD).

*Design for Strength Using Load and Resistance Factor Design (LRFD)*

Design according to the provisions for load and resistance factor design (LRFD) satisfies the requirements of this Design Guide when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations, i.e.,

$$R_u \le \phi R_n \qquad (\textit{Spec.}\ \text{Eq. B3-1})$$

where
$R_u$ = required strength using LRFD load combinations
$R_n$ = nominal strength, specified in Chapters 4 to 9
$\phi$ = resistance factor, specified in Chapters 4 to 9
$\phi R_n$ = design strength

*Design for Strength Using Allowable Strength Design (ASD)*

Design according to the provisions for allowable strength design (ASD) satisfies the requirements of this Design Guide when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations, i.e.,

$$R_a \le R_n/\Omega \qquad (\textit{Spec.}\ \text{Eq. B3-2})$$

where
$R_a$ = required strength using ASD load combinations
$R_n$ = nominal strength, specified in Chapters 4 to 9
$\Omega$ = safety factor, specified in Chapters 4 to 9
$R_n/\Omega$ = allowable strength

### 3.2.3 Design for Stability

*Design of Frames*

This Design Guide concentrates on the design of members and elements. Reference should be made to carbon steel design rules on the elastic analysis of frames, as detailed in Chapter C of the AISC *Specification*, where in Section C3, available strengths should be calculated in accordance with Sections 4 through 9 of this Guide. In particular, the designer needs to consider second-order effects in stainless steel sway frames. These could potentially be greater than in carbon steel frames if the steel is stressed into the nonlinear portion of the stress-strain curve. A method for calculating the secant modulus of elasticity of stainless steel is given in Section 6.7.

Plastic analysis of frames is not applicable to stainless steel due to a lack of research in this area.

Appendix 7 and Appendix 8 of the AISC *Specification* do not apply to stainless steel.

*Buckling of Members*

Although the buckling behavior of stainless steel columns and unrestrained beams is broadly similar to that of carbon steel, the impact of the nonlinear stress-strain curve on the strength and stiffness of a stainless steel member depends on the stress level in the member. For columns this can be explained in terms of the three distinct regions of slenderness:

(a) At high slenderness, 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.

(b) At low slenderness, i.e., when columns attain or exceed the squash load (area × proof strength), the benefits of strain hardening become apparent. For very low slenderness, stainless steels of similar yield strengths to carbon steels give superior column strengths to carbon steels.

(c) At intermediate slenderness, i.e., when the average stress in the column lies between the limit of proportionality and the 0.2% offset yield strength, stainless steel is 'softer' than carbon steel. This leads to reduced column strengths compared to similar carbon steel columns.

ASCE/SEI 8 takes the nonlinear stress-strain curve of stainless steel into account by replacing the initial elastic modulus of elasticity with the tangent modulus corresponding to the buckling stress, which involves an iterative design procedure. However, the buckling curves in this Design Guide were derived by calibration against experimental data, modifying the coefficients in the AISC buckling curves for carbon steel to align with experimental stainless steel data because it was considered preferable to have an explicit design solution as opposed to one requiring an iterative solution.

### 3.2.4 Design for Serviceability and Ponding

The guidance for serviceability and ponding of carbon steel structures provided in AISC *Specification* Chapter L and Appendix 2 is generally applicable to stainless steel structures. However, see Section 6.7 for guidance on the determination of deflections.

## 3.3 MEMBER PROPERTIES

### 3.3.1 Classification of Sections for Local Buckling

For compression, sections are classified as nonslender element or slender-element sections. For a nonslender element section, the width-to-thickness ratios of its compression elements should not exceed $\lambda_r$ from Table 3-1. If the width-to-thickness ratio of any compression element exceeds $\lambda_r$, the section is a slender-element section.

For flexure, sections are classified as compact, noncompact or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-to-thickness ratios of its compression elements should not exceed the limiting width-to-thickness ratio, $\lambda_p$, from Table 3-2. If the width-to-thickness ratio of one or more compression elements exceeds $\lambda_p$ but does not exceed $\lambda_r$ from Table 3-2, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds $\lambda_r$, the section is a slender-element section.

The coefficients in Table 3-1 and Table 3-2 were derived by calibration against experimental data and differ from those given for carbon steel in the AISC *Specification* due to the nonlinear stress-strain characteristics of stainless steel.

*Unstiffened Elements*

For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width should be taken as follows:

(a) For flanges of I-shaped members and tees, the width $b$ is one-half the full-flange width, $b_f$.

(b) For legs of angles and flanges of channels, the width $b$ is the full nominal dimension.

(c) For plates, the width $b$ is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, $d$ is taken as the full nominal depth of the section.

*Stiffened Elements*

For stiffened elements supported along two edges parallel to the direction of the compression force, the width should be taken as follows:

(a) For webs of rolled or formed sections, $h$ is the clear distance between flanges less the fillet or corner radius at each flange.

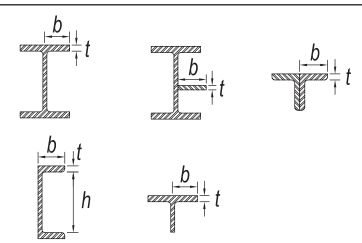
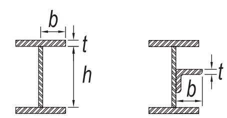
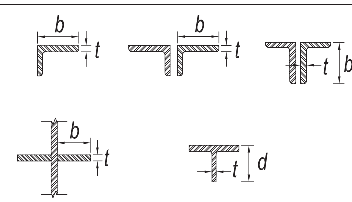
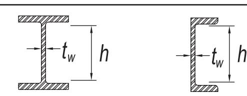
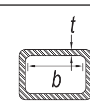
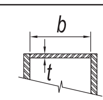
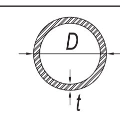
(b) For webs of built-up sections, $h$ is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used.

(c) For flange or diaphragm plates in built-up sections, the width $b$ is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections (HSS), the width $b$ is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, $h$ is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, $b$ and $h$ should be taken as the corresponding outside dimension minus three times the thickness. The thickness, $t$, is the wall thickness.

*Design Wall Thickness for HSS*

The design wall thickness is equal to the nominal wall thickness for stainless steel rectangular HSS. (Note: This differs from the requirement for electric-resistance-welded HSS

| Table 3-1. Limiting Width-To-Thickness Ratios for Compression Elements in Members Subject to Axial Compression | | | | | |
|---|---|---|---|---|---|
| | **Case** | **Description of Element** | **Width-to-Thickness Ratio** | **Limiting Width-to-Thickness Ratio, $\lambda_r$ (nonslender/ slender)** | **Example** |
| Unstiffened Elements | 1 | Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees | $b/t$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |  |
| | 2 | Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections | $b/t$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |  |
| | 3 | Legs of single angles, legs of double angles with separators, stems of tees and all other unstiffened elements | $b/t$ | $0.38\sqrt{\dfrac{E}{F_y}}$ |  |
| Stiffened Elements | 4 | Webs of doubly symmetric I-shaped sections and channels | $h/t_w$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |  |
| | 5 | Walls of rectangular HSS and boxes of uniform thickness | $b/t$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |  |
| | 6 | All other stiffened elements | $b/t$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |  |
| | 7 | Round HSS | $D/t$ | $0.10\dfrac{E}{F_y}$ |  |

$E$ = modulus of elasticity of steel, ksi (MPa), given in Table 2-9
$F_y$ = specified minimum yield stress, ksi (MPa), given in Table 2-2

made from carbon steel where the design wall thickness is equal to 0.93 times the nominal wall thickness.)

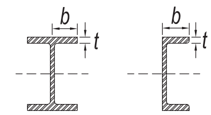
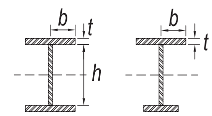
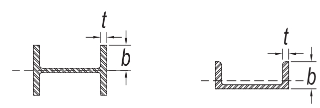
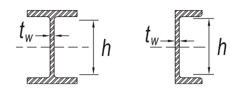
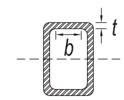
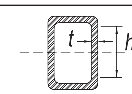
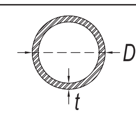
### 3.3.2 Gross and Net Area Determination

*Gross Area*

The gross area, $A_g$, of a member is the total cross-sectional area.

*Net Area*

The net area, $A_n$, of a member is the sum of the products of the thickness and the net width of each element and can be calculated for stainless steel members in the same way as for carbon steel members in the AISC *Specification* Section B4.3.

| Table 3-2. Limiting Width-To-Thickness Ratios for Compression Elements in Members Subject to Flexure | | | | | | |
|---|---|---|---|---|---|---|
| | Case | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratios | | Example |
| | | | | $\lambda_p$ (compact / noncompact) | $\lambda_r$ (noncompact / slender) | |
| Unstiffened Elements | 8 | Flanges of rolled I-shaped sections and channels | $b/t$ | $0.33\sqrt{\dfrac{E}{F_y}}$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |  |
| | 9 | Flanges of doubly and singly symmetric I-shaped built-up sections | $b/t$ | $0.33\sqrt{\dfrac{E}{F_y}}$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |  |
| | 10 | Flanges of all I-shaped sections and channels in flexure about the weak axis | $b/t$ | $0.33\sqrt{\dfrac{E}{F_y}}$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |  |
| Stiffened Elements | 11 | Webs of doubly symmetric I-shaped sections and channels | $h/t_w$ | $2.54\sqrt{\dfrac{E}{F_y}}$ | $3.01\sqrt{\dfrac{E}{F_y}}$ |  |
| | 12 | Flanges of rectangular HSS and boxes of uniform thickness | $b/t$ | $1.12\sqrt{\dfrac{E}{F_y}}$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |  |
| | 13 | Webs of rectangular HSS and boxes | $h/t$ | $2.42\sqrt{\dfrac{E}{F_y}}$ | $3.01\sqrt{\dfrac{E}{F_y}}$ |  |
| | 14 | Round HSS | $D/t$ | $0.07\dfrac{E}{F_y}$ | $0.31\dfrac{E}{F_y}$ |  |

$E$ = modulus of elasticity of steel, ksi (MPa), given in Table 2-9
$F_y$ = specified minimum yield stress, ksi (MPa), given in Table 2-2

### 3.3.3 Compact Sections in Flexure

Compact sections can be designed to their full plastic moment, $F_yZ$ (Chapter 6). The possible occurrence of any plastic deformations should be included in the estimate of deflections under service loading.

# Chapter 4
# Design of Members for Tension

## 4.1 AUSTENITIC AND DUPLEX STAINLESS STEEL TENSION MEMBERS

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, $P_n/\Omega_t$, of tension members should be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \qquad (\textit{Spec}.\ \text{Eq. D2-1})$$

$$\phi_t = 0.90\ \text{(LRFD)} \quad \Omega_t = 1.67\ \text{(ASD)}$$

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \qquad (\textit{Spec}.\ \text{Eq. D2-2})$$

$$\phi_t = 0.75\ \text{(LRFD)} \quad \Omega_t = 2.00\ \text{(ASD)}$$

where

$A_e$ = effective net area, in.$^2$ (mm$^2$)
$A_g$ = gross area of member, in.$^2$ (mm$^2$)
$F_y$ = specified minimum yield stress, ksi (MPa)
$F_u$ = specified minimum tensile strength, ksi (MPa)

Larger deformations are expected with stainless steel than with carbon steel, because stainless steel is twice as ductile and also exhibits strong strain hardening ($F_u/F_y$ is approximately 2.2 for austenitic stainless steels). For structures that are sensitive to deformations at serviceability, the following additional check should be undertaken under service loads:

$$P_n = F_y A_e \qquad (4\text{-}1)$$

The guidance for carbon steel in AISC *Specification* Section D3 is applicable to the calculation of effective net area.

The design of pin-connected members and eyebars is outside the scope of this Design Guide.

## 4.2 PRECIPITATION HARDENING STAINLESS STEEL TENSION RODS

The design tensile strength, $\phi_{tph} P_n$, and the allowable tensile strength, $P_n/\Omega_{tph}$, of an unthreaded tension rod of precipitation hardening Type S17400 stainless steel in accordance with ASTM A564/A564M which fails by tensile yielding in the gross section is given by:

$$P_n = F_y A_g \qquad (\textit{Spec}.\ \text{Eq. D2-1})$$

$$\phi_{tph} = 0.80\ \text{(LRFD)} \quad \Omega_{tph} = 1.88\ \text{(ASD)}$$

If the ends of the rod are threaded, then the available tensile strength of the threaded portion should also be checked (see Section 9.3.4).

# Chapter 5
# Design of Members for Compression

The design of unequal leg angles, slender equal leg angles, and slender circular hollow sections is outside the scope of this Design Guide.

## 5.1 GENERAL PROVISIONS

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, $P_n/\Omega_c$, should be determined as follows:

The nominal compressive strength, $P_n$, should be the lowest value obtained based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling.

$\phi_c = 0.85$ (LRFD) $\Omega_c = 1.76$ (ASD) for round HSS

$\phi_c = 0.90$ (LRFD) $\Omega_c = 1.67$ (ASD) for all other structural sections

The rules in this chapter apply to austenitic and duplex stainless steels.

## 5.2 EFFECTIVE LENGTH

The effective length factor, $K$, for calculation of member slenderness, $KL/r$, should be determined in the same way as for carbon steel,

where

$L$ = laterally unbraced length of the member, in. (mm)
$r$ = radius of gyration, in. (mm)

## 5.3 FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender element compression members as defined in Section 3.3.1 for elements in uniform compression.

The nominal compressive strength, $P_n$, should be determined based on the limit state of flexural buckling.

$$P_n = F_{cr} A_g \qquad \text{(\textit{Spec.} Eq. E3-1)}$$

The critical stress, $F_{cr}$, is determined as follows:

(a) When $\dfrac{KL}{r} \le 3.77\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \le 1.44$)

$$F_{cr} = \left(0.50^{\frac{F_y}{F_e}}\right) F_y \qquad \text{(modified \textit{Spec.} Eq. E3-2)}$$

(b) When $\dfrac{KL}{r} > 3.77\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 1.44$)

$$F_{cr} = 0.531 F_e \qquad \text{(modified \textit{Spec.} Eq. E3-3)}$$

where

$F_e$ = elastic buckling stress, which can be determined from AISC *Specification* Equation E3-4, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \qquad \text{(\textit{Spec.} Eq. E3-4)}$$

Appendix A gives an alternative, less conservative method for determining the compressive strength of I-shaped members and rectangular HSS, when:

$\dfrac{KL}{r} \le 0.63\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \le 0.04$)

and

$\bar{\lambda}_p \le 0.68$

where $\bar{\lambda}_p$ is defined in Appendix A, Section A.3.

## 5.4 TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members and certain doubly symmetric members, such as cruciform or built-up columns without slender elements, as defined in Section 3.3.1 for elements in uniform compression. In addition, this section applies to all doubly symmetric members without slender elements when the torsional unbraced length exceeds the lateral unbraced length. These provisions are required for single angles with $b/t > 20$.

The guidelines given in AISC *Specification* Section E4 for carbon steel apply except as follows:

For double angle and tee-shaped compression members, $F_{cry}$ in Equation E4-2 is taken as $F_{cr}$ from modified *Specification* Equations E3-2 and E3-3, for flexural buckling about the $y$-axis (or weak-axis) of symmetry.

For all other cases, $F_{cr}$ should be determined according to modified AISC *Specification* Equations E3-2 and E3-3, using the torsional or flexural-torsional elastic buckling stress, $F_e$, determined from Equations E4-4, E4-5 and E4-6.

## 5.5 SINGLE-ANGLE COMPRESSION MEMBERS AND BUILT-UP MEMBERS

The guidance for carbon steel in AISC *Specification* Sections E5 and E6 applies for the design of single equal-leg angle and built-up members in compression, providing the nominal compressive strength is determined in accordance with Section 5.3 or Section 5.6 of this guide, as appropriate, for axially loaded members. For single angles with $b/t > 20$, Section 5.4 of this guide is used.

## 5.6 MEMBERS WITH SLENDER ELEMENTS

This section applies to compression members with slender elements, as defined in Section 3.3.1 for elements in uniform compression.

The nominal compressive strength, $P_n$, should be the lowest value based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling.

$$P_n = F_{cr}A_g \qquad \text{(\textit{Spec.} Eq. E7-1)}$$

The critical stress, $F_{cr}$, should be determined as follows:

(a) When $\dfrac{KL}{r} \le 3.77\sqrt{\dfrac{E}{QF_y}}$ (or $\dfrac{QF_y}{F_e} \le 1.44$)

$$F_{cr} = Q\left(0.50^{\frac{QF_y}{F_e}}\right)F_y \qquad \text{(modified \textit{Spec.} Eq. E7-2)}$$

(b) When $\dfrac{KL}{r} > 3.77\sqrt{\dfrac{E}{QF_y}}$ (or $\dfrac{QF_y}{F_e} > 1.44$)

$$F_{cr} = 0.531F_e \qquad \text{(modified \textit{Spec.} Eq. E7-3)}$$

where

$F_e$ = elastic buckling stress, calculated using Equations E3-4 and E4-4 of the AISC *Specification* for doubly symmetric members, Equations E3-4 and E4-5 of the AISC *Specification* for singly symmetric members, and Equation E4-6 of the AISC *Specification* for unsymmetric members, ksi (MPa)

$Q$ = net reduction factor accounting for all slender compression elements. $Q$ = 1.0 for members without slender elements, as defined in Section 3.3.1, for elements in uniform compression

= $Q_sQ_a$ for members with slender-element sections

For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_sQ_a$. For cross sections composed of multiple unstiffened slender elements, it is conservative to use the smaller $Q_s$ from the more slender element in determining the member strength for pure compression.

### 5.6.1 Slender Unstiffened Elements, $Q_s$

The reduction factor, $Q_s$, for slender unstiffened elements is defined as follows:

For flanges, angles and plates projecting from rolled or built-up I-shaped columns or other compression members:

(i) When $\dfrac{b}{t} \le 0.47\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.0 \qquad \text{(\textit{Spec.} Eq. E7-4)}$$

(ii) When $0.47\sqrt{\dfrac{E}{F_y}} < \dfrac{b}{t} \le 0.90\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.498 - 1.06\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \qquad \text{(modified \textit{Spec.} Eq. E7-5)}$$

(iii) When $\dfrac{b}{t} > 0.90\sqrt{\dfrac{E}{F_y}}$

$$Q_s = \frac{0.44E}{F_y\left(\dfrac{b}{t}\right)^2} \qquad \text{(modified \textit{Spec.} Eq. E7-6)}$$

### 5.6.2 Slender Stiffened Elements, $Q_a$

The reduction factor, $Q_a$, for slender stiffened elements is defined as follows:

$$Q_a = \frac{A_e}{A_g} \qquad \text{(\textit{Spec.} Eq. E7-16)}$$

where

$A_g$ = gross (total) cross-sectional area of member, in.$^2$ (mm$^2$)

$A_e$ = summation of the effective areas of the cross section based on the reduced effective width, $b_e$, in.$^2$ (mm$^2$)

The reduced effective width, $b_e$, is determined as follows:

For uniformly compressed slender elements, including the flanges of square and rectangular sections, with $\dfrac{b}{t} \ge 1.24\sqrt{\dfrac{E}{f}}$:

$$b_e = 1.468t\sqrt{\frac{E}{f}}\left[1 - \frac{0.194}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b$$

(modified *Spec.* Eq. E7-17)

where

$$f = \frac{P_n}{A_e}$$

$f$ may be taken as equal to $F_y$, which results in a slightly conservative estimate of column available strength.

# Chapter 6
# Design of Members for Flexure

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports. The design of angles and tees in flexure, and sections in flexure where the web is classified as slender, are outside the scope of this chapter.

The possible occurrence of any plastic deformations should be included in the estimate of deflections under service loading.

## 6.1 GENERAL PROVISIONS

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, $M_n/\Omega_b$, should be determined as follows:

For all provisions in this chapter:

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

The rules in this chapter apply to austenitic and duplex stainless steels.

Appendix A gives an alternative, less conservative method for determining the flexural strength of I-shaped members and rectangular HSS when:

$L_b \le 0.75 L_p$

and

$\bar{\lambda}_p \le 0.68$

where $L_p$ is defined in Section 6.2 and $\bar{\lambda}_p$ is defined in Appendix A, Section A.3.

## 6.2 I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR OR MINOR AXIS

The guidelines given in AISC *Specification* Sections F2, F3, F4 and F6 for carbon steel apply except that Equations F2-2, F4-2, F2-3, F4-3, F2-5 and F4-7 have been modified, as given in the following. The guidelines in AISC *Specification* Section F5 are outside the scope of this Design Guide.

For $L_p < L_b \le L_r$

Doubly Symmetric Compact I-shaped Members and Channels

$$M_n = C_b \left[ M_p - \left( M_p - 0.45 F_y S_x \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p$$

(modified *Spec.* Eq. F2-2)

where

$$L_p = 0.8 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(modified } \textit{Spec.} \text{ Eq. F2-5)}$$

Other I-shaped Members with Compact or Noncompact Webs

$$M_n = C_b \left[ R_{pc} M_{yc} - \left( R_{pc} M_{yc} - 0.64 F_L S_{xc} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_{pc} M_{yc}$$

(modified *Spec.* Eq. F4-2)

where

$$L_p = 0.5 r_t \sqrt{\frac{E}{F_y}} \qquad \text{(modified } \textit{Spec.} \text{ Eq. F4-7)}$$

For $L_b > L_r$

$$M_n = 0.64 F_{cr} S_x \le M_p \qquad \text{(modified } \textit{Spec.} \text{ Eq. F2-3)}$$

Other I-shaped Members with Compact or Noncompact Webs

$$M_n = 0.64 F_{cr} S_{xc} \le R_{pc} M_{yc} \qquad \text{(modified } \textit{Spec.} \text{ Eq. F4-3)}$$

where

$M_p = F_y Z_x$

$F_y$ = specified minimum yield stress of the type of steel being used, ksi (MPa)

$Z_x$ = plastic section modulus taken about the $x$-axis, in.$^3$ (mm$^3$)

$L_b$ = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

$S_x$ = elastic section modulus taken about the $x$-axis, in.$^3$ (mm$^3$)

The terms $C_b$, $F_{cr}$, $L_r$, $r_{ts}$, $R_{pc}$, $M_{yc}$, $F_L$, $S_{xc}$ and $r_t$ are defined in the AISC *Specification*.

The relevant values for the limiting slendernesses used in the expressions for compression flange local buckling and tension flange yielding in Table 3-2 should be used, i.e.,

$\lambda_{pf}$ is the limiting slenderness for a compact flange, given by $\lambda_p$ in Table 3-2.

$\lambda_{rf}$ is the limiting slenderness for a noncompact flange, given by $\lambda_r$ in Table 3-2.

$\lambda_{pw}$ is the limiting slenderness for a compact web, given by $\lambda_p$ in Table 3-2.

$\lambda_{rw}$ is the limiting slenderness for a noncompact web, given by $\lambda_r$ in Table 3-2.

The design of I-shaped members with slender webs bent about the major axis is outside the scope of this Design Guide.

## 6.3 SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular HSS, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Table 3-2.

The nominal flexural strength, $M_n$, should be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling, and web local buckling under pure flexure.

In most practical cases, rectangular HSS with $h/b \leq 3$ will not be susceptible to lateral-torsional buckling. For longer lengths, beam deflection is likely to be critical. The Design Guide provides no strength equation for this limit state for rectangular HSS.

### 6.3.1 Yielding

$$M_n = M_p = F_y Z \qquad \text{(}\textit{Spec}\text{. Eq. F7-1)}$$

where

$Z$ = plastic section modulus about the axis of bending, in.$^3$ (mm$^3$)

### 6.3.2 Flange Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - F_y S\right)\left(8.33\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 9.33\right) \leq M_p$$

(modified *Spec*. Eq. F7-2)

(c) For sections with slender flanges

$$M_n = F_y S_e \qquad \text{(}\textit{Spec}\text{. Eq. F7-3)}$$

where

$S_e$ = effective section modulus determined with the effective width, $b_e$, of the compression flange taken as:

$$b_e = 1.468 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.194}{\left(b/t_f\right)}\sqrt{\frac{E}{F_y}}\right] \leq b$$

(modified *Spec*. Eq. F7-4)

### 6.3.3 Web Local Buckling

(a) For compact sections, the limit state of web local buckling does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S\right)\left(1.69\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 4.09\right) \leq M_p$$

(modified *Spec*. Eq. F7-5)

## 6.4 ROUND HSS

The guidelines given in AISC *Specification* Section F8 for bending of round HSS carbon steel sections apply except for the design of slender round HSS which is beyond the scope of this Design Guide.

## 6.5 RECTANGULAR BARS AND ROUNDS

For square and round bars, and rectangular bars bent around their minor axis, the guidance in AISC *Specification* Section F11 applies.

The design of rectangular bars bent about their major axis which are susceptible to lateral-torsional buckling is beyond the scope of this Design Guide. Note that bars with a depth-to-width ratio, $d/t$, less than 2 are not susceptible to lateral-torsional buckling.

Because the shape factor for a rectangular cross section is 1.5 and for a round section is 1.7, consideration must be given to serviceability issues such as excessive deflection or permanent deformation under service-load conditions.

## 6.6 UNSYMMETRICAL SHAPES, EXCLUDING SINGLE ANGLES

AISC *Specification* Section F12 recommendations apply for bending of unsymmetrical shapes.

## 6.7 DETERMINATION OF DEFLECTION

Deflections should be determined for the load combination at the relevant serviceability limit state. The deflection of elastic beams (i.e., those not containing a plastic hinge) may be estimated by standard structural theory, except that the secant modulus of elasticity, $E_s$, should be used instead of the initial modulus of elasticity. This is the only section in the Design Guide where the designer directly adjusts the modulus of elasticity to account for the stress level in the

| Table 6-1. Values of Constants to be Used for Determining Secant Moduli | | | | |
|---|---|---|---|---|
| Stainless Steel | | $F_y$ ksi (MPa) | $n$ | $E_s$[a] ksi (MPa) |
| Austenitic | S30400 and S31600 | 30 (205) | 5.6 | 23,800 (164,000) |
| | S30403 and S31603 | 25 (170) | | 23,000 (159,000) |
| Duplex | S32101 and S32205 | 65 (450) | 7.2 | 27,900 (193,000) |
| | S32304 | 58 (400) | | 27,800 (192,000) |

[a] This is the secant modulus corresponding to $F_{ser}/F_y = 0.6$, which can be adopted in preliminary estimates of deflection.

stainless steel member. $E_s$ varies with the stress level in the beam, but as a simplification, this variation may be neglected and the minimum value of $E_s$ for that member (corresponding to the maximum values of the stress in the member) may be used throughout its length.

This is a simplified method that is accurate for predicting deflections when the secant modulus is based on the maximum stress in the member and this maximum stress does not exceed 65% of the 0.2% offset yield strength. At higher levels of stress, the method becomes very conservative and a more accurate method (e.g., one that involves integrating along the length of the member) should be used.

The value of the secant modulus of elasticity, $E_s$, can be estimated from the following equation using the constants given in Table 6-1.

$$E_s = \frac{E}{1+0.002\dfrac{E}{F_{ser}}\left(\dfrac{F_{ser}}{F_y}\right)^n} \qquad \text{(6-1)}$$

where

$F_{ser}$ = maximum serviceability design stress

$E$ = initial modulus of elasticity (Table 2-9)

$n$ = Ramberg Osgood parameter, derived from the stress at the limit of proportionality. It is a measure of the nonlinearity of the stress-strain curve, with lower values of $n$ indicating a greater degree of nonlinearity

Table 6-1 also gives values for $E_s$ that correspond to $F_{ser}/F_y = 0.6$, which can be adopted in preliminary estimates of deflection.

# Chapter 7
# Design of Members for Shear

The design shear strength, $\phi_v V_n$, and the allowable shear strength, $V_n/\Omega_v$, should be determined as follows:

For all provisions in this chapter:

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

The guidance in AISC *Specification* Chapter G applies to austenitic and duplex stainless steels, including the expressions for $C_v$ given in Section G2.

AISC *Specification* Section G3 (Tension Field Action) and the limit state of shear buckling for round HSS (Equations G6-2a and G6-2b) are beyond the scope of this Design Guide.

# Chapter 8
# Design of Members for Combined Forces

The rules in this chapter apply to austenitic and duplex stainless steels. The design of members subject to torsion and combined torsion, flexure, shear and/or axial force is beyond the scope of this Design Guide.

## 8.1 DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

### 8.1.1 Doubly and Singly Symmetric Members Subject to Flexure and Compression

The guidance in AISC *Specification* Section H1.1 for carbon steel applies. The appropriate resistance factors or safety factors for stainless steel should be used.

For design according to LRFD, $P_c = \phi_c P_n$ and $M_c = \phi_b \text{M}_n$, where $\phi_c$ and $\phi_b$ are the resistance factors for compression and flexure given in Chapters 5 and 6, respectively.

For design according to ASD, $P_c = P_n/\Omega_c$ and $M_c = M_n/\Omega_b$, where $\Omega_c$ and $\Omega_b$ are the safety factors for compression and flexure given in Chapters 5 and 6, respectively.

### 8.1.2 Doubly and Singly Symmetric Members Subject to Flexure and Tension

The guidance in the AISC *Specification* for carbon steel applies. The appropriate resistance factors or safety factors for stainless steel should be used.

For design according to LRFD, $P_c = \phi_t P_n$ and $M_c = \phi_b M_n$, where $\phi_t$ and $\phi_b$ are the resistance factors for tension and flexure given in Chapters 4 and 6, respectively.

For design according to ASD, $P_c = P_n/\Omega_t$ and $M_c = M_n/\Omega_b$, where $\Omega_t$ and $\Omega_b$ are the safety factors for tension and flexure given in Chapters 4 and 6, respectively.

## 8.2 UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

The guidance in AISC *Specification* Section H2 for carbon steel applies. The appropriate resistance factors or safety factors for stainless steel should be used.

# Chapter 9
# Design of Connections

## 9.1 DURABILITY

The design of joints needs careful attention to maintain optimum corrosion resistance. This is especially true for joints that may become wet from the weather, spray, immersion, or condensation, etc. The possibility of avoiding or reducing associated corrosion problems by locating joints away from the source of dampness should be investigated. Alternatively, it may be possible to remove the source of dampness; for instance, in the case of condensation, by adequate ventilation or by ensuring that the ambient temperature within the structure lies above the dew point temperature.

Where it is not possible to prevent a joint involving carbon steel and stainless steel from becoming wet, consideration should be given to preventing galvanic corrosion, as discussed in Section 2.6.2. The use of carbon steel bolts with stainless steel structural elements should always be avoided. In bolted joints that would be prone to an unacceptable degree of corrosion, provision should be made to electrically isolate the carbon steel and stainless steel elements. This entails the use of insulating washers and possibly bushings; typical suitable details for bolts installed to the snug-tight condition are shown in Figure 9-1. The insulating washers and bushings should be made of a thermoset polymer such as neoprene (synthetic rubber), which is flexible enough to seal the joint when adequate pressure is applied and long lasting to provide permanent metal separation. Sealing the joint is important to prevent moisture infiltration which would lead to crevice corrosion. Note also that the insulating washer should not extend beyond the stainless steel washer in case a crevice is created. In atmospheric conditions with chloride exposure, an additional strategy to protect against crevice corrosion is to insert an insulating, flexible washer directly under the bolt head, or to cover the area with clear silicone sealant. Care should be taken in selecting appropriate materials for the environment to avoid crevice corrosion in bolted joints (see Section 2.6.2).

For welded joints involving carbon and stainless steels, it is generally recommended that any paint system applied to the carbon steel should extend over the weldment onto the stainless steel up to a distance of about 2 in. (50 mm).

The heating and cooling cycle involved in welding affects the microstructure of all stainless steels; this is of particular importance for duplex materials. It is essential that suitable welding procedures and filler metals are used and that qualified welders undertake the work. Guidance on this matter is given in Section 12.5.

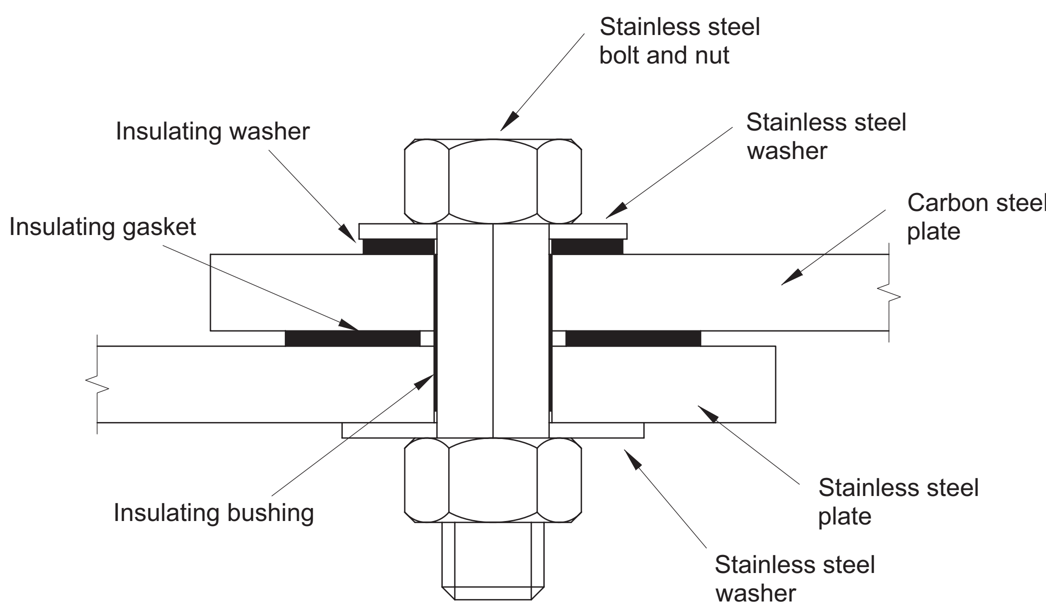



*Fig. 9-1. Typical detail for connecting dissimilar materials (to avoid galvanic corrosion) for bolts installed to the snug-tight condition.*

| Table 9-1. Prequalified Filler Metal Classifications from AWS D1.6/D1.6M | | | |
|---|---|---|---|
| **AWS A5.4/A5.4M:2006** | **AWS A5.9/A5.9M:2006** | **AWS A5.22-95R** | **AWS A5.30-97** |
| Filler Metal Group A—70 ksi (490 MPa) Minimum Tensile Strength, $F_{EXX}$ | | | |
| E316L-XX | ER316L | E316LTX-X | IN316L |
| | ER316LSi | R316LT1-5 | |
| | EC316L | | |
| Filler Metal Group B—75 ksi (520 MPa) Minimum Tensile Strength, $F_{EXX}$ | | | |
| E308L-XX | ER308L | E308LTX-X | IN308L |
| E308MoL-XX | ER308MoL | E308LMoTX-X | IN316 |
| E309L-XX | ER309L | E309LTX-X | |
| E309MoL-XX | ER309MoL | E309LMoTX-X | |
| E316-XX | ER316 | E309LCbTX-X | |
| E316HL-XX | ER316H | E316TX-X | |
| E317L-XX | ER317L | E317LTX-X | |
| E347L-XX | ER347 | E347TX-X | |
| | | R308LT1-5 | |
| | | R309LT1-5 | |
| | | R347T1-5 | |

Note:
1. The base metal grouping of the following prequalified austenitic stainless steels is as follows:
   Base Metal Group A: S30403 and S31603 to ASTM A240 and ASTM A276
   Base Metal Group B: S30400 and 316 to ASTM A240, A276, A554 and S30403 and S31603 to ASTM A554
2. Filler metals of Group B are prequalified for Group A base metals. For prequalified base and filler metals groups of higher strength, refer to Clause 3 of AWS D1.6/D1.6M.

## 9.2 DESIGN OF WELDED CONNECTIONS

Welding should be carried out in accordance with AWS D1.6/D1.6M. It is essential that welds are made using correct procedures, including compatible filler metals, with suitably qualified welders (see also Section 12.5). This is important not only to ensure the strength of the weld and to achieve a defined weld profile but also to maintain the corrosion resistance of the weld and surrounding material. The use of a compatible filler metal results in the weld yield strength and ultimate strengths exceeding those of the parent material. The choice of filler metal should comply with the requirements for matching filler metals given in AWS D1.6/D1.6M. If questions arise because of unusual conditions, metal combinations, or if proprietary types of stainless steel have been selected, the stainless steel producer should be consulted.

The guidance in AISC *Specification* Section J2, Welds, concerning the calculation of the effective area and limitations applies. The guidance in the AISC *Specification* for calculating the strength of groove, fillet, and plug and slot welds also applies, providing the following reduced resistance factor and safety factors are used:

$$\phi = 0.55 \text{ (LRFD)} \quad \Omega = 2.70 \text{ (ASD)}$$
for austenitic stainless steels

$$\phi = 0.60 \text{ (LRFD)} \quad \Omega = 2.50 \text{ (ASD)}$$
for duplex stainless steels

Table 9-1 summarizes the prequalified filler metals for welding Type S30400/S30403 and Type S31600/S31603 and the corresponding minimum ultimate tensile strengths, $F_{EXX}$. For a complete list of base metals and prequalified matching filler metals, see AWS D1.6/D1.6M. Duplex stainless steels are not prequalified so a welding procedure specification (WPS) qualification is required in accordance with Clause 4 of AWS D1.6/D1.6M.

For austenitic stainless steels (especially the low carbon varieties), the ratio of yield strength to typical filler metal ultimate strength is low. Therefore, the weld strength for fillet and partial-joint-penetration groove welds is more likely to be governed by the strength of the base metal, which is not often the case for structural carbon steels.

It should be noted that greater welding distortions are associated with the austenitic stainless steels than with carbon steels (see Section 12.5.4). Attention should also be paid to the requirements for subsequent inspection and maintenance.

## 9.3 DESIGN OF BOLTED CONNECTIONS

### 9.3.1 General

The design of joints formed with stainless steel bolts is outside the scope of the *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2009). The following recommendations apply to bolts installed in clearance holes to the snug-tight condition (i.e., the tightness required to bring the connected plies into firm contact) and loaded in shear, tension, or a combination of shear and tension. The recommendations only apply to connections where the shear forces are transferred by bearing between the bolts and the connected parts. No recommendations are given for connections in which shear is transferred by frictional resistance, as in slip-critical connections; however, see Section 9.3.2 for the use of pretensioned bolts.

It is good practice to provide stainless steel washers under both the bolt head and the nut. Guidance on appropriate materials for bolts and nuts is given in Section 2.3.2. Because installation tension for snug-tight stainless steel fasteners is not as high or as well controlled as it is for high-strength steel bolts, the use of lock washers is quite common with stainless steel fasteners. Lock washers are placed under the nut and help to reduce loosening due to structure vibration and load fluctuation (FHWA, 2005).

Holes can be formed by drilling or punching. However, the cold working associated with punching may increase the susceptibility to corrosion and therefore punched holes are less suitable in aggressive environments (e.g., heavy industrial and marine environments).

The strength of a connection is to be taken as the lesser of the strength of the connected parts (Section 9.4) and that of the fasteners (Section 9.3). To restrict irreversible deformation in bolted connections, the stresses in bolts and the net cross section of the connecting material at bolt holes under service loads should be limited to the yield strength (see Section 4.1).

### 9.3.2 Pretensioned Bolts

Stainless steel bolts may be used as pretensioned bolts provided appropriate tensioning techniques are used. If stainless steel bolts are highly torqued, galling can be a problem (see Section 12.7). When pretension is applied, consideration should be given to time-dependent stress relaxation. Connections should not be designed as slip resistant at either the serviceability or ultimate limit state unless acceptability in the particular application can be demonstrated by testing. Slip coefficients for stainless steel faying surfaces are likely to be lower than those for carbon steel faying surfaces.

### 9.3.3 Size and Use of Holes, Spacing and Edge Distance

Guidance in AISC *Specification* Sections J3.2, J3.3, J3.4 and J3.5 on size of holes, minimum spacing, minimum edge distance, maximum spacing, and edge distance applies to stainless steel.

### 9.3.4 Tension and Shear Strength of Bolts and Threaded Parts

The design tension or shear strength, $\phi R_n$, and the allowable tension or shear strength, $R_n/\Omega$, of a snug-tightened bolt or threaded part should be determined according to the limit states of tension rupture and shear rupture as follows:

$$R_n = F_n A_b \qquad (\textit{Spec.}\text{ Eq. J3-1})$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

$A_b$ = nominal unthreaded body area of bolt or threaded part, in.$^2$ (mm$^2$)

$F_n$ = nominal tensile stress, $F_{nt}$, or shear stress, $F_{nv}$, ksi (MPa)

$F_{nt} = 0.75F_u$

$F_{nv} = 0.45F_u$ if threads are not excluded from the shear planes

$= 0.55F_u$ if threads are excluded from the shear planes

The value for $F_u$ should be taken as the specified minimum tensile strength of the bolt given in the relevant ASTM standard (see Table 2-4 through Table 2-8).

The required tensile strength should include any tension resulting from prying action produced by deformation of the connected parts.

Note that these design rules only apply where matching bolt/nut assemblies are used to preclude the possibility of failure by thread stripping, i.e., bolts in accordance with ASTM F593 (or F738M) used with nuts in accordance with ASTM F594 (or F836M) and with dimensions in accordance with ASME B18.2.1 (ASME, 2012).

The preceding design rules can also be applied to precipitation hardening fasteners in accordance with ASTM F593 or F738M with the following resistance and safety factors:

$$\phi_{ph} = 0.67 \text{ (LRFD)} \quad \Omega_{ph} = 2.25 \text{ (ASD)}$$

The force that can be resisted by a snug-tightened bolt or threaded part may be limited by the bearing strength of the material at the bolt hole (Section 9.3.6). The effective strength of an individual fastener may be taken as the lesser of the fastener shear strength according to Section 9.3.4 or the bearing strength of the material at the bolt hole per Section 9.3.6. The strength of the bolt group is taken as the sum of the effective strengths of the individual fasteners.

### 9.3.5 Combined Tension and Shear in Bearing-Type Connections

The guidance in AISC *Specification* Section J3.7 applies to combined tension and shear in bearing-type connections with stainless steel bolts.

The required tensile strength (including any force due to prying action) must also be less than the available tensile strength.

### 9.3.6 Bearing Strength at Bolt Holes

The available bearing strength at bolt holes, $\phi R_n$ and $R_n/\Omega$, should be determined for the limit state of bearing as follows:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

(a) For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force, the nominal bearing strength of the connected material, $R_n$, is determined as follows:

(i) When deformation at the bolt hole at service load is a design consideration

$$R_n = \alpha_d t d F_u \tag{9-1}$$

where

$$\alpha_d = 1.25\left(\frac{e_1}{2d_h}\right) \le 1.25 \text{ for end bolts} \tag{9-2}$$

$$\alpha_d = 1.25\left(\frac{p_1}{4d_h}\right) \le 1.25 \text{ for inner bolts} \tag{9-3}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = \alpha_1 t d F_u \tag{9-4}$$

where

$$\alpha_1 = 2.5\left(\frac{e_1}{3d_h}\right) \le 2.5 \tag{9-5}$$

for $e_2/d_h > 1.5$ for end bolts

$$\alpha_1 = 2.5\left(\frac{e_1}{3d_h}\right) \le 2.0 \tag{9-6}$$

for $e_2/d_h \le 1.5$ for end bolts

$$\alpha_1 = 2.5\left(\frac{p_1}{6d_h}\right) \le 2.5 \tag{9-7}$$

for $p_2/d_h > 3.0$ for inner bolts

$$\alpha_1 = 2.5\left(\frac{p_1}{6d_h}\right) \le 2.0 \tag{9-8}$$

for $p_2/d_h \le 3.0$ for inner bolts

Note that the use of Equation 9-4 may lead to the occurrence of plastic deformation under service loads.

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force

$$R_n = 0.6\alpha_1 t d F_u \tag{9-9}$$

where

$F_u$ = specified minimum tensile strength of the connected material, ksi (MPa)
$d$ = nominal bolt diameter, in. (mm)
$e_1$ = minimum value of the end distance, in. (mm)
$e_2$ = minimum value of the edge distance, in. (mm)
$p_1$ = minimum value of the center-to-center spacing of bolts in the direction of stress, in. (mm)
$p_2$ = minimum value of the center-to-center spacing of bolts normal to the direction of stress, in. (mm)
$d_h$ = hole diameter, in. (mm)
$t$ = thickness of connected material, in. (mm)

Figure 9-2 shows the symbols used to define the position of holes.

For connections, the bearing resistance should be taken as the sum of the bearing resistances of the individual bolts.

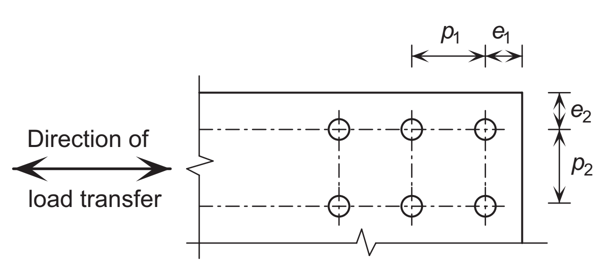



*Fig. 9-2. Symbols for defining position of holes.*

### 9.3.7 Special Fasteners

The nominal strength of special fasteners other than the bolts covered in Section 2.3.2 should be verified by tests.

## 9.4 AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

The guidance for strength of elements in tension in AISC *Specification* Section J4.1 applies to stainless steel except that the stainless steel resistance and safety factors from Chapter 4 should be used. The guidance for strength of elements in shear in AISC *Specification* Section J4.2 applies to stainless steel. For shear yielding, the stainless steel resistance and safety factors in Chapter 7 should be used. For shear rupture, the following resistance and safety factors apply:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

The guidance for block shear strength of elements in AISC *Specification* Section J4.3 applies to stainless steel, with the following resistance and safety factors:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

The guidance for strength of elements in compression in AISC *Specification* Section J4.4 applies to stainless steel, provided the stainless steel resistance and safety factors from Chapter 5 are used. When $KL/r > 25$, the provisions of Chapter 5 apply.

The guidance for strength of elements in flexure in AISC *Specification* Section J4.5 applies to stainless steel, provided the stainless steel resistance and safety factors from Chapter 6 are used.

## 9.5 BEARING STRENGTH

The guidance for the bearing strength of surfaces in contact (finished surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners) in AISC *Specification* Section J7, including the resistance and safety factors, applies.

## 9.6 FLANGES AND WEBS WITH CONCENTRATED FORCES

The guidance in AISC *Specification* Section J10 applies to stainless steel members.

# Chapter 10
# Fire Resistance

This chapter provides criteria for the design and evaluation of austenitic and duplex stainless steel structural members under fire loading conditions. Guidance on the mechanical properties of precipitation hardening Type S17400 at high temperatures can be found in *Metallic Materials Properties, Development and Standardization* (FAA, 2003).

## 10.1 GENERAL PROVISIONS

The guidance in AISC *Specification* Appendix 4, Section 4.1 applies.

## 10.2 STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

The guidance in AISC *Specification* Appendix 4, Sections 4.2.1 and 4.2.2 applies.

### 10.2.1 Thermal Elongation

Table 10-1 gives the mean coefficient of thermal expansion for austenitic and duplex stainless steels.

The mean coefficient of thermal expansion for precipitation hardening Type S17400 is $6.1\times10^{-6}$/°F ($11.0\times10^{-6}$/°C) in the temperature range 68 to 212 °F (20 to 100 °C).

### 10.2.2 Mechanical Properties at Elevated Temperature

Reduction factors are used to calculate the deterioration of strength and stiffness at elevated temperatures. The reduction factors are the ratio of the design values at elevated temperature normalized by their corresponding values at ambient temperature, assumed to be 68 °F (20 °C). The reduction factors corresponding to the modulus of elasticity, $k_E(T)$, specified minimum yield stress, $k_y(T)$, and specified minimum tensile strength, $k_u(T)$, are given in Tables 10-2 to 10-5. It is permitted to interpolate between the values provided.

### 10.2.3 Specific Heat

The specific heat of austenitic and duplex stainless steel, $c_s$, may be determined from the following:

$$c_s = 0.107 + 0.372\times10^{-5}(T_s - 32) - 2.15\times10^{-8}(T_s - 32)^2 - 5.49\times10^{-12}(T_s - 32)^3 \text{ BTU/(lb-°F)} \tag{10-1}$$

$$c_s = 450 + 0.280T_s - \left(2.91\times10^{-4}\right)T_s^2 + \left(1.34\times10^{-7}\right)T_s^3 \quad \text{J/kg-K (S.I.)} \tag{10-1M}$$

### 10.2.4 Emissivity

The parameter, $\varepsilon_F$, which accounts for the emissivity of the fire and the view factor, can be taken as 0.4 for stainless steel.

## 10.3 STRUCTURAL DESIGN REQUIREMENTS

The guidance in AISC *Specification* Appendix 4, Sections 4.2.4.1, 4.2.4.2 and 4.2.4.3a applies.

### 10.3.1 Simple Methods of Analysis

*Tension Members*

The guidance in AISC *Specification* Appendix 4, Section 4.2.4.3b(1) applies.

*Compression Members*

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as determined by the design-basis fire defined in the AISC *Specification* Appendix 4, Section 4.2.1.

The design strength of a compression member should be determined using the provisions of Chapter 5 of this Design Guide with stainless steel properties as stipulated in Section 10.2.2, as follows:

(a) When $\dfrac{F_y(T)}{F_e(T)} \le 1.44$

$$F_{cr}(T) = \left(0.50^{\frac{F_y(T)}{F_e(T)}}\right)F_y(T) \tag{10-2}$$

(b) When $\dfrac{F_y(T)}{F_e(T)} > 1.44$

$$F_{cr}(T) = 0.531F_e(T) \tag{10-3}$$

where

$$F_e(T) = \frac{\pi^2 E(T)}{\left(\dfrac{KL}{r}\right)^2} \tag{10-4}$$

where $F_y(T)$ is the yield stress at elevated temperature and $E(T)$ is the modulus of elasticity at elevated temperature. $F_y(T)$ and $E(T)$ are obtained using coefficients from Tables 10-2 to 10-5.

| Table 10-1. Mean Coefficient of Thermal Expansion | | | | |
|---|---|---|---|---|
| Steel Temperature Range | Austenitic Stainless Steel | | Duplex Stainless Steel | |
| °F (°C) | $10^{-6}$/°F | $10^{-6}$/°C | $10^{-6}$/°F | $10^{-6}$/°C |
| 68 (20) – 200 (93) | 9.3 | 16.7 | 7.3 | 13.1 |
| 68 (20) – 400 (204) | 9.6 | 17.3 | 7.7 | 13.9 |
| 68 (20) – 600 (316) | 9.8 | 17.7 | 7.9 | 14.2 |
| 68 (20) – 800 (427) | 10.1 | 18.2 | 8.2 | 14.8 |
| 68 (20) – 1000 (538) | 10.3 | 18.6 | 8.4 | 15.1 |
| 68 (20) – 1200 (649) | 10.6 | 19.1 | 8.7 | 15.7 |
| 68 (20) – 1400 (760) | 10.7 | 19.3 | 8.9 | 16.0 |
| 68 (20) – 1600 (871) | 10.8 | 19.5 | 9.3 | 16.7 |
| 68 (20) – 1800 (982) | 10.9 | 19.6 | 9.5 | 17.1 |
| 68 (20) – 2000 (1090) | 11.1 | 20.0 | 9.7 | 17.5 |

| Table 10-2. Reduction Factors for Type S30400/S30403 Stainless Steel | | | |
|---|---|---|---|
| Steel Temperature, $T$ °F (°C) | $k_E(T) = E(T)/E$ | $k_y(T) = F_y(T)/F_y$ | $k_u(T) = F_u(T)/F_u$ |
| 68 (20) | 1.00 | 1.00 | 1.00 |
| 200 (93) | 0.96 | 0.80 | 0.83 |
| 400 (204) | 0.92 | 0.65 | 0.72 |
| 600 (316) | 0.87 | 0.59 | 0.68 |
| 750 (399) | 0.84 | 0.55 | 0.66 |
| 800 (427) | 0.83 | 0.54 | 0.65 |
| 1000 (538) | 0.78 | 0.48 | 0.58 |
| 1200 (649) | 0.74 | 0.42 | 0.47 |
| 1400 (760) | 0.66 | 0.30 | 0.31 |
| 1600 (871) | 0.50 | 0.18 | 0.16 |
| 1800 (982) | 0.24 | 0.08 | 0.09 |
| 2000 (1090) | 0.11 | 0.05 | 0.05 |

*Flexural Members*

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate the bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member should be determined using the provisions of Chapter 6 of this Design Guide with steel properties as stipulated in Section 10.2.2. AISC *Specification* Equation A-4-3 and a modified Equation A-4-4 are used in lieu of modified Equations F2-2 and F2-3 to calculate the nominal flexural strength for lateral-torsional buckling of laterally unbraced doubly symmetric members:

(a) When $L_b \le L_r(T)$

$$M_n(T) = C_b\left[M_r(T) + \left[M_p(T) - M_r(T)\right]\left(1 - \frac{L_b}{L_r(T)}\right)^{c_x}\right]$$

(*Spec*. Eq. A-4-3)

| Table 10-3. Reduction Factors for Type S31600/S31603 Stainless Steel | | | |
|---|---|---|---|
| **Steel Temperature, *T*** <br> **°F (°C)** | $k_E(T) = E(T)/E$ | $k_y(T) = F_y(T)/F_y$ | $k_u(T) = F_u(T)/F_u$ |
| 68 (20) | 1.00 | 1.00 | 1.00 |
| 200 (93) | 0.96 | 0.87 | 0.88 |
| 400 (204) | 0.92 | 0.72 | 0.80 |
| 600 (316) | 0.87 | 0.66 | 0.78 |
| 750 (399) | 0.84 | 0.62 | 0.77 |
| 800 (427) | 0.83 | 0.61 | 0.76 |
| 1000 (538) | 0.78 | 0.58 | 0.71 |
| 1200 (649) | 0.74 | 0.53 | 0.59 |
| 1400 (760) | 0.66 | 0.45 | 0.41 |
| 1600 (871) | 0.50 | 0.27 | 0.23 |
| 1800 (982) | 0.24 | 0.15 | 0.12 |
| 2000 (1090) | 0.11 | 0.07 | 0.07 |

| Table 10-4. Reduction Factors for Type S32304 Stainless Steel | | | |
|---|---|---|---|
| **Steel Temperature, *T*** <br> **°F (°C)** | $k_E(T) = E(T)/E$ | $k_y(T) = F_y(T)/F_y$ | $k_u(T) = F_u(T)/F_u$ |
| 68 (20) | 1.00 | 1.00 | 1.00 |
| 200 (93) | 0.96 | 0.84 | 0.95 |
| 400 (204) | 0.92 | 0.75 | 0.87 |
| 600 (316) | 0.87 | 0.67 | 0.78 |
| 750 (399) | 0.84 | 0.58 | 0.70 |
| 800 (427) | 0.83 | 0.54 | 0.67 |
| 1000 (538) | 0.78 | 0.37 | 0.54 |
| 1200 (649) | 0.74 | 0.21 | 0.40 |
| 1400 (760) | 0.66 | 0.10 | 0.25 |
| 1600 (871) | 0.50 | 0.05 | 0.12 |
| 1800 (982) | 0.24 | — | — |
| 2000 (1090) | 0.11 | — | — |

(b) When $L_b > L_r(T)$

$$M_n(T) = 0.4F_{cr}(T)S_x \qquad \text{(modified } Spec. \text{ Eq. A-4-4)}$$

where

$$F_{cr}(T) = \frac{C_b \pi^2 E(T)}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad (Spec. \text{ Eq. A-4-5})$$

$$L_r(T) = 1.95 r_{ts}\frac{E(T)}{F_y(T)}\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left[\frac{F_y(T)}{E(T)}\right]^2}} \qquad \text{(modified } Spec. \text{ Eq. A-4-6)}$$

$$M_r(T) = 0.4S_x F_y(T) \qquad \text{(modified } Spec. \text{ Eq. A-4-7)}$$

$$M_p(T) = Z_x F_y(T) \qquad (Spec. \text{ Eq. A-4-9})$$

| Table 10-5. Reduction Factors for Types S32101 and S32205 Stainless Steel | | | |
|---|---|---|---|
| Steel Temperature, $T$ °F (°C) | $k_E(T) = E(T)/E$ | $k_y(T) = F_y(T)/F_y$ | $k_u(T) = F_u(T)/F_u$ |
| 68 (20) | 1.00 | 1.00 | 1.00 |
| 200 (93) | 0.96 | 0.84 | 0.96 |
| 400 (204) | 0.92 | 0.70 | 0.91 |
| 600 (316) | 0.87 | 0.64 | 0.87 |
| 750 (399) | 0.84 | 0.60 | 0.82 |
| 800 (427) | 0.83 | 0.58 | 0.79 |
| 1000 (538) | 0.78 | 0.49 | 0.65 |
| 1200 (649) | 0.74 | 0.35 | 0.47 |
| 1400 (760) | 0.66 | 0.20 | 0.28 |
| 1600 (871) | 0.50 | 0.09 | 0.16 |
| 1800 (982) | 0.24 | 0.02 | 0.07 |
| 2000 (1090) | 0.11 | — | — |

$$c_x = 0.53 + \frac{T}{450} \le 3.0 \quad \text{where } T \text{ is in °F}$$

(*Spec*. Eq. A-4-10)

$$c_x = 0.6 + \frac{T}{250} \le 3.0 \quad \text{where } T \text{ is in °C} \quad \text{(S.I.)}$$

(*Spec*. Eq. A-4-10M)

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, are calculated in accordance with Tables 10-2 to 10-5, and other terms are as defined in Chapter 6.

# Chapter 11
# Fatigue

Consideration should be given to metal fatigue in structures or parts of structures subjected to significant levels of repeated stress. No fatigue assessment is normally required for building structures, except for members supporting lifting appliances, rolling loads or vibrating machinery, and for members subject to wind-induced oscillation.

Similar to carbon steel structures, the combination of stress concentrations and defects at welded joints leads to these locations being almost invariably more prone to fatigue failure than other parts of the structure. Guidance on estimating the fatigue strength of carbon steel structures is applicable to austenitic and duplex stainless steels (see Appendix 3 of the AISC *Specification*). Guidance on the mechanical properties of precipitation hardening Type S17400 can be found in *Metallic Materials Properties, Development and Standardization* (FAA, 2003).

Much can be done to reduce the susceptibility of a structure to fatigue by adopting good design practice. This involves judiciously selecting the overall structural configuration and carefully choosing constructional details that are fatigue resistant. The key to fatigue resistant design is a rational consideration of fatigue early in the design process. A fatigue assessment performed only after other design criteria have been satisfied may result in an inadequate or costly structure. It is also important to consider the needs of the fabricator and erector. It is therefore recommended that early consultations be held with them to point out areas of the structure that are subjected to cyclic loading, stress reversals and conditions that could lead to fatigue cracking, to discuss special precautions and to become aware of fabrication and erection problems. In particular, the use of holes or lifting attachments to ease fabrication or erection, and how they might affect fatigue resistance, should be considered during the fatigue evaluation.

It may be possible to eliminate potential fatigue problems by giving due regard to constructional details and avoiding:

- Sharp changes in cross section and stress concentrations in general
- Misalignments and eccentricities
- Small discontinuities such as scratches and grinding marks
- Unnecessary welding of secondary attachments, e.g., lifting lugs
- Partial-joint-penetration groove welds, fillet welds, intermittent welding, and backing strips
- Arc strikes

Although weld improvement techniques such as weld profile control, weld toe grinding, and shot and hammer peening may improve the fatigue strength of a joint, there are insufficient data to quantify the possible benefits for stainless steel. It should also be noted that the techniques are all labor-intensive and require the skill and experience of the operator to achieve maximum benefit. They should not, except in special cases, be seen as a design option.

# Chapter 12
# Fabrication and Erection

## 12.1 INTRODUCTION

The purpose of this Section is to highlight relevant aspects of stainless steel fabrication for the design engineer, including recommendations for good practice. It also enlightens the designer or owner so they may make a preliminary assessment of the suitability of a fabricator to perform the work.

Stainless steel is not a difficult material to work with. However, in some respects it is different from carbon steel and should be treated accordingly. Many fabrication and joining processes are similar to those used for carbon steel, but the different characteristics of stainless steel require special attention in a number of areas. It is important that effective communication is established between the designer and fabricator early in the project to ensure that appropriate fabrication practices can and are adopted.

An overriding objective is to maintain the corrosion resistance of the steel. It is essential that precautions are taken at all stages of storing, handling and forming to minimize influences that jeopardize the formation of the self-repairing passive layer. Special care must be taken to restore the full corrosion resistance of the welded zone. Although essential, the precautions are simple and, in general, are matters of good engineering practice.

It is important to preserve the good surface appearance of stainless steel throughout fabrication, especially when aesthetics are a high priority design criterion. Not only are surface blemishes unsightly, but they are usually unacceptable, as well as being time consuming and expensive to correct. While surface blemishes are normally hidden by paint in carbon steel structures, this is rarely the case for stainless steel structures.

The structural form may be dictated by the availability of materials. It should be recognized that the available range of hot rolled stainless steel sections is more limited than for carbon steel. This results in a greater use of cold-formed and welded members than is normally encountered. Also, because of brake press length capabilities, only relatively short lengths can be formed, which leads to an increased use of splices. In detailing joints, consideration should be given to clearances for bolts near bend radii and to potential fit-up problems arising from weld distortion.

Fabrication and erection should generally be carried out in accordance with the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2010a). *Erection and Installation of Stainless Steel Structural Components* (Euro Inox, 2008) also gives further information.

The relevant standard for welding stainless steels is AWS D1.6/D1.6M.

## 12.2 SAFETY AND HEALTH

Safety and health issues are beyond the scope of this Design Guide; however, one hazard associated with processing stainless steel is the potential exposure to hexavalent chromium during high temperature operations such as welding or heat cutting. For more information, reference should be made to Chromium (VI), 29 CFR 1926.1026 (OSHA, 2006) and AWS/ANSI Z49.1, *Safety in Welding, Cutting and Allied Processes* (AWS, 2005).

## 12.3 STORAGE AND HANDLING

Generally, greater care is required in storing and handling stainless steel than carbon steel to avoid damaging the surface finish (especially for bright annealed or polished finishes) and to avoid surface contamination by carbon steel and iron leading to increased potential for surface corrosion. Storage and handling procedures should be agreed between the relevant parties to the contract in advance of any fabrication and in sufficient detail to accommodate any special requirements. The procedures should cover, for instance, the following items:

- The steel should be inspected immediately after delivery for any surface damage.
- The steel may have a protective plastic or other covering. This should be left on as long as possible, removing it just before final fabrication. The protective covering should be called for in the procurement document if it is required (e.g., for bright annealed finishes).
- If a plastic strippable adhesive backed film is used instead of loosely wrapped plastic sheeting, it must be UV rated to prevent premature deterioration and residual adhesive surface contamination. Furthermore, the film life must be monitored so that it is removed within the manufacturer's suggested service life, which is generally up to 6 months.
- Storage in salt-laden humid atmospheres should be avoided. If this is unavoidable, packaging should prevent salt intrusion. Strippable films should never be left in place if surface salt exposure is expected because they are permeable to both salt and moisture and create the ideal conditions for crevice corrosion.

- Storage racks should not have carbon steel rubbing surfaces and should, therefore, be protected by wooden, rubber, or plastic battens or sheaths. Sheets and plates should preferably be stacked vertically; horizontally stacked sheets may get walked on with a risk of iron contamination and surface damage.
- Carbon steel lifting tackle, e.g., chains, hooks and cleats, should be avoided. Again, the use of isolating materials or the use of suction cups prevents iron pickup. The forks of forklift trucks should also be protected.
- Contact with chemicals including acids, alkaline products, oils and greases (which may stain some finishes) should be avoided.
- Ideally, segregated fabrication areas for carbon steel and stainless steel should be used. Only tools dedicated to stainless steel should be employed (this particularly applies to grinding wheels and wire brushes). Note that wire brushes and wire wool should be of stainless steel and generally in a stainless steel that is equivalent in terms of corrosion resistance (e.g., do not use ferritic or lower alloyed austenitic stainless steel brushes on a more corrosion-resistant stainless steel).
- As a precaution during fabrication and erection, it is advisable to ensure that any sharp burrs formed during shearing operations are removed.
- Consideration should be given to requirements for protecting the finished fabrication during transportation.

## 12.4 SHAPING OPERATIONS

Austenitic stainless steels work harden significantly during cold working. This can be both a useful property, enabling extensive forming during stretch forming without risk of premature fracture, and a disadvantage, especially during machining when special attention to cutting feeds and speeds is required. The rate of work hardening differs with different stainless steels.

### 12.4.1 Cutting

Stainless steel is a relatively expensive material compared to some other metals and care is needed in marking out plates and sheets to avoid waste in cutting. Note that more waste may result if the material has a polishing grain (or a unidirectional pattern) which has to be maintained in the fabrication. If the same grain direction is not maintained, then inverted pieces will reflect light differently and appear to be a different color. Some marking pens/crayons prove difficult to remove, or cause staining, if used directly on the surface (rather than on any protective film); checks should be made that markers are satisfactory and that any solvents used to remove marks are compatible as well.

Stainless steel may be cut by usual methods, e.g., shearing and sawing, but power requirements are greater than those for similar thicknesses of carbon steel due to work hardening. If possible, cutting (and machining in general) should be carried out when the metal is in the annealed (softened) state to limit work hardening and tool wear. Plasma arc techniques are particularly useful for cutting thick plates and profiles up to 5 in. (125 mm) thick and where the cut edges are to be machined, e.g., for weld preparation. Water jet cutting is appropriate for cutting material up to 8 in. (200 mm) thick, without heating, distorting or changing the properties of the stainless steel. Laser cutting is suitable for stainless steel, particularly when tighter tolerances are required or when cutting nonlinear shapes or patterns. Good quality cut edges can be produced with little risk of distortion to the steel. For cutting straight lines, guillotine shearing is widely used. By using open-ended guillotines, a continuous cut greater in length than the shear blades can be achieved, although at the risk of introducing small steps in the cut edge. Oxyacetylene cutting is not satisfactory for cutting stainless steel, unless a powder fluxing technique is used.

### 12.4.2 Holes

Holes may be drilled, punched or laser cut. In drilling, positive cutting must be maintained to avoid work hardening and this requires sharp bits with correct angles of rake and correct cutting speeds. The use of a round tipped center punch is not recommended as this work hardens the surface. A center drill should be used; if a center punch has to be used, it should be of the triangular pointed type. Punched holes can be made in austenitic stainless steel up to about ¾ in. (20 mm) in thickness. The higher strength of duplex stainless steels leads to a smaller limiting thickness—the minimum diameter of hole that can be punched out is 0.08 in. (2 mm) greater than the sheet thickness.

Tooling can potentially leave a jagged edge and may embed carbon steel particles from the die. Punched samples from the potential supplier should be examined. Post-punching pickling is sometimes used to remove carbon steel contamination when decorative punched hole edges are exposed to a potentially corrosive environment.

## 12.5 WELDING

### 12.5.1 Introduction

The relevant standard for welding stainless steel is AWS D1.6/D1.6M, *Structural Welding Code—Stainless Steel*. AWS D10.18M/D10.18, *Guide for Welding Ferritic/*

*Austenitic Duplex Stainless Steel Piping and Tubing* (AWS, 2008) also contains relevant information. The section below is a brief introduction to welding of stainless steels.

Austenitic stainless steels are readily welded using common processes, provided that suitable filler metals are used. Duplex stainless steels require more temperature control during welding and may require post-weld heat treatment or special welding consumables. *Practical Guidelines for the Fabrication of Duplex Stainless Steels* (IMOA, 2009) gives further information on the welding of duplex stainless steels.

General cleanliness and the absence of contamination are important for attaining good weld quality. Oils or other hydrocarbons, dirt and other debris, strippable plastic film, and wax crayon marks should be removed to avoid their decomposition and the risk of carbon pickup and weld surface contamination. The weld should be free from zinc, including that arising from galvanized products, and from copper and its alloys. (Care needs to be taken when copper backing bars are used; a groove should be provided in the bar immediately adjacent to the fusion area.)

It is important in stainless steel to reduce sites at which crevice corrosion (see Section 2.6.2) may initiate. Welding deficiencies such as undercut, lack of penetration, weld spatter, slag, and arc strikes are all potential sites and should thus be minimized to avoid corrosion. Arc strikes or arcing at loose connections also damage the passive layer, and possibly give rise to crevice corrosion, thereby ruining the appearance of a fabrication.

Where the weld appearance is important, the engineer should specify the as-welded profile and surface condition required. This may influence the welding process selected or the post-weld treatment. Consideration should also be given to the location of the weld; is it possible to apply the appropriate post-weld treatment?

The engineer should be aware that welding distortion is generally greater in stainless steel than in carbon steel (see Section 12.5.4). Heat input and interpass temperatures need to be controlled to minimize distortion and to avoid potential metallurgical problems (see Section 12.5.5).

Welding should be carried out to a general welding procedure specification (WPS) in accordance with AWS D1.6/D1.6M. A WPS contains the following elements:

- Verification of the welding method by detailing the derivation and testing requirements of weld procedures
- The qualifications of welders including proof of current certification for the technique and stainless steel type
- The control of welding operations during preparation, actual welding and post-weld treatment
- The level of inspection and nondestructive testing techniques to be applied
- The acceptance criteria for the permitted level of weld defects

The general WPS should be accompanied by detailed WPS applicable to ranges of essential variables defined in AWS D1.6/D1.6M. AWS D1.6/D1.6M contains provisions for prequalified WPS, as well as qualification provisions for ranges of process variables which are outside the limits for prequalification. To this effect, all welding of duplex steels requires WPS qualification. AWS also publishes standard WPS, including three for austenitic stainless steel in the series of B2.1-8-023, -24 and -25 documents. In accordance with AWS D1.6/D1.6M Clause 3.0, it should be noted that the use of prequalified or standard WPS cannot replace engineering judgment regarding suitability of application to a welded assembly or connection. AWS D1.6/D1.6M allows the engineer to accept WPS and welders' qualifications granted to other standards.

Lock welding of the nut to the bolt should never be allowed, as the materials are formulated for strength and not for fusion welding. Upsetting the bolt threads may be an acceptable alternative in a situation where the nuts are to be locked in place.

### 12.5.2 Processes

As mentioned above, the common fusion methods of welding can be used on stainless steel. Table 12-1 shows the suitability of various processes for thickness ranges, etc. *The Welding of Stainless Steels* (Euro Inox, 2007) gives more information.

Preheating of austenitic and duplex stainless steels is not normally performed, except to evaporate any condensation (water) on the surface.

### 12.5.3 Filler Metals

Commercial filler metals have been formulated to give weld deposits of equivalent strength and corrosion resistance to the parent metal and to minimize the risk of solidification cracking. For specialist applications, such as unusually aggressive environments or where nonmagnetic properties are required, the advice of steel producers and manufacturers of filler metals should be sought. All filler metals should conform to the requirements specified in AWS D1.6/D1.6M; they should be kept free from contaminants and stored according to the manufacturer's instructions. Some processes such as GTAW (TIG) or laser welding may not use filler metals.

### 12.5.4 Welding Distortion

In common with other metals, stainless steel suffers from distortion due to welding. The types of distortion (angular, bowing, shrinkage, etc.) are similar in nature to those found in carbon steel structures. However, the distortion of stainless

| Weld Process | Suitable Product Forms | Types of Welded Joint | Material Thickness Range | Weld Positions | Suitable Shop/Site Conditions |
|---|---|---|---|---|---|
| Shielded metal arc welding (SMAW), also called stick electrode welding | All but not sheet | All | $^1/_8$ in. (3 mm)[a] or greater | All | All |
| Gas metal arc welding (GMAW), also called metal inert gas (MIG) welding | All | All | 0.08 in. (2 mm)[a] or greater | All | All[b] |
| Gas tungsten arc welding (GTAW), also called tungsten inert gas (TIG) welding | All | All | Up to approx. $^3/_8$ in. (10 mm) | All | All[b] |
| Submerged arc welding (SAW) | All but not sheet | All | ¼ in. (6 mm)[a] or greater | Flat | All |
| Resistance welding | Sheet only | All | Up to approx. $^1/_8$ in. (3 mm) | All | All |
| Laser beam welding (LBW) | All | All | Depending on the section, up to 1 in. (25 mm) may be possible | All | Shop only |
| Flux cored arc welding (FCAW) | All | All | 0.08 in. (2 mm)[a] or greater | All | All |

**Table 12-1. Welding Processes and Their Suitability**

[a] Depends upon type of weld joint used
[b] More sensitive to weather than other processes and better environmental protection is required

steel, particularly of austenitic stainless steels, is greater than that of carbon steel because of higher coefficients of thermal expansion and lower thermal conductivities (which lead to steeper temperature gradients) (see Section 2.4). Duplexes have less movement during welding than austenitic stainless steels.

Welding distortion can only be controlled, not eliminated, by taking the following measures:

- Remove the necessity to weld, e.g., specify, if available, hot-rolled sections to ASTM A276 (ASTM, 2010a) and round or rectangular HSS, or laser fused sections to ASTM A1069/A1069M (ASTM, 2011b) (laser fusing results in less distortion).
- Reduce the extent of welding.
- Reduce the cross section of welds. For instance in thick sections, specify double V, U or double U preparations in preference to single V.
- Use symmetrical joints.
- Design to accommodate wider dimensional tolerances.

Measures the fabricator can take:

- Use efficient clamping jigs. If possible the jig should incorporate copper or aluminum bars to help conduct heat away from the weld area.
- When efficient jigging is not possible, use closely spaced tack welds laid in a balanced sequence.
- Ensure that good fit-up and alignment are obtained prior to welding.
- Use the lowest heat input commensurate with the selected weld process, material and thickness.
- Use balanced welding and appropriate sequences (e.g., backstepping and block sequences).
- Specify hot-rolled sections to ASTM A276 or laser fused sections to ASTM A1069/A1069M.

### 12.5.5 Metallurgical Considerations

It is beyond the scope and intent of this Design Guide to cover the metallurgy of stainless steels except for some of the more significant factors.

*Formation of Precipitates in the Austenitic Types*

In the austenitic steels, the heat affected zone is relatively tolerant to grain growth and to the precipitation of brittle and intermetallic phases. Welding procedures are usually designed to control the time spent in the critical temperature range for precipitation effects, i.e., 840 to 1,650 °F (450 to 900 °C). Excessive weld repair naturally increases the time spent and is thus usually restricted to three major repairs.

The formation of chromium carbide precipitates and the ensuing loss of corrosion resistance is discussed in Section 2.6.2 where it is noted that this is not normally a problem with the low carbon austenitic stainless steels (i.e., Types S30403 and S31603). However, weld decay effects may be manifested in the standard carbon Types S30400 and S31600 in welded construction.

*Solidification Cracking in the Austenitic Stainless Steels*

Solidification cracking of welds is avoided when the weld structure contains approximately 5% ferrite. Steelmakers balance the composition and heat treatment of the common austenitic stainless steels to ensure that they contain virtually no ferrite when delivered but will form sufficient ferrite in an autogenous weld (i.e., a weld with no filler added). Even so, to reduce any likelihood of cracking, it is prudent to minimize heat inputs, interpass temperatures and restraint when making autogenous welds. In thicker materials, filler metal is added and the use of good quality filler metals again ensures the appropriate amount of ferrite is formed. It is not normally necessary to measure the precise amount of ferrite formed; appropriate weld procedures (confirmed by applicable qualification procedures and testing) and filler metals will ensure that solidification cracking will not occur.

*Embrittlement of Duplex Stainless Steels*

Duplex steels are sensitive to 890 °F (475 °C) and σ-phase embrittlement. 890 °F (475 °C) embrittlement occurs when the steel is held within or cooled slowly through the approximate temperature range 1,020 to 750 °F (550 to 400 °C) and this produces an increase in tensile strength and hardness with a decrease in tensile ductility and impact strength. σ-phase embrittlement might occur after a long exposure at a temperature in the range 1,050 to 1,650 °F (565 to 900 °C) but can occur in as short as half an hour under appropriate conditions (depending on the composition and the thermomechanical state of the steel). The effects of σ-phase embrittlement are greatest at room temperature or lower. Both forms of embrittlement have an adverse effect on corrosion resistance and toughness. Both forms of embrittlement can be adequately controlled by adopting correct welding procedures; a maximum interpass temperature of 390 °F (200 °C) is suggested. Particular care must be exercised when welding heavy sections. To avoid embrittlement, long-term exposure at temperatures above 572 °F (300 °C) should be avoided. ASTM A923, *Standard Test Methods for Detecting Detrimental Intermetallic Phase in Duplex Austenitic/Ferritic Stainless Steels* (ASTM, 2003), gives test methods for detecting whether embrittlement has occurred and may be specified as part of a weld procedure.

### 12.5.6 Post-Weld Treatment

Post-weld heat treatment of austenitic and duplex stainless steel welds is rarely done outside a producing mill environment. In certain circumstances, a stress relief heat treatment may be required. However, any heat treatment may involve risk and specialist advice should be sought.

Post-weld finishing is generally necessary, as discussed in the following paragraphs, especially if arc welding processes are involved. It is important to define the required post-weld treatment for avoiding cost overruns and possible poor service performance. Finishing techniques common to all fabrications are covered in Section 12.8.

The processes usually employed for weld dressing are wire brushing and grinding. The amount of dressing should be minimized by the fabricator and, if possible, limited to wire brushing. This is because the heat produced in grinding can affect the corrosion resistance. Note that wire brushes should be made of compatible stainless steel (see Section 12.3). Intense brushing of welds may lead to incrustation of surface contaminants, which may cause corrosion. As an alternative, soft abrasives such as 3M Scotch Brite disks may be considered. They lose their particles in the process and do not recontaminate the cleaned area.

It is good practice to remove all traces of heat tint. However, yellow heat tint may prove satisfactory when the stainless steel offers a good margin of resistance for the particular environment. Where this is not so, or where the tint is not acceptable on aesthetic grounds, it may be removed by pickling or glass bead blasting. Pickling may be carried out by immersion in a bath (see Section 12.8) or by using pastes in accordance with the manufacturer's instructions.

Peening the surface of a weld is a beneficial post-weld treatment. It introduces compressive stresses into the surface, which improves fatigue and stress corrosion cracking resistance and aesthetic appearances. However, peening cannot be used to justify a change in fatigue assessment.

The action of removing metal during substantial machining gives rise to stress relieving and hence distortion of the as-welded product. In those cases where the distortion is such that dimensional tolerances cannot be achieved, a thermal stress relief may be required.

### 12.5.7 Inspection of Welds

Inspection should be carried out by AWS certified weld inspectors, duly experienced in welding stainless steel (see also Section 6.7 of AWS D1.6/D1.6M). Table 12-2 compares the examination methods commonly used on stainless steel welds and on carbon steel welds. The methods are used as necessary depending on the degree of structural and corrosion integrity required for the environment under consideration. However, visual inspection should be carried out

| Table 12-2. Examination Methods for Welds | | | |
|---|---|---|---|
| **Examination** | **Austenitic Stainless Steel** | **Duplex Stainless Steel** | **Carbon Steel** |
| Surface | Visual<br>Dye Penetrant | Visual<br>Dye Penetrant<br>Magnetic Particle | Visual<br>Dye Penetrant<br>Magnetic Particle |
| Volumetric | Radiographic<br>(X-ray, Gamma) | Radiographic<br>(X-ray, Gamma) | Radiographic<br>(X-ray, Gamma)<br>Ultrasonic |

during all stages of welding as it can prevent many problems from becoming troublesome as fabrication continues. Surface examination of stainless steel is more important than that of carbon steel, since stainless steel is primarily used to combat corrosion and even a small surface flaw can render the material susceptible to corrosion attack.

Magnetic particle inspection is not an option for the austenitic steels because these are not magnetic. Ultrasonic methods are of limited use on welds because of difficulties in interpretation; however, they can be used on parent material. Gamma radiography is not suitable for detecting cracking or lack of fusion in stainless steel materials less than $3/8$ in. (10 mm) thick.

## 12.6 INSTALLING STAINLESS STEEL BOLTS

Installation methods for stainless steel fasteners are not standardized. As with high-strength bolts, proper joint fit-up that does not induce bending into the bolts, selection of proper bolt length to allow full nut engagement, and rules related to the use of washers must be adhered to.

## 12.7 GALLING AND SEIZURE

If surfaces are under load and in relative motion, fastener thread galling or cold welding can occur due to local adhesion and rupture of the surfaces with stainless steel, aluminum, titanium and other alloys which self-generate a protective oxide surface film for corrosion protection. In applications where disassembly will not occur and any loosening of fasteners is structurally undesirable, it may be an advantage.

In applications where easy fastener removal for repairs is important, galling should be avoided. Several precautions can be taken to avoid this problem with stainless steel:

- Slow down the installation RPM speed.
- Lubricate the internal or external threads with products containing molybdenum disulfide, mica, graphite or talc, or a suitable proprietary pressure wax (but care should be taken to evaluate the suitability of a commercial anti-galling dressing for the application in question).
- Use different stainless steel types with different hardness levels for the bolt and nut.
- Make sure that the threads are as smooth as possible.

## 12.8 FINISHING

The surface finish of stainless steel is an important design criterion and should be clearly specified according to architectural or functional requirements. Finer finishes are more expensive. This is where precautions taken earlier in handling and welding will pay off. Initial planning is important in reducing costs. For instance, if a tube-to-tube weld in a handrail or balustrade is hidden inside a supporting member, there will be a reduced finishing cost and a significant improvement in the final appearance of the handrail. When polishing, grinding, or finishes other than mill or abrasive blasting are specified, it is generally most cost effective for polishing houses to apply those finishes prior to fabrication. For example, hot-formed angles and channels, tube, pipe, and plate can be polished before they are welded or otherwise connected to other components.

The surface of the steel should be restored to its corrosion-resisting condition by removing all scale and contamination. Pickling in an acid bath will loosen any scale, enabling it to be brushed off with a bristle brush, but it may change the appearance of the finish to a more matte or dull finish. Pickling will also dissolve any embedded iron or carbon steel particles, which, if not removed, can show up as rust spots on the stainless steel surface. Abrasive treatments, such as grinding, finishing, polishing and buffing, produce unidirectional finishes and thus the blending of welds may not be easy on plates/sheets with normal rolled surfaces. A degree of experimentation may be required to determine detailed procedures to obtain a suitable finish. Laser welding is generally preferred for welded aesthetic structural components because the joint is less visible. Electropolishing produces a bright shiny surface similar to a highly buffed surface finish. It removes a thin layer of metal along with any light surface oxides. Heavy oxides must be removed by pickling or grinding to insure a uniform appearance after electropolishing. When component size permits, the electropolishing is carried out by immersion in a tank containing an electrolyte and electrical connections. Handheld units can be used to selectively remove heat tint from the weld zone or polish selective areas. There are other finishing processes (electroplating, tumbling, etching, coloring and surface blackening) but these would only rarely be used for structural stainless steel and so are not described here.

It is worth noting again that the surface should be free of contaminants in the assembled structure. Particular consideration should be given to the possibility of contamination arising from work on adjacent carbon steelwork, especially from grinding dust or sparks from abrasive cutting. Either the stainless steel should be protected by removable plastic film or another barrier, or final cleaning after completion of the structure should be specified in the contract documents.

# Chapter 13
# Testing

## 13.1 GENERAL

Testing of stainless steel materials and members may be required for a number of reasons:

- If advantage is to be taken of the strength enhancement of cold-formed corners in members
- If the geometry of a member is such that it lies outside applicable limits
- If a number of structures or components are to be based on prototype testing
- If confirmation of consistency of production is required
- For qualifying welding procedure specifications (WPS)

The usual precautions and requirements for test procedures and results evaluation appertaining to carbon steel testing also apply to stainless steel testing. However, there are particular aspects of the behavior of stainless steels that need to be given more thought in the design of the tests than perhaps would be the case for carbon steels. The following brief guidance is offered.

## 13.2 STRESS-STRAIN CURVE DETERMINATION

When carrying out tensile tests on stainless test coupons, it is recommended that loading be accomplished by pins passing through the ends of the coupon that are of sufficient area to sustain the shear. This is to ensure the coupon is axially loaded, thus enabling the actual shape of the stress-strain curve to be discerned without any spurious effect caused by premature yielding due to load eccentricity. Axiallity of loading may be confirmed by elastic tests with an extensometer placed at various orientations about the specimen. Because stainless steel exhibits a degree of anisotropy (different stress-strain characteristics parallel and transverse to the rolling directions), with higher strengths transverse to the rolling direction, it is recommended that due consideration is given to the orientation of the test specimens. Stainless steels have a strong strain rate dependency; for verification of tensile properties, the same strain rate as was used for establishing the mill certificate is recommended.

## 13.3 TESTS ON MEMBERS

It is recommended that member tests be full scale or as near to full scale as possible, depending on test facilities, and that the specimens be manufactured by the same fabrication processes to be used in the final structure. If the components are welded, the prototype should be welded in the same way.

Due to anisotropy, it is recommended that the specimens are prepared from the plate or sheet in the same orientation (i.e., transverse or parallel to the rolling direction) as intended for the final structure. If the final orientation is unknown or cannot be guaranteed, it may be necessary to conduct tests for both orientations and take the less favorable set of results. For work hardened materials, both the tensile and compressive strength should be determined in the direction in question. Evaluation of the test results should be carried out with the relevant strength as reference.

Stainless steel displays higher ductility and greater strain hardening than carbon steel and therefore the test rig capabilities may need to be greater than those required for testing carbon steel members of equivalent material yield strength. This not only applies to rig loading capacity but also to the ability of the rig to allow greater deformation of the specimen. It should also be noted that at higher specimen loads, the effects of creep become more manifest and this may mean that strain or displacement readings do not stabilize within a reasonable time.

# Appendix A
# The Continuous Strength Method

## A.1 GENERAL

The continuous strength method (CSM) is a deformation based design approach for determining the strength of members of low slenderness incorporating the benefits of strain hardening. This Appendix applies only to the compression and flexural strength of I-shaped members and rectangular hollow structural sections (HSS) with a cross-section slenderness for plate buckling, $\bar{\lambda}_p \le 0.68$, where $\bar{\lambda}_p$ is defined in Section A.3. The strength of members for compression and flexure at the limit state of buckling should be determined in accordance with Sections 5 and 6.

The strain hardening material model is specified in Section A.2, and Section A.3 gives expressions for determining the deformation capacity of the section. Sections A.4 and A.5 give expressions for determining the compression and flexural strength. This Appendix does not apply to round HSS. This Appendix applies only to static design.

## A.2 MATERIAL MODELLING

The elastic, linear hardening material model used with the CSM is shown in Figure A-1, where

$F_y$ = specified minimum yield stress, ksi (MPa), taken from Table 2-2
$E$ = initial modulus of elasticity, ksi (MPa), taken from Table 2-9
$\varepsilon_y$ = yield strain, taken as $\varepsilon_y = F_y/E$
$E_{sh}$ = strain hardening modulus, ksi (MPa)

The strain hardening modulus, $E_{sh}$, may be determined from:

$$E_{sh} = \frac{F_u - F_y}{0.16\varepsilon_u - \varepsilon_y} \tag{A-1}$$

where

$F_u$ = specified minimum tensile strength, ksi (MPa), taken from Table 2-2
$\varepsilon_u$ = strain at the ultimate tensile stress, taken as $\varepsilon_u = 1 - F_y/F_u$

## A.3 DEFORMATION CAPACITY

For sections where $\bar{\lambda}_p \le 0.68$, the normalized deformation capacity, $\varepsilon_{csm}/\varepsilon_y$, is determined as follows:

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\bar{\lambda}_p^{\,3.6}} \text{ but } \frac{\varepsilon_{csm}}{\varepsilon_y} \le \text{minimum}\left(15, \frac{0.10\varepsilon_u}{\varepsilon_y}\right) \tag{A-2}$$

where

$\varepsilon_{csm}$ = cross-section failure strain

The CSM does not apply for sections where $\bar{\lambda}_p > 0.68$.

The cross-section slenderness, $\bar{\lambda}_p$, may be determined from the elastic buckling stress of the full cross section under the applied loading conditions:

$$\bar{\lambda}_p = \sqrt{\frac{F_y}{\sigma_{e,s}}} \tag{A-3}$$

where

$\sigma_{e,s}$ = full cross-section elastic critical buckling stress, ksi (MPa)

or, conservatively, on the basis of the most slender constituent plate element, it may be defined as:

$$\bar{\lambda}_p = \sqrt{\frac{F_y}{\sigma_e}} = \left[\frac{b}{t}\sqrt{\frac{F_y}{E}\frac{12\left(1-\nu^2\right)}{\pi^2 k}}\right] \tag{A-4}$$

where

$\sigma_e$ = plate element elastic critical buckling stress, ksi (MPa)
$b$ = plate element flat width, in. (mm)
$t$ = plate element thickness, in. (mm)
$\nu$ = Poisson's ratio = 0.3
$k$ = plate buckling coefficient characteristic of the plate stress distribution and plate edge restraint

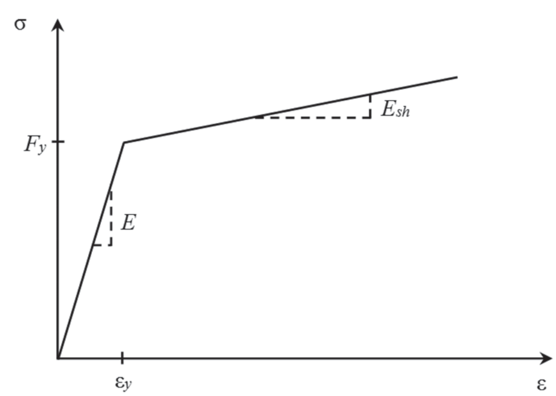



*Fig. A-1. Elastic, linear hardening material model.*

= 0.425 for unstiffened compression elements
= 4.00 for stiffened compression elements
= 23.9 for stiffened elements subject to flexure

## A.4 COMPRESSIVE STRENGTH

For sections where $\bar{\lambda}_p \leq 0.68$ and $\dfrac{KL}{r} \leq 0.63\sqrt{\dfrac{E}{F_y}}$ or $\dfrac{F_y}{F_e} \leq 0.04$, the design compressive strength, $\phi_c P_{n,csm}$, and the allowable compressive strength, $P_{n,csm}/\Omega_c$, should be determined as follows:

The nominal compressive strength at the limit state of yielding, $P_{n,csm}$, is given by:

$$P_{n,csm} = F_{csm} A_g \tag{A-5}$$

where

$A_g$ = gross area of member, in.$^2$ (mm$^2$)

$F_{csm}$ = stress corresponding to $\varepsilon_{csm}$

$$= F_y + E_{sh}\,\varepsilon_y\left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) \tag{A-6}$$

$K$, $L$ and $r$ are defined in Section 5.2, $F_e$ is defined in Section 5.3, and $\phi_c$ and $\Omega_c$ are given in Section 5.1.

## A.5 FLEXURAL STRENGTH

For sections where $\bar{\lambda}_p \leq 0.68$ and $L_b \leq 0.75L_p$, the design flexural strength, $\phi_b M_{n,csm}$, and the allowable flexural strength, $M_{n,csm}/\Omega_b$, should be determined as follows:

The nominal flexural strength at the limit state of yielding, $M_{n,csm}$, is given by:

(a) Major axis bending

$$M_{n,csm,x} = M_{p,x}\left(1 + \frac{E_{sh}}{E}\frac{S_x}{Z_x}\left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) - \left(1 - \frac{S_x}{Z_x}\right)\Big/\left(\frac{\varepsilon_{csm}}{\varepsilon_y}\right)^2\right) \tag{A-7}$$

(b) Minor axis bending

$$M_{n,csm,y} = M_{p,y}\left(1 + \frac{E_{sh}}{E}\frac{S_y}{Z_y}\left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) - \left(1 - \frac{S_y}{Z_y}\right)\Big/\left(\frac{\varepsilon_{csm}}{\varepsilon_y}\right)^\alpha\right) \tag{A-8}$$

where

$M_p$ = plastic bending moment, kip-in. (N-mm)

$Z$ = plastic section modulus about axis of bending, in.$^3$ (mm$^3$)

$S$ = elastic section modulus about axis of bending, in.$^3$ (mm$^3$)

$\alpha$ = 2.0 for rectangular HSS or 1.2 for I-shaped sections

$L_b$ and $L_p$ are defined in Section 6.2 and $\phi_b$ and $\Omega_b$ are given in Section 6.1.

# Appendix B
# Commentary to the Design Provisions

## B.1 INTRODUCTION

### B.1.1 Purpose of the Commentary

This Appendix describes the work undertaken to derive the design provisions in this Design Guide. This Appendix will also facilitate the development of revisions to the design rules as, and when, new data become available.

### B.1.2 How Does the Structural Performance of Stainless Steel Differ from Carbon Steel?

The structural performance of stainless steel differs from that of carbon steel because stainless steel has no definite yield point, shows an early departure from linear elastic behavior, and exhibits pronounced strain hardening. This impacts design rules in the following ways:

- The design strength is based on the 0.2% offset yield strength.
- There is a different buckling response for members subject to compression, unrestrained bending and shear buckling (also different levels of residual stresses for welded members).
- Greater deflections will occur in beams at high strains (the secant modulus is generally used for estimating these deflections).
- Different rules for the bearing strength of connections are necessary in order to limit deformation.

### B.1.3 Design Specifications for Structural Stainless Steel

Specifications for the design of cold-formed structural stainless steel are available in the U.S. (ASCE, 2002), Australia/New Zealand (AS-NZS, 2001), South Africa (SABS, 1997), and Japan (SSBA, 2005). However, there are only European (CEN, 2006a) and Japanese (SSBA, 1995) specifications which cover the design of structural sections made from thicker walled material (welded, hot rolled, structural hollow sections). The Japanese specification is not available in English. A comparison of the various structural design standards for stainless steel is made in Baddoo (2003).

*Eurocode 3: Design of Steel Structures, Supplementary Rules for Stainless Steels,* Part 1-4 (EN 1993-1-4) (CEN, 2006a) gives rules which can be applied to welded, hot-rolled and cold-formed stainless steel members. It is a supplement rather than a standalone document, referring extensively to the following parts of Eurocode 3:

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 1993-1-5 *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*

The design rules in the 1996 public draft of EN 1993-1-4 were initially based on the first edition of the European *Design Manual for Structural Stainless Steel* (Euro Inox and SCI, 1994), following a European joint industry project. The rules were derived on the basis of an extensive test program and took into account all known work carried out in Europe, U.S., South Africa and Australia. The *Design Manual* included a commentary which explains the basis of the development of the design rules and presents the results of the relevant test programs. Since 1994, the European *Design Manual* has been revised and extended two times, taking into account the results of further European research projects and new work from other parts of the world. EN 1993-1-4 (CEN, 2006a) aligns with the recommendations in the current third edition of the European *Design Manual* (Euro Inox and SCI, 2006a), except in the area of fire resistance where the rules in the *Design Manual* are less conservative.

### B.1.4 Scope of the Design Guide

The intention at the start of writing this Design Guide was to modify the structural stainless steel rules in EN 1993-1-4 and present them in a format aligned with the AISC *Specification for Structural Steel Buildings*. However, due to the fundamental differences between the design rules in the AISC *Specification* and EN 1993-1-4, this approach was not possible and the following procedure was implemented:

1. Compare the rules for carbon steel and stainless steel in Eurocode 3.
2. Compare the rules for carbon steel in the AISC

*Specification* against all available stainless steel test data on members and connections.

3. Modify the AISC *Specification* carbon steel rules to suit the stainless steel data where necessary.

4. Calculate the stainless steel resistance factors to use with the recommended stainless steel design rules.

An Evolution Group has been established for each part of Eurocode 3 to oversee maintenance and future development activities and it is expected that revisions to all parts of Eurocode 3 will be issued in the future. The Evolution Group for EN 1993-1-4 is considering a number of developments to the standard, most of which will lead to less conservative design rules due to the far greater body of test data which is now available for structural stainless steel. These proposed developments have been taken into account during the preparation of this Design Guide.

The Design Guide gives guidance that a designer familiar with designing to the AISC *Specification* should be able to use easily. Where stainless steel behaves in a similar way to carbon steel, the Design Guide simply refers to the relevant section in the AISC *Specification*. Where the guidance in the AISC *Specification* would be unconservative or unduly conservative when applied to stainless steel, specific rules for stainless steel have been presented in a format as close as possible to the equivalent expressions in the AISC *Specification* for carbon steel.

The assumptions made and data used in order to calculate the resistance factors by means of a reliability analysis are described in Section B.2. Sections B.3 to B.11 deal with different aspects of structural design. In each section the design provisions in Eurocode 3 for both carbon steel and stainless steel are presented and compared to the provisions for carbon steel in the AISC *Specification*. Stainless steel data are then compared to the AISC provisions and new provisions for stainless steel presented where necessary. The results of the reliability analysis are then given.

It should be noted that the Design Guide is applicable to hot-rolled materials. Structural design of cold-formed stainless steels (including cold-worked austenitic stainless steels) are covered by ASCE/SEI 8-02, *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2002). See also Section B.1.3.

## B.2 DETERMINATION OF STAINLESS STEEL RESISTANCE FACTORS

### B.2.1 Probabilistic Basis and Reliability Index

Structural safety is a function of the resistance, $R$, of the structure as well as the load effects, $Q$. It is assumed that the resistance and the load effects are random variables because of the uncertainties associated with their inherent randomness. Based on the assumed probability distributions and first-order probabilistic theory, the reliability index, $\beta$, can be expressed as:

$$\beta = \frac{\ln\left(R_m/Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \qquad (\textit{Spec}.\ \text{Eq. C-B3-2})$$

where

$Q_m$ = mean value of the load effect
$R_m$ = mean value of the resistance
$V_Q$ = coefficient of variation of the load effect, $Q$ (i.e., standard deviation divided by the mean)
$V_R$ = coefficient of variation of the resistance, $R$ (i.e., standard deviation divided by the mean)

In accordance with the assumptions made in the development of the LRFD approach for hot-rolled steel structures in the AISC *Specification*, a target reliability index has been set for members of $\beta = 2.6$ and for connections of $\beta = 4.0$ (Bartlett et al., 2003).

### B.2.2 Load and Load Effects

A dead load factor of 1.2 and a live load factor of 1.6 for the basic combination of dead plus live load were assumed in the stainless steel reliability analysis. The analyses were carried out for a dead-to-live load ratio of 1:5 and 1:3. For all modes of loading, the load ratio 1:5 gave slightly more severe results, however, in accordance with the assumptions taken for the reliability analysis carried out for the AISC *Specification*, the values for a dead-to-live load ratio of 1:3 are considered more applicable for hot-rolled and welded structural sections (Bartlett et al., 2003) and were thus used to calculate the resistance factors.

$Q_m$ and $V_Q$ were calculated from the following equations given in Ellingwood et al. (1980). These expressions were also used by Lin et al. (1998):

$$Q_m = c(D_m + L_m) \qquad \text{(B-1)}$$

$$V_Q = \frac{\sqrt{\left(D_m V_D\right)^2 + \left(L_m V_L\right)^2}}{\left(D_m + L_m\right)} \qquad \text{(B-2)}$$

where

$c$ = influence coefficient which transfers load intensities to load effects

The following values for the parameters were adopted: $D_m = 1.05D_n$, $V_D = 0.1$, $L_m/L = 1.0$, and $V_L = 0.25$.

The subscripts $m$, $n$, $D$ and $L$ refer to mean, nominal, dead and live respectively. Assuming a dead load-to-live load ratio of 1:3 gives $V_Q = 0.19$ and $Q_m = 1.33cL_m$.

### B.2.3 Resistance

The randomness of the resistance, $R$, of a structural element is due to the variability inherent in the mechanical properties of the material, variations in dimensions, and the uncertainties in the design theory used to express the member strength. The mean resistance of a structural member, $R_m$, is defined as follows:

$$R_m = R_n(M_m)(F_m)(P_m) \quad \text{(B-3)}$$

where

$R_n$ = nominal resistance of the structural elements

$M_m$, $F_m$, $P_m$ = mean values of the random variables reflecting the uncertainties in material properties (i.e., $F_y$, $F_u$, etc.), the geometry of the cross section (i.e., $A$, $t$, $L$, etc.), and the design assumptions, respectively

$M$, known as the material factor, is taken as the ratio of the actual measured value of a mechanical property to the minimum specified value of that property given in the relevant ASTM specification. Similarly, $F$, known as the fabrication factor, is taken as the ratio of the actual measured value of that geometrical property to the nominal value of that property. $P$, known as the professional factor, is taken as the ratio of the measured failure load to the failure mode predicted from the design provision.

The coefficient of variation of the resistance, $V_R$, is calculated as the square-root-sum-of-squares of the material, fabrication and design model uncertainty coefficients of variation:

$$V_R = \sqrt{V_m^2 + V_f^2 + V_p^2} \quad \text{(B-4)}$$

#### B.2.3.1 Material Factor, $M_m$

Data on the statistical variation of material strengths were collected from literature (Groth and Johansson, 1990; Leffler, 1990; Outokumpu, 2006a; Outokumpu, 2006b; Outokumpu, 2008). Steel producers and manufacturers of stainless steel sections also supplied more recent data for this analysis. Much of their data was supplied on a confidential basis, so it is not possible to give a detailed breakdown of the material data herein.

The data analyzed demonstrated values of $M_m > 1.3$ for austenitic stainless steel and $M_m > 1.1$ for duplex stainless steel for the 0.2% offset yield overstrength ratio. Duplex stainless steels were introduced into standards in the 1970s and 1980s, so the minimum specified values are based on modern steelmaking technology and the gap between the actual and minimum specified values is less than that for austenitics. It is important to note that load-bearing duplex stainless steel represents only approximately 1 to 3% of the total tonnage of structural stainless steel, with austenitic stainless steels making up the balance. In order to best utilize the greater conservatism in the assessment of 0.2% offset yield strength for austenitic stainless steel, it was decided to analyze austenitic and duplex stainless steel as separate populations. For austenitic stainless steel, the material factor, $M_m$, is taken as 1.3, while for duplex stainless steel, $M_m$ is taken as 1.1.

The choice of $M_m = 1.1$ for the 0.2% offset yield strength in the cold-formed stainless steel specification, ASCE/SEI 8-02, is perhaps surprisingly low. However, the analysis of material data carried out in order to select a value of $M_m$ for this specification, showed that the cold-worked Types S30100 and S20100 (cold-worked tempers of ¼ hard and ½ hard) and Types S40900, S43000 and S43900 all demonstrated considerably lower values of $M_m$ than hot-rolled Type S30400 stainless steel (Lin et al., 1998). As these types of stainless steel are not included in this Design Guide, there is no need to retain this value of $M_m = 1.1$.

Note that, nowadays, no significant difference is expected between the strengths of standard (e.g., S30400) and low carbon (e.g., S30403) types. Steelmakers generally produce material that fulfills both standard and L specifications, as only the maximum carbon content is specified. The low specified minimum yield stress values in ASTM A240 for Type S30403/S31603 (170 MPa compared to 205 MPa for standard types) are historical and are not representative of today's practice. As the smaller specified minimum yield stress will lead to artificially high $M_m$ values, it was decided not to include the data for the L types in this assessment.

The coefficient of variation, $V_m$, was also calculated from the body of material data collected for this project and a value of 0.105 was taken as representative for both austenitic and duplex stainless steel populations. Parametric studies showed that the value of the resistance factor, $\phi$, strongly correlates with the overstrength ratio, $M_m$, whereas variations in $V_m$ only lead to small changes in $\phi$. The choice of coefficient of variation is therefore less significant than the choice of a conservative $M_m$ factor. The material data indicated a value of $M_m = 1.1$ and $V_m = 0.05$ was applicable to the ultimate tensile strength.

Table B-1 shows the values for $M_m$ that have been assumed in AISC and ASCE specifications for hot-rolled and cold-formed carbon steel and stainless steel. The values for the coefficient of variation are given in brackets ($V_m$). The values assumed for this Design Guide are also given for comparison.

#### B.2.3.2 Fabrication Factor, $F_m$

This factor takes into account uncertainties caused by initial imperfections, tolerances and variations in geometric properties. It also reflects the differences between the designed and manufactured cross-sectional dimensions. No data was collected in this study. It was assumed that the values used in the cold-formed stainless steel specification, ASCE/SEI 8-02,

| Table B-1. Reliability and Random Variable Factors for U.S. Steel Design Standards and Eurocode 3 (EN 1993-1-1 and -4) | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | AISC 360-10 (AISC, 2010c) | AISI *Cold-Formed Specification* (AISI, 2007) | ASCE/SEI 8-02 (ASCE, 2002) | AISC Design Guide on Stainless Steel | EN 1993-1-1 (CEN, 2005a) | EN 1993-1-4 (CEN, 2006a) |
| | | Carbon Steel | Carbon Steel | Stainless Steel | Stainless Steel | Carbon Steel | Stainless Steel |
| | | Hot-Rolled/ Welded | Cold-Formed | Cold-Formed | Hot-Rolled/ Welded | Hot-Rolled/ Welded | Hot-Rolled/ Welded and Cold-Formed |
| β Reliability index | Members | 2.60 | 2.50 | 3.00 | 2.60 | 3.80 | 3.80 |
| | Connections | 4.00 | 3.50 | 4.00 | 4.00 | 3.80 | 3.80 |
| Material random variable | $M_m(V_m)$ | 1.028 (0.058) | 1.10 (0.10) | $F_y$ : 1.10 (0.10)<br>$F_u$ : 1.10 (0.05) | Austenitic:<br>$F_y$ : 1.3 (0.105)<br>$F_u$ : 1.1 (0.105)<br>Duplex:<br>$F_y$ : 1.1 (0.105)<br>$F_u$ : 1.1 (0.105) | N/A | N/A |
| Geometry random variable | $F_m(V_f)$ | Members:<br>1.00 (0.05)<br>Bolted conns:<br>1.00 (0.05)<br>Welded conns:<br>1.00 (0.15) | 1.00 (0.05) | Members:<br>1.00 (0.05)<br>Bolted conns:<br>1.00 (0.05)<br>Welded conns:<br>1.00 (0.15) | Members:<br>1.00 (0.05)<br>Bolted conns:<br>1.00 (0.05)<br>Welded conns:<br>1.00 (0.15) | N/A | N/A |

The shading indicates carbon steel factors; no shading indicates stainless steel factors.

which were the same as those used in the development of the AISC LRFD criteria for hot-rolled structural steel members, apply. The following values are assumed:

For stainless steel members and bolted connections, $F_m$ = 1.00 and $V_f$ = 0.05

For welded connections, $F_m$ = 1.00 and $V_f$ = 0.15

These values are also shown in Table B-1.

### B.2.3.3 Professional Factor, $P_m$

The professional factor depends on the failure mode in question, and is defined for each specific case in Sections B.3 to B.11. Note that there are no test data for hot-rolled austenitic stainless steel structural sections (a few tests have been carried out on hot-rolled ferritic stainless steel sections). The test data used to assess the professional factor were data on hollow structural sections (HSS) (austenitic and duplex) and welded I-shaped members. As a general rule, it is expected that hot-rolled sections will perform better than welded sections because of the absence of residual stresses developed during welding. In some cases, data on cold-formed stainless steel sections were also considered.

Table B-2 and Table B-3 show the values for random variables, $P_m$ and $V_p$, for austenitic and duplex stainless steels, respectively, which were calculated in this reliability analysis from an assessment of the stainless steel data against the recommended design models.

### B.2.4 Determination of Resistance Factor

Following the assumptions and procedures described in Lin et al. (1992) and Bartlett et al. (2003), the resistance factor was calculated from:

$$\phi = \frac{1.481 M_m F_m P_m}{\exp\left(\beta\sqrt{V_R^2 + V_Q^2}\right)} \tag{B-5}$$

Using all of the assumptions discussed in the previous section, values of the resistance factor, ϕ, were derived for each expression in this Design Guide and these are presented in Table B-2 and Table B-3.

In general, the reliability analysis shows that the carbon steel resistance factors can be safely used with the AISC stainless steel design curves with the following two exceptions:

- Round HSS in compression ($\phi_{stainless\ steel} = 0.85$, $\phi_{carbon\ steel} = 0.90$)

- Fillet welds ($\phi_{aust\ stainless\ steel} = 0.55$, $\phi_{duplex\ stainless\ steel} = 0.60$, $\phi_{carbon\ steel} = 0.75$)

The safety factor, Ω, for use in allowable strength designs, was calculated in accordance with Duncan et al. (2006).

Note that Eurocode 3 defines only three partial safety factors for resistance:

- Resistance of cross sections to excessive yielding, including local buckling, $\gamma_{M0}$
- Resistance of members to instability assessed by member checks, $\gamma_{M1}$
- Resistance of cross sections in tension to fracture, $\gamma_{M2}$
- Resistance of bolts, rivets, welds, pins and plates in bearing, $\gamma_{M2}$

The recommended values of these factors for stainless steel are $\gamma_{M0} = \gamma_{M1} = 1.1$ and $\gamma_{M2} = 1.25$. For carbon steel, the values are $\gamma_{M0} = \gamma_{M1} = 1.0$ and $\gamma_{M2} = 1.25$. Re-evaluation of these factors is now underway in Europe.

### B.2.5 Precipitation Hardening Stainless Steels

Design provisions relating to precipitation hardening stainless steel Type S17400 in this Design Guide are limited to:

- Strength of unthreaded tension rods failing by yielding
- Tension and shear strength of bolts and threaded parts

Insufficient data were available to enable a reliability analysis to be carried out for tension rods and bolts in the same way as for austenitic and duplex stainless steels. Therefore the appropriate resistance factors for austenitic and duplex stainless steel were reduced by 10% for precipitation hardening Type S17400 stainless steel to give an extra margin of safety.

## B.3 SECTION CLASSIFICATION

### B.3.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

Compression elements of cross sections are classified as Class 1, 2 or 3 in Eurocode 3 depending upon their width-to-thickness ratios. Those compression elements that do not meet the criteria for Class 3 are then classified as Class 4 elements. The limiting ratios for stainless steel in EN 1993-1-4 are more conservative than those for carbon steel in EN 1993-1-1. The limiting ratios for Class 3 elements were derived from experimental stainless steel data whereas the limiting ratios for Classes 1 and 2 were derived during the preparation of the first edition of the European *Design Manual for Structural Stainless Steel* in the late 1980s by making reference to other data and applying engineering argument. The process of deriving these ratios is described in the Commentary to the European *Design Manual* (Euro Inox and SCI, 2006b).

### B.3.2 The AISC *Specification* Methodology for Carbon Steel

The AISC *Specification* similarly adopts the concept of section classification. For compression elements used in members subject to flexure the terms are compact, noncompact and slender, while for compression elements used in members subject to compression, the terms are nonslender and slender. The class "compact" effectively covers Class 1 and Class 2 in the Eurocodes. [Note that the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2010b) uses the terms highly and moderately ductile, where the former corresponds to Class 1 in the Eurocode.]

### B.3.3 Recommendations for the AISC Design Guide

Over the last twenty years, considerable further research has been conducted on structural stainless steel. Many additional experimental results on cross-section resistance now exist, including both stub column and bending tests. Analysis of the test data by Gardner and Theofanous (2008) reveals that the current slenderness limits in EN 1993-1-4 for stainless steels are overly conservative and that in many cases harmonization with the equivalent carbon steel limits in EN 1993-1-1 are justified. As it is expected that these new limits proposed in this paper will be adopted in the next revision of EN 1993-1-4, it has been decided to adopt these less onerous limits in this Design Guide.

The section classification limits for carbon steel in the AISC *Specification* are given in Table B-4a and Table B-4b. The limits adopted in this Design Guide (Section 3.3.1) are also shown in this table. In general, these are the limits recommended in Gardner and Theofanous (2008); however, in the cases where the stainless limits were higher than the AISC carbon steel limits (web and flange of HSS in bending, and round HSS in bending), the limits were reduced to match the AISC carbon steel limits.

Note that there are minor differences in the width-to-thickness definitions, e.g., in the AISC *Specification*, half the flange width is used to calculate the flange slenderness whereas in Eurocode 3 only the outstanding portion of the flange, measured from the toe of the fillet, is used.

### B.3.4 Determination of Resistance Factors

Gardner and Theofanous (2008) report that a statistical analysis in accordance with EN 1990 Annex D (CEN, 2002) was

| Table B-2. Summary of Results for Derivation of $\phi$ Factors for AISC Design Guide Expressions—Austenitic Stainless Steel | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Limit State** | **No. Results** | $M_m$[a] | $F_m$ | $P_m$ | $R_m/R_n$ | $V_m$ | $V_f$ | $V_p$ | $V_R$ | **$\phi$ (Calculated)[b]** | **$\phi$ (Recommended)** |
| Round HSS in compression, nonslender | 25 | 1.3 | 1 | 1.043 | 1.356 | 0.105 | 0.05 | 0.154 | 0.193 | 0.998 | 0.85[c] |
| | | | | | | | | | | | |
| Rect. HSS in compression, nonslender | 33 | 1.3 | 1 | 1.388 | 1.805 | 0.105 | 0.05 | 0.210 | 0.240 | 1.211 | 0.90 |
| Welded I-shaped members in compression, nonslender | 12 | 1.3 | 1 | 1.116 | 1.451 | 0.105 | 0.05 | 0.238 | 0.265 | 0.925 | 0.90 |
| **Rect. HSS and welded I-shaped members in compression, nonslender** | 45 | 1.3 | 1 | 1.316 | 1.711 | 0.105 | 0.05 | 0.234 | 0.261 | 1.100 | 0.90 |
| | | | | | | | | | | | |
| Rect. HSS in compression, slender | 23 | 1.3 | 1 | 1.521 | 1.978 | 0.105 | 0.05 | 0.340 | 0.360 | 1.021 | 0.90 |
| I-shaped members in compression, slender | 9 | 1.3 | 1 | 1.136 | 1.594 | 0.105 | 0.05 | 0.164 | 0.201 | 1.071 | 0.90 |
| | | | | | | | | | | | |
| Flexural-torsional buckling | 15 | 1.3 | 1 | 1.261 | 1.639 | 0.105 | 0.05 | 0.268 | 0.292 | 0.985 | 0.90 |
| | | | | | | | | | | | |
| Round HSS in flexure, yielding | 8 | 1.3 | 1 | 1.399 | 1.818 | 0.105 | 0.05 | 0.281 | 0.304 | 1.064 | 0.90 |
| Rect. HSS in flexure, yielding | 37 | 1.3 | 1 | 1.413 | 1.837 | 0.105 | 0.05 | 0.122 | 0.169 | 1.414 | 0.90 |
| I-Shaped members in flexure, yielding | 5 | 1.3 | 1 | 1.137 | 1.478 | 0.105 | 0.05 | 0.033 | 0.121 | 1.227 | 0.90 |
| **All members in flexure, yielding** | 50 | 1.3 | 1 | 1.383 | 1.807 | 0.105 | 0.05 | 0.163 | 0.201 | 1.306 | 0.90 |
| | | | | | | | | | | | |
| Lateral-torsional buckling | 14 | 1.3 | 1 | 1.261 | 1.640 | 0.105 | 0.05 | 0.191 | 0.224 | 1.139 | 0.90 |
| Shear buckling | 15 | 1.3 | 1 | 1.116 | 1.451 | 0.105 | 0.05 | 0.108 | 0.159 | 1.137 | 0.90 |

| Table B-2 (continued). Summary of Results for Derivation of ϕ Factors for AISC Design Guide Expressions—Austenitic Stainless Steel | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Limit State | No. Results | $M_m$[a] | $F_m$ | $P_m$ | $R_m/R_n$ | $V_m$ | $V_f$ | $V_p$ | $V_R$ | ϕ (Calculated)[b] | ϕ (Recommended) |
| Combined flexure & compression | 26 | 1.3 | 1 | 1.570 | 2.041 | 0.105 | 0.05 | 0.341 | 0.360 | 1.052 | 0.90 |
| | | | | | | | | | | | |
| Fillet weld (long.) | 11 | 1.1 | 1 | 0.941 | 1.035 | 0.050 | 0.15 | 0.033 | 0.162 | 0.571 | 0.55 |
| Fillet weld (transverse) | 12 | 1.1 | 1 | 1.141 | 1.255 | 0.050 | 0.15 | 0.048 | 0.165 | 0.685 | 0.60 |
| Groove welds | No Data | — | — | — | — | — | — | — | — | — | 0.60 |
| | | | | | | | | | | | |
| Tension rupture | 8 | 1.1 | 1 | 1.193 | 1.312 | 0.050 | 0.05 | 0.200 | 0.073 | 0.870 | 0.75 |
| Shear bolts | 11 | 1.1 | 1 | 1.076 | 1.184 | 0.050 | 0.05 | 0.050 | 0.086 | 0.769 | 0.75 |
| Bearing bolts | 4 | 1.1 | 1 | 1.451 | 1.596 | 0.050 | 0.05 | 0.072 | 0.101 | 1.076 | 0.75 |
| Tension bolts | 12 | 1.1 | 1 | 1.091 | 1.200 | 0.050 | 0.05 | 0.015 | 0.072 | 0.797 | 0.75 |

[a] $M_m$ = 1.3 for 0.2% offset yield strength and = 1.1 for ultimate tensile strength.
[b] If $M_m$ was assumed to be 1.2 instead of 1.3, the calculated values of ϕ would still lie above the recommended values of ϕ in all cases except for welded I-shape compressive buckling (0.854).
[c] Assumed resistance factor was affected by the presence of outlying test points (see Section B.5.1).

carried out to verify that a partial safety factor, $\gamma_{M0}$, of 1.1 could be used in conjunction with the section classification limits. The reliability analysis carried out for this Design Guide is reported in Section B.5 for members in compression and Section B.6 for members in flexure.

## B.4 DESIGN OF MEMBERS FOR TENSION

The design of tension members in Eurocode 3 (carbon steel and stainless steel) involves comparing the plastic resistance of the gross section, $N_{pl,Rd}$, (with appropriate resistance factors) to the design ultimate resistance of the net section at holes for fasteners, $N_{u,Rd}$, (again, with appropriate resistance factors) and taking the smaller value, where

$$N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}} \text{ and } N_{u,Rd} = \frac{0.9 A_{net} f_u}{\gamma_{M2}} \quad \text{(6.6 and 6.7 of EN 1993-1-1)}$$

The approach in the AISC *Specification* for carbon steel is similar to that given in Eurocode 3 except a shear lag factor, $U$, is introduced into the expression for the ultimate resistance in place of the factor 0.9 in the Eurocode.

The design guidance presented in the AISC *Specification* for carbon steel is adopted unaltered in this Design Guide (Chapter 4).

### B.4.1 Determination of Resistance Factor

For tensile yielding in the gross section, $P_m$ is 1.0 and $V_p$ is 0, as the theory can be assumed to be exactly correct, and it is the fabrication and material variability that cause fluctuations in the result. This gives a resistance factor of 0.98, which justifies the use of the AISC *Specification* carbon steel factor, $\phi_t = 0.90$.

For a discussion of tension rupture failure at the net section, see Section B.9.2.

## B.5 DESIGN OF MEMBERS FOR COMPRESSION

### B.5.1 Flexural Buckling of Members Without Slender Elements

#### B.5.1.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

For design of columns to Eurocode 3, the flexural buckling resistance of compression members, $N_{b,Rd}$, is calculated from:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{(6.47 of EN 1993-1-1)}$$

Where the flexural buckling reduction factor, $\chi$, is given by:

**Table B-3. Summary of Results for Derivation of $\phi$ Factors for AISC Design Guide Expressions—Duplex Stainless Steel**

| Limit State | No. Results | $M_m$ | $F_m$ | $P_m$ | $R_m/R_n$ | $V_m$ | $V_f$ | $V_p$ | $V_R$ | $\phi$ (Calculated) | $\phi$ (Recommended) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Round HSS in compression, nonslender | No Data | — | — | — | — | — | — | — | — | — | 0.85 |
| Rect. HSS in compression, nonslender | No Data | — | — | — | — | — | — | — | — | — | 0.90 |
| Welded I-shaped members in compression, nonslender | 3 | 1.1 | 1 | 1.093 | 1.202 | 0.105 | 0.05 | 0.083 | 0.143 | 0.965 | 0.90 |
| | | | | | | | | | | | |
| Rect. HSS in compression, slender | No Data | — | — | — | — | — | — | — | — | — | 0.90 |
| I-shaped members in compression, slender | 6 | 1.1 | 1 | 1.221 | 1.343 | 0.105 | 0.05 | 0.102 | 1.343 | 1.058 | 0.90 |
| | | | | | | | | | | | |
| Flexural-torsional buckling | No Data | — | — | — | — | — | — | — | — | — | 0.90 |
| | | | | | | | | | | | |
| Round HSS in flexure, yielding | 3 | 1.1 | 1 | 1.314 | 1.445 | 0.105 | 0.05 | 0.011 | 0.117 | 1.207 | 0.90 |
| Rect. HSS in flexure, yielding | 15 | 1.1 | 1 | 1.253 | 1.378 | 0.105 | 0.05 | 0.069 | 0.135 | 1.121 | 0.90 |
| I-shaped members in flexure, yielding | 1 | 1.1 | 1 | 1.262 | 1.389 | 0.105 | 0.05 | 0.000 | 0.116 | 1.160 | 0.90 |
| **All members in flexure, yielding** | 19 | 1.1 | 1 | 1.263 | 1.389 | 0.105 | 0.05 | 0.063 | 0.132 | 1.135 | 0.90 |
| | | | | | | | | | | | |
| Lateral-torsional buckling | 2 | 1.1 | 1 | 1.503 | 1.654 | 0.105 | 0.05 | 0.267 | 0.291 | 0.997 | 0.90 |
| Shear buckling | 4 | 1.1 | 1 | 1.169 | 1.286 | 0.105 | 0.05 | 0.092 | 0.149 | 1.024 | 0.90 |
| Combined flexure & compression | No Data | — | — | — | — | — | — | — | — | — | 0.90 |
| | | | | | | | | | | | |

| Table B-3 (continued). Summary of Results for Derivation of $\phi$ Factors for AISC Design Guide Expressions—Duplex Stainless Steel | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Limit State** | **No. Results** | $M_m$ | $F_m$ | $P_m$ | $R_m/R_n$ | $V_m$ | $V_f$ | $V_p$ | $V_R$ | **$\phi$ (Calcu-lated)** | **$\phi$ (Recom-mended)** |
| Fillet weld (long.) | 11 | 1.1 | 1 | 1.017 | 1.118 | 0.050 | 0.15 | 0.025 | 0.160 | 0.619 | 0.60 |
| Fillet weld (transverse) | 12 | 1.1 | 1 | 1.268 | 1.395 | 0.050 | 0.15 | 0.022 | 0.160 | 0.773 | 0.60 |
| Groove welds | No Data | — | — | — | — | — | — | — | — | — | 0.60 |
| | | | | | | | | | | | |
| Tension rupture | 2 | 1.1 | 1 | 1.181 | 1.299 | 0.050 | 0.05 | 0.083 | 0.050 | 0.810 | 0.75 |
| Shear in bolts | 7 | 1.1 | 1 | 1.046 | 1.151 | 0.050 | 0.05 | 0.042 | 0.050 | 0.753 | 0.75 |
| Bearing in bolts | No Data | — | — | — | — | — | — | — | — | — | 0.75 |
| Tension in bolts | No Data | — | — | — | — | — | — | — | — | — | 0.75 |

| Table B-4a. Section Classification Limits in AISC *Specification* and AISC Design Guide, *Structural Stainless Steel* | | | | | |
|---|---|---|---|---|---|
| **Members Subject to Axial Compression** | | | | | |
| | | | **Width-to-Thickness Ratio** | **Limiting Width-to-Thickness Ratio $\lambda_r$ (nonslender/slender)** | |
| | **Case** | **Description of Element** | | **Carbon Steel** | **Stainless Steel** |
| Unstiffened Elements | 1 | Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees | $b/t$ | $0.56\sqrt{\dfrac{E}{F_y}}$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |
| | 2 | Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections | $b/t$ | $0.64\sqrt{\dfrac{k_cE}{F_y}}$ where $k_c = \dfrac{4}{h/t_w}$ | $0.47\sqrt{\dfrac{E}{F_y}}$ |
| | 3 | Legs of single angles, legs of double angles with separators, and all other unstiffened elements | $b/t$ | $0.45\sqrt{\dfrac{E}{F_y}}$ | $0.38\sqrt{\dfrac{E}{F_y}}$ |
| Stiffened Elements | 4 | Webs of doubly symmetric I-shaped sections and channels | $h/t_w$ | $1.49\sqrt{\dfrac{E}{F_y}}$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |
| | 5 | Walls of rectangular HSS and boxes of uniform thickness | $b/t$ | $1.40\sqrt{\dfrac{E}{F_y}}$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |
| | 6 | All other stiffened elements | $b/t$ | $1.49\sqrt{\dfrac{E}{F_y}}$ | $1.24\sqrt{\dfrac{E}{F_y}}$ |
| | 7 | Round HSS | $D/t$ | $0.11\dfrac{E}{F_y}$ | $0.10\dfrac{E}{F_y}$ |

**Table B-4b. Section Classification Limits in AISC *Specification* and AISC Design Guide, *Structural Stainless Steel***

| Members Subject to Flexure | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Case | Description of Element | Width-to-Thickness Ratio | $\lambda_p$ (compact / noncompact) Carbon Steel | $\lambda_p$ (compact / noncompact) Stainless Steel | $\lambda_r$ (noncompact / slender) Carbon Steel | $\lambda_r$ (noncompact / slender) Stainless Steel |
| Unstiffened Elements | 8 | Flanges of rolled I-shaped sections and channels | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | $0.33\sqrt{\frac{E}{F_y}}$ | $1.0\sqrt{\frac{E}{F_y}}$ | $0.47\sqrt{\frac{E}{F_y}}$ |
| Unstiffened Elements | 9 | Flanges of doubly and singly symmetric I-shaped built-up sections | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | $0.33\sqrt{\frac{E}{F_y}}$ | $0.95\sqrt{\frac{k_c E}{F_L}}$ | $0.47\sqrt{\frac{E}{F_y}}$ |
| Unstiffened Elements | 10 | Flanges of all I-shaped sections and channels in flexure about the weak axis | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | $0.33\sqrt{\frac{E}{F_y}}$ | $1.0\sqrt{\frac{E}{F_y}}$ | $0.47\sqrt{\frac{E}{F_y}}$ |
| Stiffened Elements | 11 | Webs of doubly symmetric I-shaped sections and channels | $h/t_w$ | $3.76\sqrt{\frac{E}{F_y}}$ | $2.54\sqrt{\frac{E}{F_y}}$ | $5.70\sqrt{\frac{E}{F_y}}$ | $3.01\sqrt{\frac{E}{F_y}}$ |
| Stiffened Elements | 12 | Flanges of rectangular HSS and boxes of uniform thickness | $b/t$ | $1.12\sqrt{\frac{E}{F_y}}$ | $1.12\sqrt{\frac{E}{F_y}}$ | $1.4\sqrt{\frac{E}{F_y}}$ | $1.24\sqrt{\frac{E}{F_y}}$ |
| Stiffened Elements | 13 | Webs of rectangular HSS and boxes | $h/t$ | $2.42\sqrt{\frac{E}{F_y}}$ | $2.42\sqrt{\frac{E}{F_y}}$ | $5.70\sqrt{\frac{E}{F_y}}$ | $3.01\sqrt{\frac{E}{F_y}}$ |
| Stiffened Elements | 14 | Round HSS | $D/t$ | $0.07\frac{E}{F_y}$ | $0.07\frac{E}{F_y}$ | $0.31\frac{E}{F_y}$ | $0.31\frac{E}{F_y}$ |

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$$

(6.49 of EN 1993-1-1)

and

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right] \quad \text{(B-6)}$$

where

$A$ = cross-sectional area (note that for Class 4 slender sections, the effective area is used)

$f_y$ = yield strength

$N_{cr}$ = elastic critical buckling load

$A_{eff}$ = effective cross-sectional area for Class 4 slender sections

$\lambda$ = slenderness (buckling length/radius of gyration)

$\alpha$ = imperfection factor

$\bar{\lambda}$ = $\sqrt{\dfrac{Af_y}{N_{cr}}} = \dfrac{\lambda}{\pi}\sqrt{\dfrac{f_y(A_{eff}/A)}{E}}$

$\bar{\lambda}_0$ = nondimensional limiting slenderness factor

For carbon steel, $\bar{\lambda}_0$ = 0.2 and $\alpha$ varies from 0.21 to 0.76 depending on the axis of buckling, the $h/b$ ratio, the flange thickness, the cross section shape, and the method of manufacture.

For stainless steel, $\bar{\lambda}_0 = 0.4$ and $\alpha = 0.49$ for cold-formed open sections and hollow sections. For welded open sections (buckling about the major axis) $\bar{\lambda}_0 = 0.2$ and $\alpha = 0.49$ and for welded open sections, (buckling about the minor axis) $\bar{\lambda}_0 = 0.2$ and $\alpha = 0.76$.

The carbon steel curve for $\bar{\lambda}_0 = 0.2$ and $\alpha = 0.34$ is shown in Figure B-1. This is the curve recommended for hot-rolled sections buckling about the minor axis where $h/b \leq 1.2$ and $t_f \leq 40$ mm.

#### B.5.1.2 The AISC *Specification* Methodology for Carbon Steel

For design of carbon steel to the AISC guidelines, the nominal compressive strength based on the limit state of flexural buckling is calculated from:

$$P_n = F_{cr}A_g \quad \text{(\textit{Spec}. Eq. E3-1)}$$

When $\dfrac{F_y}{F_e} \leq 2.25$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \qquad (\textit{Spec}.\ \text{Eq. E3-2})$$

When $\dfrac{F_y}{F_e} > 2.25$

$$F_{cr} = 0.877 F_e \qquad (\textit{Spec}.\ \text{Eq. E3-3})$$

where

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \qquad (\textit{Spec}.\ \text{Eq. E3-4})$$

$F_y$ = specified minimum yield strength, ksi

The AISC *Specification* gives only one buckling curve that applies to all sections. It is also shown in Figure B-1. The horizontal axis in Figures B-1 to B-5 is the member slenderness for flexural buckling, which is defined as:

$$\bar{\lambda} = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{\lambda}{\pi}\sqrt{\frac{F_y}{E}} \qquad \text{(B-7)}$$

Figure B-2 shows the stainless steel buckling curve in Eurocode 3 for welded open sections (buckling about the minor axis) alongside the test data that is available for stainless steel compression members. Using the test data for calibration and retaining the format of the AISC buckling expression, an AISC stainless steel buckling curve was derived and is also shown in Figure B-2.

The AISC stainless steel buckling curve is defined by the equations given in Section 5.3.

### B.5.1.3 Determination of Resistance Factor

Test data for this analysis was obtained from Young and Lui (2003), Talja and Salmi (1995), and Gardner and Nethercot (2004) for rectangular HSS. Data for I-shaped members was found in Talja (1997). Data for round HSS sections was found in Young and Hartono (2002), Talja (1997), Way (2000), and Rasmussen and Hancock (1990).

In accordance with the procedure described in Section B.2, a resistance factor, $\phi_c$, of 0.96 was calculated for flexural buckling of all compression members, which justifies the use of the AISC *Specification* carbon steel resistance factor, $\phi_c = 0.90$.

For a significant proportion (greater than 5%) of the round HSS tests, the predicted design strength, $\phi_c P_n$, exceeds the measured strength. This was a concern, even though it does not imply that the design model is unconservative since the predictions were based on the measured material properties and not the minimum specified material properties. In order not to penalize the entire range of section shapes in the study unnecessarily, round HSS were isolated into a separate population with a more conservative resistance factor. This conservatism can be reviewed as more test data becomes

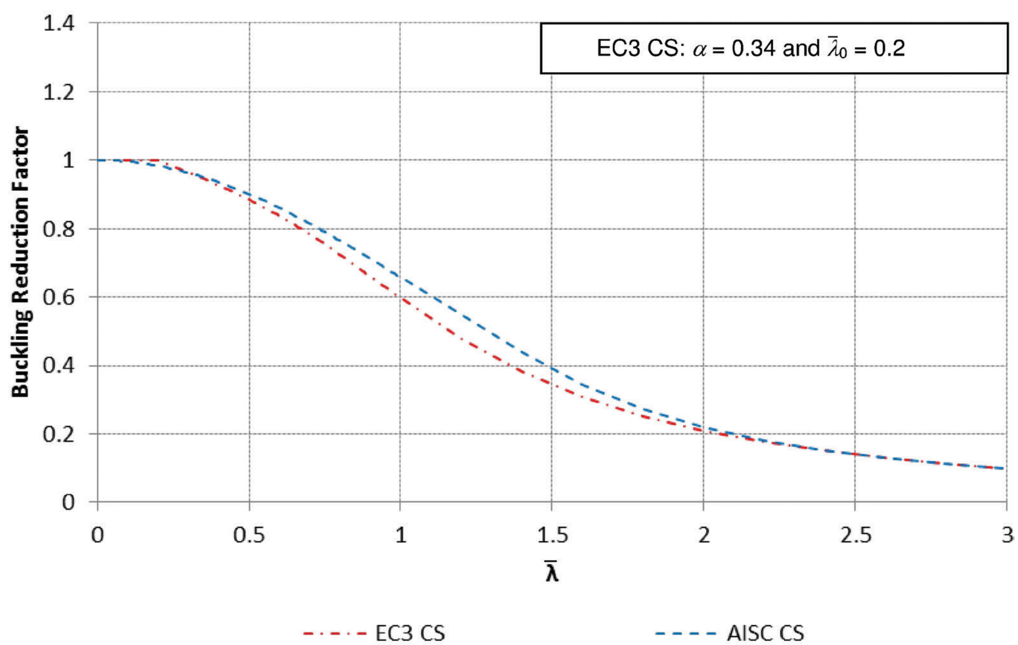



*Fig. B-1. Flexural buckling curves for carbon steel given in Eurocode 3 and the AISC* Specification.

available. (The reason why round HSS show inferior performance to rectangular HSS is that during the forming of rectangular HSS, the corners are cold worked and this increases the strength locally which improves buckling resistance. It is very likely that a more conservative buckling curve for round HSS will be introduced into EN 1993-1-4 in the next revision.)

For all section shapes, apart from round HSS, test data indicate $P_m = 1.316$ and $V_p = 0.234$. A resistance factor of 1.10 can therefore be derived. Figure B-3 shows the ratio of measured-to-predicted strengths versus member slenderness. The predicted strengths were calculated using the measured 0.2% offset yield strength and geometrical properties of the section. If they had been calculated using the minimum specified 0.2% offset strength, then the points would be further away from the line $\phi_c = 0.90$.

For round HSS, test data indicate $P_m = 1.043$ and $V_p = 0.154$. A resistance factor of 0.998 can therefore be derived. However, as discussed above, the presence of outlying results meant that in a few cases, the design strength, $\phi_c P_n$, exceeded the measured strength. The highest resistance factor which gives conservative results across the range of round HSS tested is $\phi_c = 0.85$.

Figure B-4 shows the ratio of measured-to-predicted strengths versus member slenderness for the round HSS data.

### B.5.2 Torsional and Flexural-Torsional Buckling of Members Without Slender Elements

The guidance in AISC *Specification* Section E4 applies to stainless steel, providing the stainless steel expressions for $F_{cr}$ given in modified AISC *Specification* Equations E3-2 and E3-3 are used where appropriate (see Section 5.3). Test data used in this analysis are given by van den Berg (1988) and are shown in Figure B-5.

By comparison of the proposed design model with test data, values of $P_m = 1.261$ and $V_p = 0.268$ were calculated for austenitic stainless steel. A resistance factor of 0.985 can therefore be derived in accordance with the procedure described in Section B.2. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended.

No data is available for duplex stainless steel. A resistance factor of 0.90 is therefore recommended, based on the findings for austenitic stainless steel.

### B.5.3 Single Angle Compression Members and Built-Up Members

There are no test data on the effects of eccentricity on stainless steel single angle members. It is assumed that the guidance for carbon steel in AISC *Specification* Section E5 applies. Similarly there are no test data on the performance of

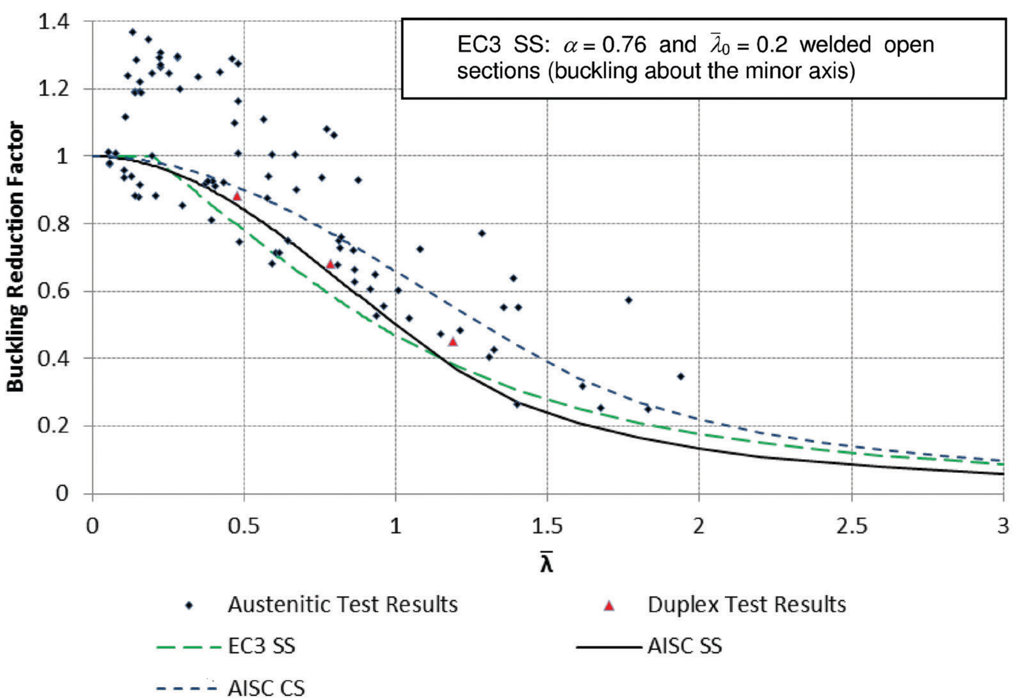



*Fig. B-2. Stainless steel flexural buckling curves and experimental results.*

built-up stainless steel compression members. It is assumed that the guidance for carbon steel in AISC *Specification* Section E6 applies. Both conditions are discussed in Section 5.5.

### B.5.4 Members with Slender Elements

#### B.5.4.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

For the design of Class 4 (slender) elements, Eurocode 3 uses an effective width approach. For slender compression members, an effective area is calculated on the basis of this effective width, which is used to modify the expressions for the member slenderness and buckling resistance. The effective width is found by applying a reduction factor, ρ, to the full width. Expressions for the reduction factor for stainless steel elements were derived by fitting curves to experimental data. Table B-5 gives the reduction factors for carbon steel and stainless steel presented in Eurocode 3. Gardner and Theofanous (2008) proposes modified expressions for stainless steel for defining plate buckling for slender elements, which align with the new cross-section classification recommendations. These are also given in the final column of Table B-5. The recommended expression for unstiffened elements in compression is the same as that for carbon steel in EN 1993-1-1 and the expression for stiffened elements is slightly lower than the corresponding one for carbon steel in EN 1993-1-1.

#### B.5.4.2 The AISC *Specification* Methodology for Carbon Steel

For the design of carbon steel slender elements, AISC uses two factors that reduce the available strength:

$Q_s$ is applied to unstiffened elements (i.e., flanges)
$Q_a$ is for stiffened elements (i.e., webs)

Once $Q_a$ and $Q_s$ have been calculated, a net reduction factor, $Q$, is calculated, where $Q = Q_a Q_s$. The yield strength is then reduced by this factor in the expressions for calculating the nominal compressive strength, $P_n$.

*Slender Unstiffened Elements, $Q_s$*

For flanges, angles and plates projecting from rolled columns or other compression members:

When $\dfrac{b}{t} \le 0.56\sqrt{\dfrac{E}{F_y}}$

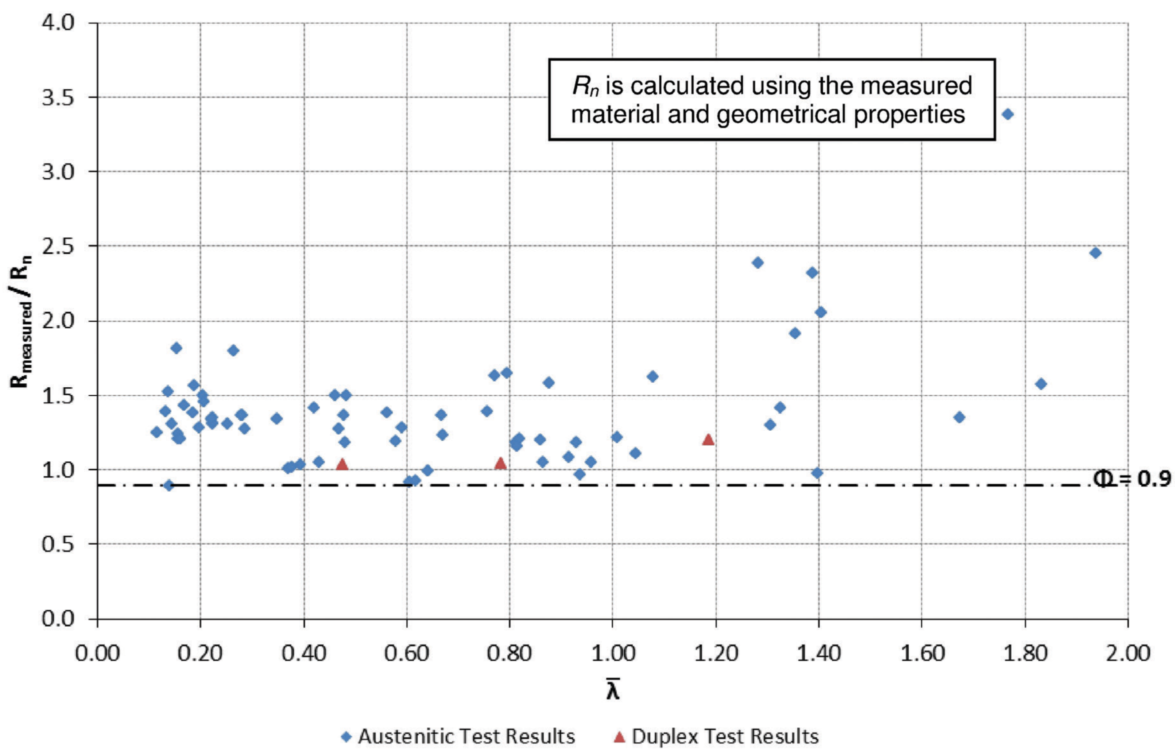



*Fig. B-3. Measured/predicted strengths versus member slenderness for members subject to compression (not including round HSS).*

| Table B-5. Carbon Steel and Stainless Steel Reduction Factors for Plate Buckling in Eurocode 3 | | | |
|---|---|---|---|
| **Element** | **Carbon Steel EN 1993-1-5** | **Stainless Steel EN 1993-1-4** | **Stainless Steel**[a] |
| Stiffened compression element | $\rho = \frac{1}{\bar{\lambda}_p} - \frac{0.22}{\bar{\lambda}_p^2} \leq 1$ | $\rho = \frac{0.772}{\bar{\lambda}_p} - \frac{0.125}{\bar{\lambda}_p^2} \leq 1$ | $\rho = \frac{0.772}{\bar{\lambda}_p} - \frac{0.079}{\bar{\lambda}_p^2} \leq 1$ |
| Unstiffened compression element | $\rho = \frac{1}{\bar{\lambda}_p} - \frac{0.188}{\bar{\lambda}_p^2} \leq 1$ | Cold-formed<br>$\rho = \frac{1}{\bar{\lambda}_p} - \frac{0.231}{\bar{\lambda}_p^2} \leq 1$<br>Welded<br>$\rho = \frac{1}{\bar{\lambda}_p} - \frac{0.242}{\bar{\lambda}_p^2} \leq 1$ | $\rho = \frac{1}{\bar{\lambda}_p} - \frac{0.188}{\bar{\lambda}_p^2} \leq 1$ |

[a] Expressions that are proposed for inclusion in the next revision of EN 1993-1-4 for stainless steel (Gardner and Theofanous, 2008).

Notes:

$\bar{\lambda}_p$ is the plate slenderness defined as: $\bar{\lambda}_p = \frac{b/t}{28.4\varepsilon\sqrt{k_\sigma}}$

$k_\sigma$ is the buckling factor corresponding to the stress ratio in the element. $k_\sigma = 4$ for a stiffened compression element subject to uniform compression and $k_\sigma = 0.429$ for an unstiffened element subject to uniform compression.

$\varepsilon$ is defined in Section B.7.1

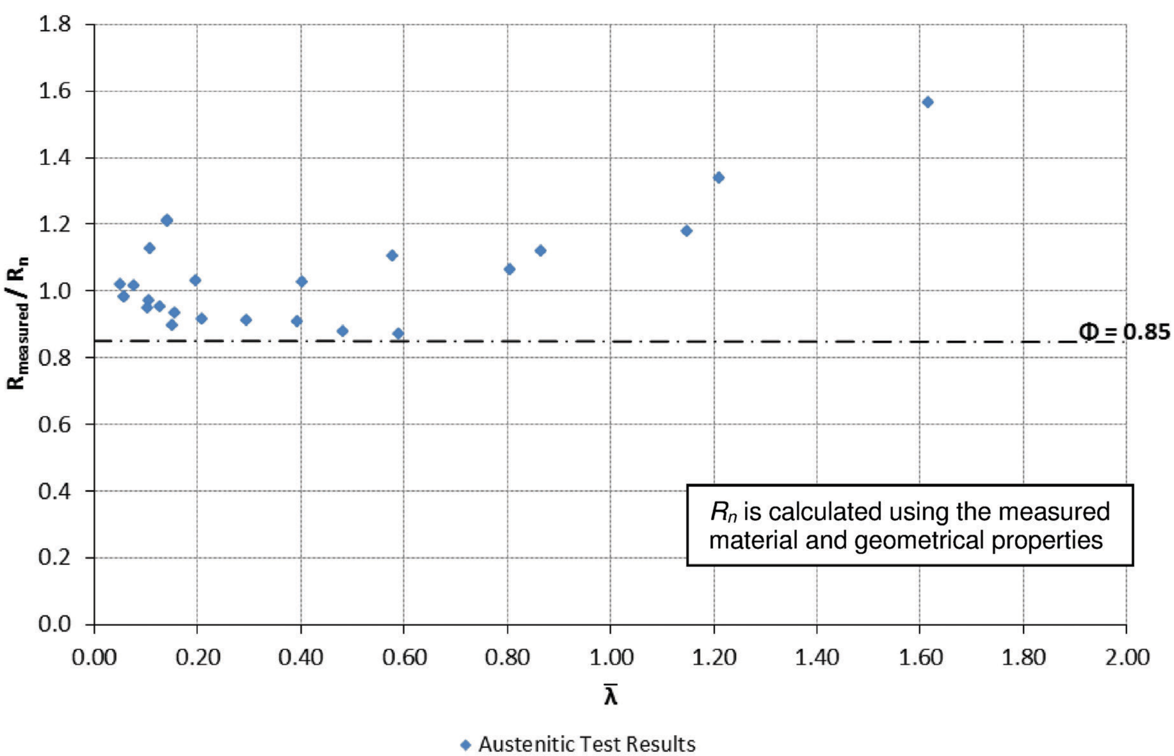



*Fig. B-4. Measured/predicted strengths versus member slenderness for round HSS members subject to compression.*

$$Q_s = 1.0 \qquad (Spec.\ \text{Eq. E7-4})$$

When $0.56\sqrt{\dfrac{E}{F_y}} < b/t \le 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \qquad (Spec.\ \text{Eq. E7-5})$$

When $b/t \ge 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = \frac{0.69}{F_y\left(\dfrac{b}{t}\right)^2}E \qquad (Spec.\ \text{Eq. E7-6})$$

For flanges, angles and plates projecting from built-up I-shaped columns or other compression members, similar expressions are given. These AISC equations are plotted in Figure B-6, alongside the plot of an equivalent expression for stainless steel unstiffened elements in Gardner and Theofanous (2008) (also given in the final column of Table B-5) denoted as "Provisional EC3 SS" in the key of the graph. (Note also that this expression is identical to the Eurocode carbon steel equation.)

The horizontal axis in Figures B-6 and B-7 is the plate slenderness given by $\bar{\lambda}_p = \dfrac{b}{t}\sqrt{\dfrac{F_y}{E}}$.

*Slender Stiffened Elements, $Q_a$*

$Q_a$ is calculated from:

$$Q_a = \frac{A_e}{A_g} \qquad (Spec.\ \text{Eq. E7-16})$$

where $A_e$ is the summation of the effective areas of the cross section based on the reduced effective width, $b_e$.

For uniformly compressed slender elements (except the flanges of square and rectangular HSS):

When $\dfrac{b}{t} \ge 1.49\sqrt{\dfrac{E}{f}}$

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{b/t}\sqrt{\frac{E}{f}}\right] \le b \qquad (Spec.\ \text{Eq. E7-17})$$

where $f$ is taken as $F_{cr}$ with $F_{cr}$ calculated based on $Q = 1.0$.

For flanges of square and rectangular HSS:

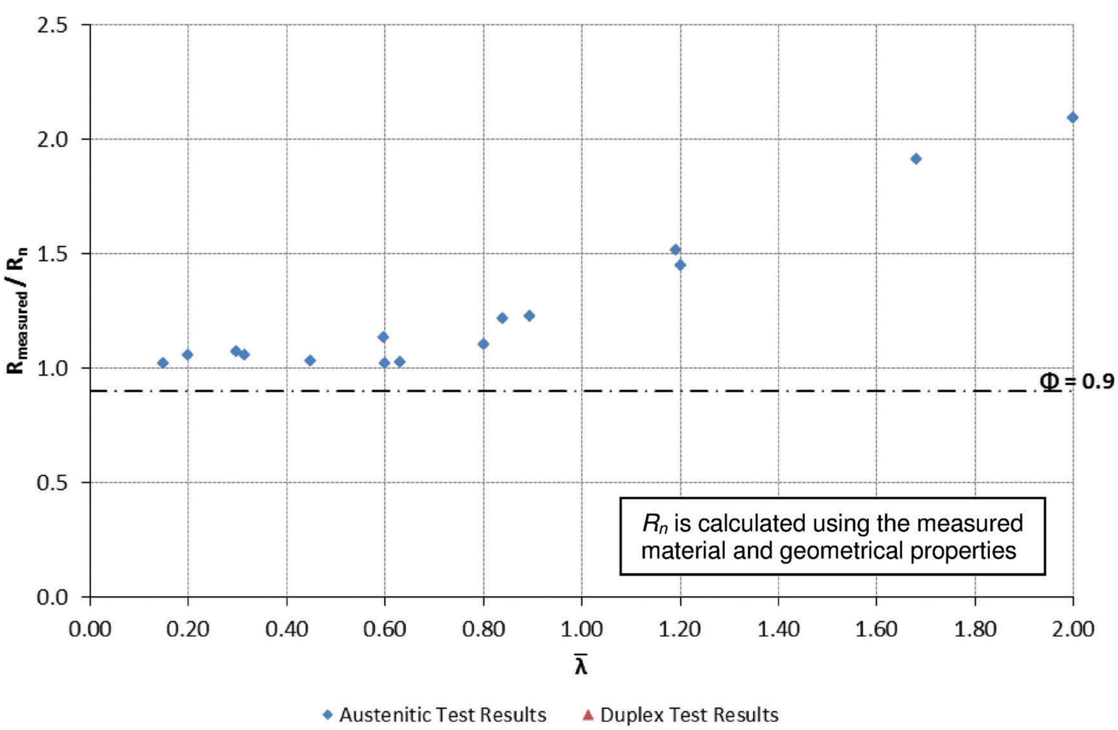



*Fig. B-5. Measured/predicted strengths versus member slenderness for members subject to flexural-torsional buckling.*

When $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{b/t}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(\textit{Spec}. Eq. E7-18)}$$

where $f = P_n/A_e$, or may conservatively be taken as equal to $F_y$.

The AISC *Specification* Equation E7-17 is plotted in Figure B-7 alongside the equivalent expression for stainless steel given in the final column of Table B-5.

### B.5.4.3 Recommendations for the AISC Design Guide

*Slender Unstiffened Elements, $Q_s$*

For unstiffened elements, the AISC equations for carbon steel have been modified to fit with the stainless steel section classification limits in Section B.3.3 and the stainless reduction factors given in the final column of Table B-5, as follows:

For flanges, angles and plates projecting from rolled or built-up I-shaped columns or other compression members:

For $\frac{b}{t} \leq 0.47\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \qquad \text{(\textit{Spec}. Eq. E7-4)}$$

For $0.47\sqrt{\frac{E}{F_y}} < b/t \leq 0.90\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.498 - 1.06\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \qquad \text{(modified \textit{Spec}. Eq. E7-5)}$$

For $b/t \geq 0.90\sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.44E}{F_y\left(\frac{b}{t}\right)^2} \qquad \text{(modified \textit{Spec}. Eq. E7-6)}$$

No guidance is given for single angles because there are

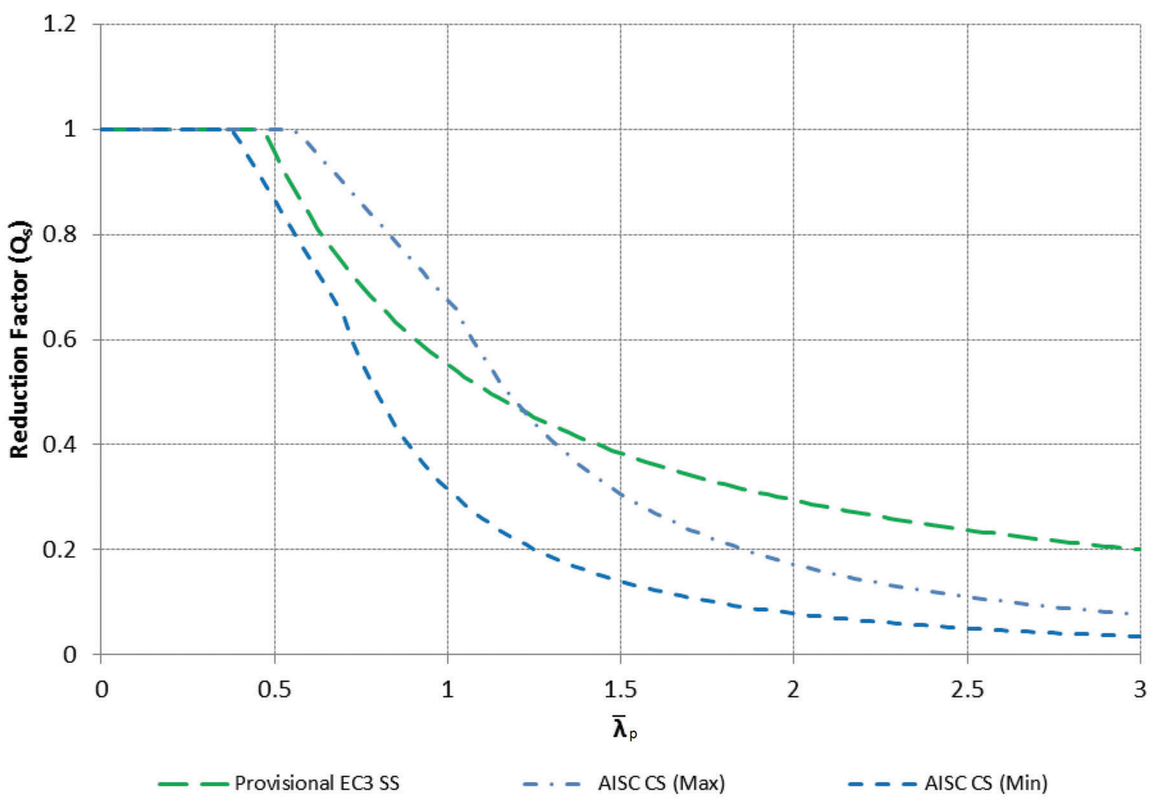



*Fig. B-6. Comparison of reduction factors for carbon and stainless steel versus plate slenderness for slender unstiffened elements (flanges).*

no stainless steel test data with which to verify the design expressions.

The expressions are plotted in Figure B-8 for flanges of I-shaped members and compared to the stainless reduction factor in the final column of Table B-5, denoted as "Provisional EC3 SS" in the key of the graph. Note that elements where $\bar{\lambda}_p > 1$ are very uncommon.

*Slender Stiffened Elements, $Q_a$*

For stiffened elements, including the flanges of square and rectangular HSS, the stainless steel reduction factor for stiffened elements given in the final column of Table B-5 was rearranged into the format of the AISC *Specification* equation to give:

When $\dfrac{b}{t} \geq 1.24\sqrt{\dfrac{E}{f}}$

$$b_e = 1.468t\sqrt{\frac{E}{f}}\left[1 - \frac{0.194}{(b/t)}\right]\sqrt{\frac{E}{f}} \leq b$$

(modified *Spec*. Eq. E7-17)

*Net Reduction Factor, Q*

In accordance with the AISC design procedure, once $Q_a$ and $Q_s$ have been calculated, the net reduction factor, $Q$, is calculated, where $Q = Q_a Q_s$. The yield strength is then reduced by this factor in the expressions for calculating the nominal compressive strength, $P_n$.

### B.5.4.4 Determination of Resistance Factor

Data for use in this analysis was found in the Commentary to the European *Design Manual* (Euro Inox and SCI, 2006b). The data set available for compression members with slender elements was split into two separate populations—rectangular HSS and I-shaped members. The two populations were analyzed separately.

By comparing the proposed design model with test data, a value of $P_m = 1.521$ and $V_p = 0.340$ was calculated for austenitic stainless steel rectangular HSS. A resistance factor of 1.021 can therefore be derived in accordance with the procedure described in Section B.2. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended. No data are available for duplex stainless

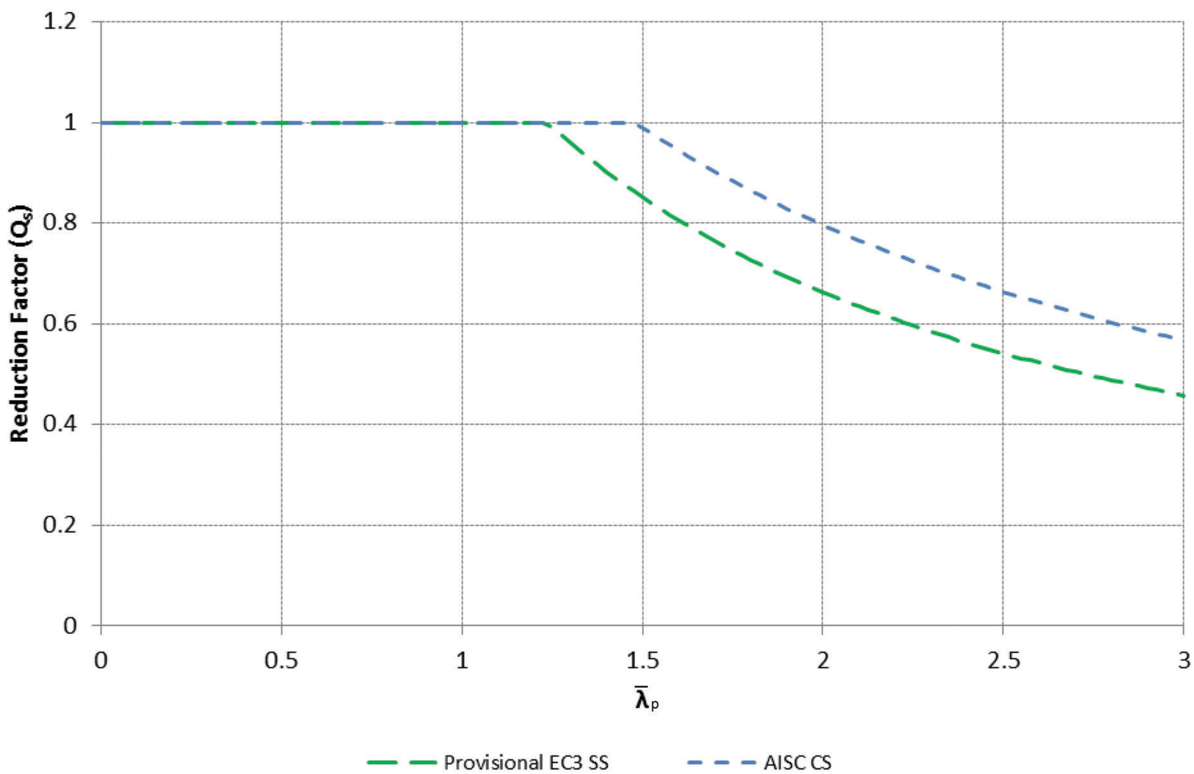



*Fig. B-7. Comparison of reduction factors against plate slenderness for slender stiffened elements (webs).*

steel. Figure B-9 shows the ratio of measured-to-predicted strengths versus plate slenderness for slender elements in rectangular HSS compression members.

By comparing the proposed design model with the test data, a value of $P_m = 1.136$ and $V_p = 0.164$ was calculated for austenitic I-shaped compression members with slender elements. A resistance factor of 1.071 can therefore be derived in accordance with the procedure described in Section B.2. For duplex stainless steel, values of $P_m = 1.221$ and $V_p = 0.102$ were determined. A resistance factor of 1.058 can therefore be derived. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended for both cases.

Figure B-10 shows the ratio of measured-to-predicted strengths versus plate slenderness for the slender elements of I-shaped compression members. A single result falls beneath the $\phi_c = 0.90$ line. This is statistically acceptable; therefore no adjustment was made to the resistance factor. (If there was more than one outlying point, the design line might need to be adjusted.)

Although the proposed AISC guidance appears too conservative, most of the conservative predictions were for unusually slender profiles (marked with circles) and outside the range of practical application (ratio of effective area to gross area < 0.45). If these sections are disregarded, then the proposed guidance is adequate.

## B.6 DESIGN OF MEMBERS FOR FLEXURE

Chapter 6 of this Design Guide addresses simple bending about one principal axis. It does not cover the design of angles and tees in flexure, nor the design of cross sections with slender webs in flexure (i.e., the AISC *Specification* Sections F5, F9 and F10). Section F13, Proportions of Beams and Girders, is also outside the scope of this Design Guide.

### B.6.1 Laterally Restrained Members

#### B.6.1.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

In the absence of shear and axial forces, the design moment resistance of a cross section subject to a uniaxial moment, $M_{c,Rd}$, should be taken as:

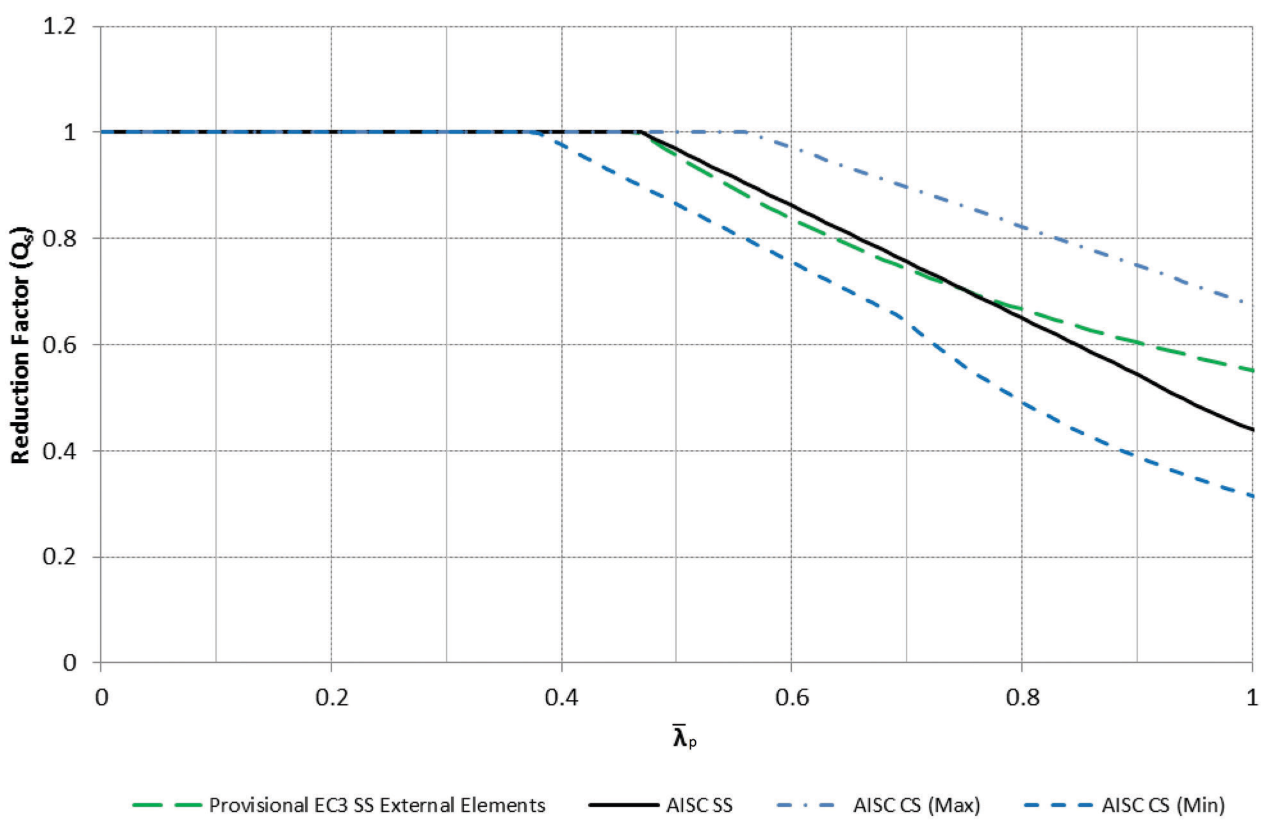



*Fig. B-8. Proposed reduction factor versus plate slenderness for unstiffened stainless steel elements compared to carbon steel (flanges of I-shaped members).*

For Class 1 or 2 cross sections

$$M_{c,Rd} = W_{pl} f_y / \gamma_{M0} \quad \text{(6.13 of EN 1993-1-1)}$$

For Class 3 cross sections

$$M_{c,Rd} = W_{el,min} f_y / \gamma_{M0} \quad \text{(6.14 of EN 1993-1-1)}$$

For Class 4 cross sections

$$M_{c,Rd} = W_{eff,min} f_y / \gamma_{M0} \quad \text{(6.15 of EN 1993-1-1)}$$

where

$W_{pl}$ = plastic section modulus

$W_{el,min}$ = elastic section modulus corresponding to the fiber with the maximum elastic stress

$W_{eff,min}$ = elastic modulus of effective section corresponding to the fiber with the maximum elastic stress

The effective section is calculated from the effective widths of the slender elements in the cross section; this approach is based on the post buckling reserve strength concept.

### B.6.1.2 The AISC *Specification* Methodology for Carbon Steel

Like Eurocode 3, for sections with compact webs and compact flanges, the member is permitted to reach its plastic moment strength ($F_yZ$) (e.g., AISC *Specification* Equations F2-1, F6-1, F7-1 and F11-1).

For members with noncompact webs or flanges, a different approach is taken from Eurocode 3 in that an intermediate strength between the plastic moment strength and elastic moment strength is calculated, which depends on the slenderness of the noncompact element (e.g., AISC *Specification* Equations F3-1, F4-13 and F6-2). This leads to higher values of capacity than those predicted by Eurocode 3.

For members with slender flanges, AISC utilizes the elastic critical buckling moment approach (e.g., AISC *Specification* Equations F3-2, F4-14 and F6-3). This leads to lower values of capacity than those predicted by Eurocode 3.

### B.6.1.3 Recommendations for the AISC Design Guide

The expressions in the AISC *Specification* for sections with compact webs and compact flanges for the limit state of yielding apply directly to stainless steel (e.g., AISC *Specification* Equations F2-1, F6-1, F7-1 and F11-1).

*I-Shaped Members and Channels with Noncompact Flanges or Noncompact Webs*

The expressions in the AISC *Specification* concerning compression flange local buckling (Equation F3-1), compression

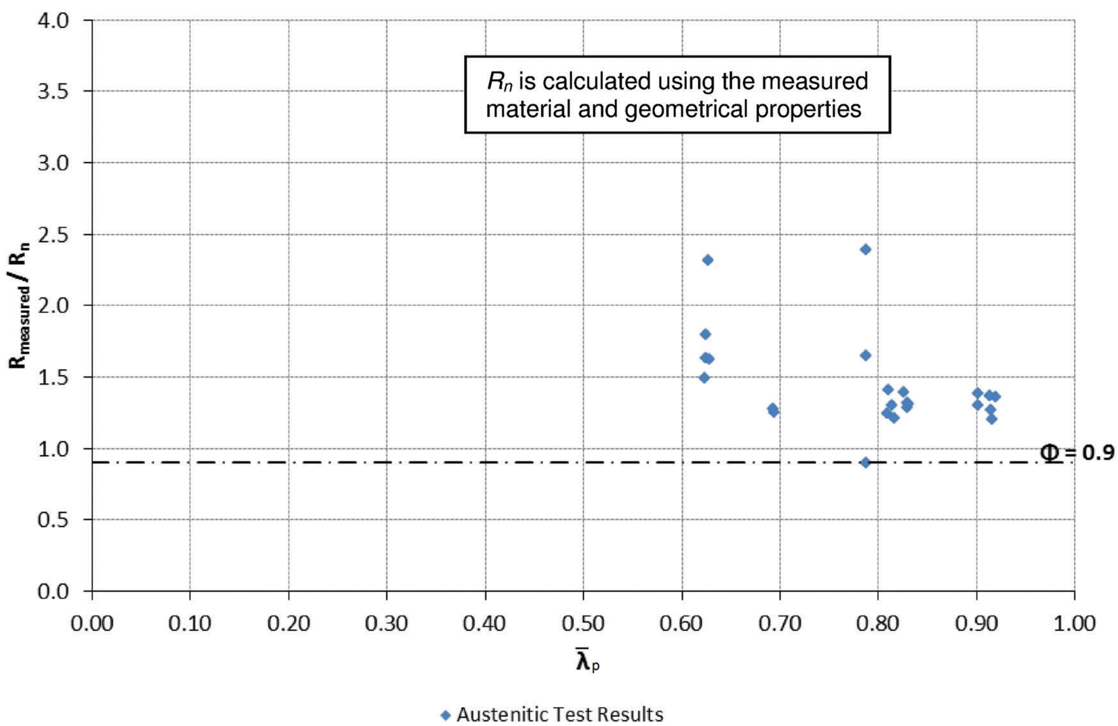



*Fig. B-9. Measured/predicted strengths versus plate slenderness for slender elements of rectangular HSS compression members.*

flange yielding (Equation F4-1), compression flange local buckling (Equation F4-13), tension flange yielding (Equation F4-15), and flange local buckling (Equation F6-2) apply directly to stainless steel, providing the correct values of $\lambda_{pf}$ and $\lambda_{rf}$ are used from Table 3-2 in this Design Guide.

*I-Shaped Members and Channels with Slender Flanges*

Because Eurocode 3 gives the same expression for plate buckling for unstiffened elements in compression for carbon steel and stainless steel, it seems reasonable to assume that the more conservative AISC *Specification* expressions for carbon steel slender flanges can also be applied to stainless steel. It is therefore assumed that the expressions in the AISC *Specification* concerning compression flange local buckling (Equations F3-2 and F4-14), and flange local buckling (Equation F6-3) apply directly to stainless steel.

*Square and Rectangular HSS and Box-Shaped Members with Noncompact Flanges*

The AISC *Specification* gives the following expression for flange local buckling for sections with noncompact flanges:

$$M_n = M_p - \left(M_p - F_y S\right)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

(*Spec*. Eq. F7-2)

Because the expression does not include the parameters $\lambda_{pf}$ and $\lambda_{rf}$ but is based on $\lambda_{pf} = 1.12\sqrt{E/F_y}$ and $\lambda_{rf} = 1.40\sqrt{E/F_y}$, it requires modification to align with the limiting width-to-thickness ratios for stainless steel in Table 3-2 of this Design Guide ($\lambda_{pf} = 1.12\sqrt{E/F_y}$ and $\lambda_{rf} = 1.24\sqrt{E/F_y}$). The modified expression is:

$$M_n = M_p - \left(M_p - F_y S\right)\left(14.3\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 16.7\right) \le M_p$$

(modified *Spec*. Eq. F7-2)

*Square and Rectangular HSS and Box-Shaped Members with Slender Flanges*

In the AISC *Specification* for sections with slender flanges, the nominal flexural strength for the limit state of flange local buckling is based on an effective section modulus that is based on an effective width given in Equation F7-4:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{\left(b/t_f\right)}\sqrt{\frac{E}{F_y}}\right] \le b$$

(*Spec*. Eq. F7-4)

However, as explained in Section B.5.4.3, the expression needs modification for stainless steel and becomes:

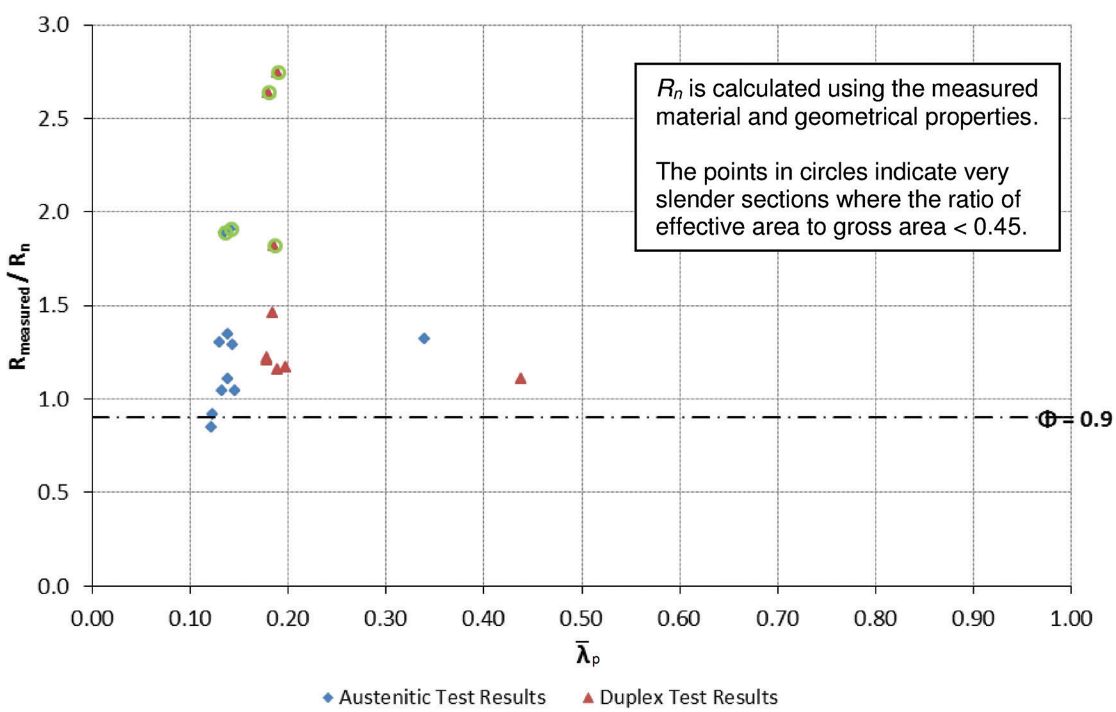



*Fig. B-10. Measured/predicted strengths versus plate slenderness for slender elements of I-shaped compression members.*

$$b_e = 1.468 t_f \sqrt{\frac{E}{F_y}} \left[ 1 - \frac{0.194}{(b/t_f)} \sqrt{\frac{E}{F_y}} \right] \le b$$

(modified *Spec*. Eq. F7-4)

The expression in Equation F7-5 for web local buckling in the AISC *Specification* for sections with noncompact webs is:

$$M_n = M_p - (M_p - F_y S_x)\left(0.305 \frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p$$

(*Spec*. Eq. F7-5)

Again, as the expression does not include the parameters $\lambda_{pw}$ and $\lambda_{rw}$ but is based on $\lambda_{pw} = 2.42\sqrt{E/F_y}$ and $\lambda_{rw} = 5.70\sqrt{E/F_y}$, it requires modification to align with the limiting width-to-thickness ratios for stainless steel in Table 3-2 ($\lambda_{pw} = 2.42\sqrt{E/F_y}$ and $\lambda_{rfw} = 3.01\sqrt{E/F_y}$):

$$M_n = M_p - (M_p - F_y S_x)\left(2.13 \frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 5.40\right) \le M_p$$

(modified *Spec*. Eq. F7-5)

*Noncompact Round HSS*

The expression for nominal flexural strength in Equation F8-2 for local buckling in the AISC *Specification* for noncompact sections is:

$$M_n = \left[ \frac{0.021E}{\left(\frac{D}{t}\right)} + F_y \right] S \qquad \text{(\textit{Spec}. Eq. F8-2)}$$

As the $\lambda_p$ and $\lambda_r$ values for round HSS in bending are nearly identical for carbon steel and stainless steel (see Table B-4b in this document), it is recommended that this expression be adopted for stainless steel without change.

Note: There is extensive test evidence showing that the plastic bending moment, $M_p = F_y Z$, is reached in stainless steel members at relatively low rotations, probably lower than carbon steel because of the strain hardening (Theofanous et al., 2009).

#### B.6.1.4 Determination of Resistance Factor

Data for this study were obtained from Kiymaz (2005), Talja (1997), and Rasmussen and Hancock (1990) for round HSS sections. Data for rectangular HSS in bending were obtained from Real (2001), Talja and Salmi (1995), Gardner (2002), Zhou and Young (2005), Gardner et al. (2006), and Theofanous (2010). I-shaped member data were obtained from Talja (1997) and Real (2001).

In accordance with the procedure described in Section B.2, a value of $P_m$ = 1.383 and $V_p$ = 0.163 for austenitic stainless steel was calculated, which leads to a resistance factor of 1.306. For duplex stainless steel, $P_m$ = 1.263 and $V_p$ = 0.063 were calculated, which leads to a resistance factor of 1.135. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended for both cases.

Figure B-11 shows the ratio of measured-to-predicted strengths versus elastic section modulus.

### B.6.2 Laterally Unrestrained Members (Lateral-Torsional Buckling)

#### B.6.2.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

Eurocode 3 uses a buckling curve approach similar to that used for flexural buckling for both carbon and stainless steels. The buckling resistance moment is given by:

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \qquad \text{(6.55 of EN 1993-1-1)}$$

where

$\chi_{LT}$ = lateral-torsional buckling resistance factor

$$= \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \le 1.0 \qquad \text{(6.56 of EN 1993-1-1)}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

The choice of value for the imperfection factor, $\alpha_{LT}$ (0.21, 0.34, 0.49 or 0.76), depends on the shape of cross section, method of manufacture, and *h*/*b* ratio.

$W_y = W_{pl,y}$ for Class 1 or 2 cross sections
$W_y = W_{el,y}$ for Class 3 cross sections
$W_y = W_{eff,y}$ for Class 4 cross sections

For stainless steel, the same expressions apply except the limiting slenderness is 0.4 instead of 0.2:

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.4\right) + \bar{\lambda}_{LT}^2\right]$$

(5.11 of EN 1993-1-4)

$\alpha_{LT}$ is taken as 0.34 for cold-formed sections and hollow sections (welded and seamless) and 0.76 for welded open sections and other sections for which no test data is available.

### B.6.2.2 The AISC *Specification* Methodology for Carbon Steel

The AISC approach is based on elastic theory, though part of the curve is lower than elastic theory due to residual stresses. The equations for doubly symmetric compact I-shaped members and channels bent about their major axis are as follows:

For $L_b \le L_p$

$$M_n = M_p = F_y Z_x \qquad \text{(Spec. Eq. F2-1)}$$

For $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(Spec. Eq. F2-2)}$$

For $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p \qquad \text{(Spec. Eq. F2-3)}$$

where the terms are all defined in the AISC *Specification*.

Figure B-12 compares the curves in Eurocode 3 (carbon and stainless steel) and the AISC *Specification*. The AISC curve is significantly higher than the Eurocode curves.

### B.6.2.3 Recommendations for the AISC Design Guide

Three modifications were made to the AISC expressions in order to generate a lower design curve close to the stainless steel test data for flexural strength. The modifications involved changing the coefficient in AISC *Specification* Equations F2-2, F2-3 and F2-5 for doubly symmetric I-shaped members and channels with compact webs. Equivalent modifications were made to the corresponding equations for other I-shaped members with compact or noncompact webs:

For $L_p < L_b \le L_r$

Doubly Symmetric Compact I-Shaped Members and Channels

$$M_n = C_b\left[M_p - \left(M_p - 0.45F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(modified Spec. Eq. F2-2)}$$

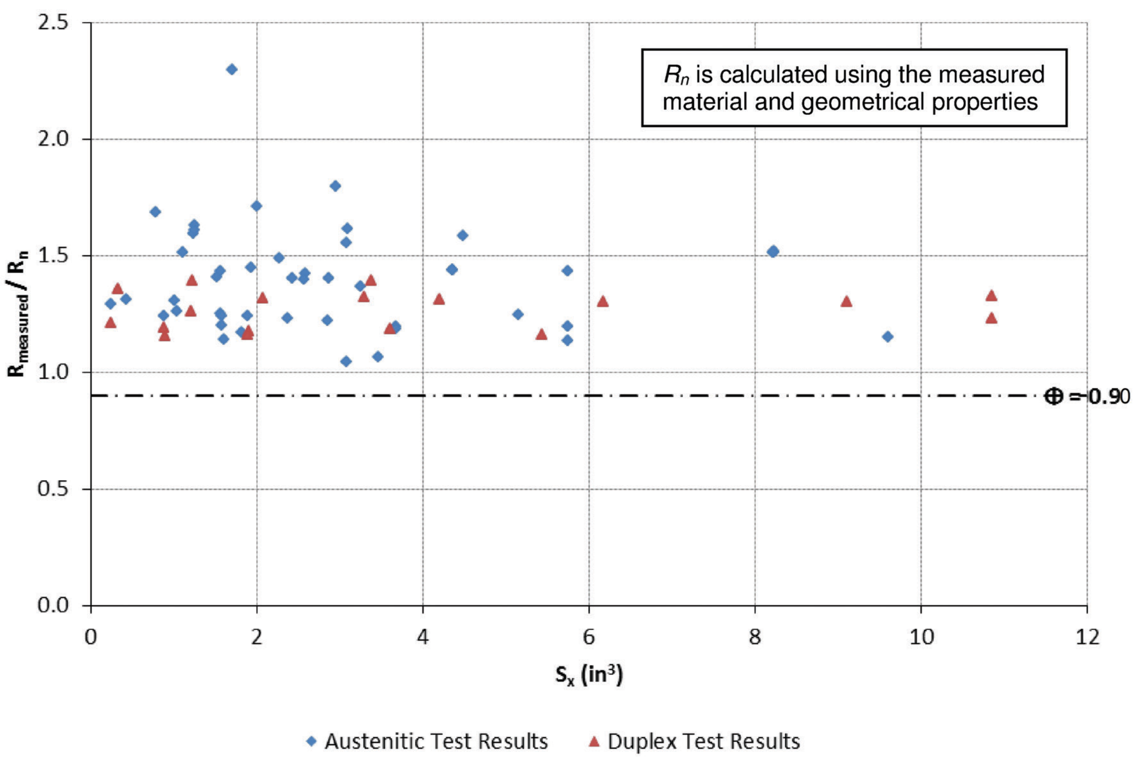



*Fig. B-11. Measured/predicted bending strengths versus elastic section modulus for laterally restrained members.*

Other I-Shaped Members with Compact or Noncompact Webs

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - 0.64F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc}$$

(modified *Spec*. Eq. F4-2)

For $L_b > L_r$

Doubly Symmetric Compact I-Shaped Members and Channels

$$M_n = 0.64F_{cr}S_x \le M_p \qquad \text{(modified } \textit{Spec}\text{. Eq. F2-3)}$$

where

$$L_p = 0.8r_y\sqrt{\frac{E}{F_y}} \qquad \text{(modified } \textit{Spec}\text{. Eq. F2-5)}$$

I-Shaped Members with Compact or Noncompact Webs

$$M_n = 0.64F_{cr}S_x \le R_{pc}M_{yc} \qquad \text{(modified } \textit{Spec}\text{. Eq. F4-3)}$$

where

$$L_p = 0.5r_t\sqrt{\frac{E}{F_y}} \qquad \text{(modified } \textit{Spec}\text{. Eq. F4-7)}$$

Figure B-13 shows these modified curves compared to the stainless steel welded Eurocode 3 curve and AISC curve for carbon steel. The horizontal axis is the member slenderness for lateral-torsional buckling, $\bar{\lambda}_{LT}$, which is defined in Section B.6.2.1.

#### B.6.2.4 Determination of Resistance Factor

Data for this analysis was obtained from Stangenberg (2000a) and van Wyk et al. (1990). By comparing the design model with test data, values of $P_m$ = 1.261 and $V_p$ = 0.191 were calculated for austenitic stainless steel. A resistance factor of 1.139 can therefore be derived in accordance with the procedure described in Section B.2. For duplex stainless steel, values of $P_m$ = 1.503 and $V_p$ = 0.267 were calculated, although only two results were available for calibration. A resistance factor of 0.997 can be derived. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is again recommended for both duplex and austenitic stainless steel.

Figure B-14 shows the ratio of measured-to-predicted strengths against lateral-torsional buckling slenderness for laterally unrestrained members. One test point falls beneath the $\phi_b$ = 0.90 line, which is statistically acceptable.

### B.6.3 Determination of Deflection

This approach is taken directly from EN 1993-1-4, with the justification explained in the Commentary to the European *Design Manual* (Euro Inox and SCI, 2006b). The values for *n,* the Ramberg Osgood parameter, given in Table 6-1 have been taken from Afshan et al. (2013), which describes a thorough assessment of all available stainless steel stress-strain data based on testing carried out in 2012. Most of the data available were for cold-formed material, which displays a more rounded stress-strain curve than hot-rolled material and hence are characterized by lower values of *n*. The *n*

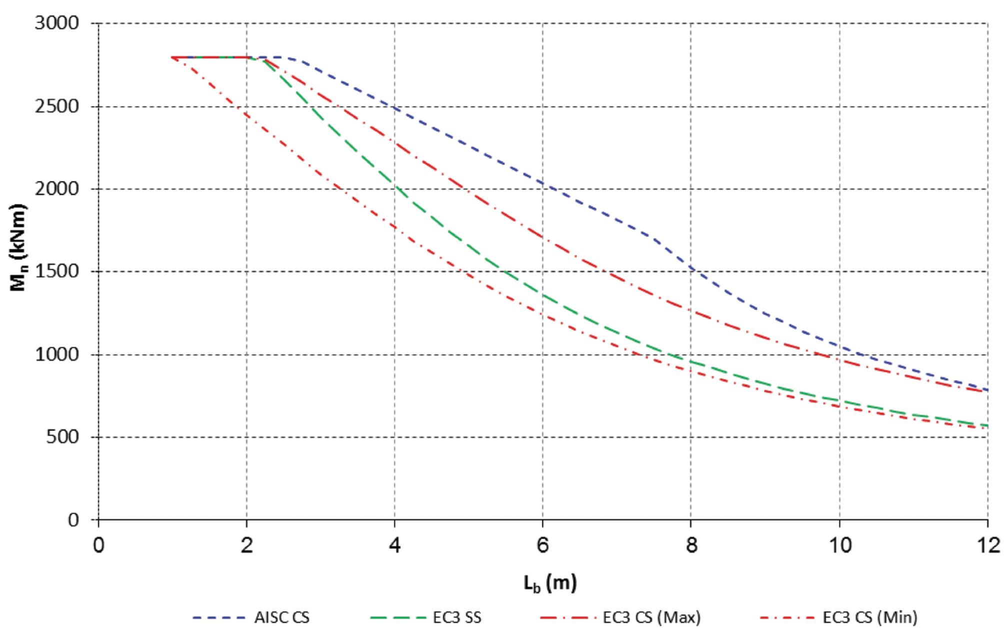



*Fig. B-12. AISC and EC3 design curves for lateral-torsional buckling.*

values derived for the cold-formed data are therefore conservative for the assessment of deflections in hot-rolled and welded sections.

## B.7 DESIGN OF MEMBERS FOR SHEAR

### B.7.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

The design plastic shear resistance is given by:

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\,\gamma_{M0}} \qquad \text{(6.18 of EN 1993-1-1)}$$

where $A_v$ is the shear area.

For welded sections, $A_v$ is taken as the web height multiplied by the web thickness, multiplied by a factor η, i.e., $A_v = \eta h_w t$. For rolled sections, the radius of the section is included in the shear area with a lower bound value of $A_v = \eta h_w t$.

Shear buckling of webs without stiffeners should be checked if:

$h_w/t \geq 72\varepsilon/\eta$ for carbon steel

and

$h_w/t \geq 52\varepsilon/\eta$ for stainless steel

Eurocode 3 gives one method of calculating the contribution of the shear design resistance from the web. The method does not take into account the post-buckling strength of the web (tension field action). The resistance is given by:

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\,\gamma_{M1}} \qquad \text{(5.1 of EN 1993-1-5)}$$

For transverse stiffeners at supports only:

$$\bar{\lambda}_w = \frac{h_w}{86.4\, t\varepsilon} \qquad \text{(5.5 of EN 1993-1-5)}$$

where $\varepsilon = \sqrt{\dfrac{235}{f_y}}$ for carbon steel and $\varepsilon = \left[\dfrac{235}{f_y}\,\dfrac{E}{210{,}000}\right]^{0.5}$ for stainless steel.

Assuming the case of a beam with a nonrigid end post:

Carbon steel

For $\bar{\lambda}_w < 0.83/\eta$ $\qquad \chi_w = \eta$ (Table 5.1 of EN 1993-1-5)

For $\bar{\lambda}_w \geq 0.83/\eta$ $\qquad \chi_w = 0.83/\bar{\lambda}_w$ (Table 5.1 of EN 1993-1-5)

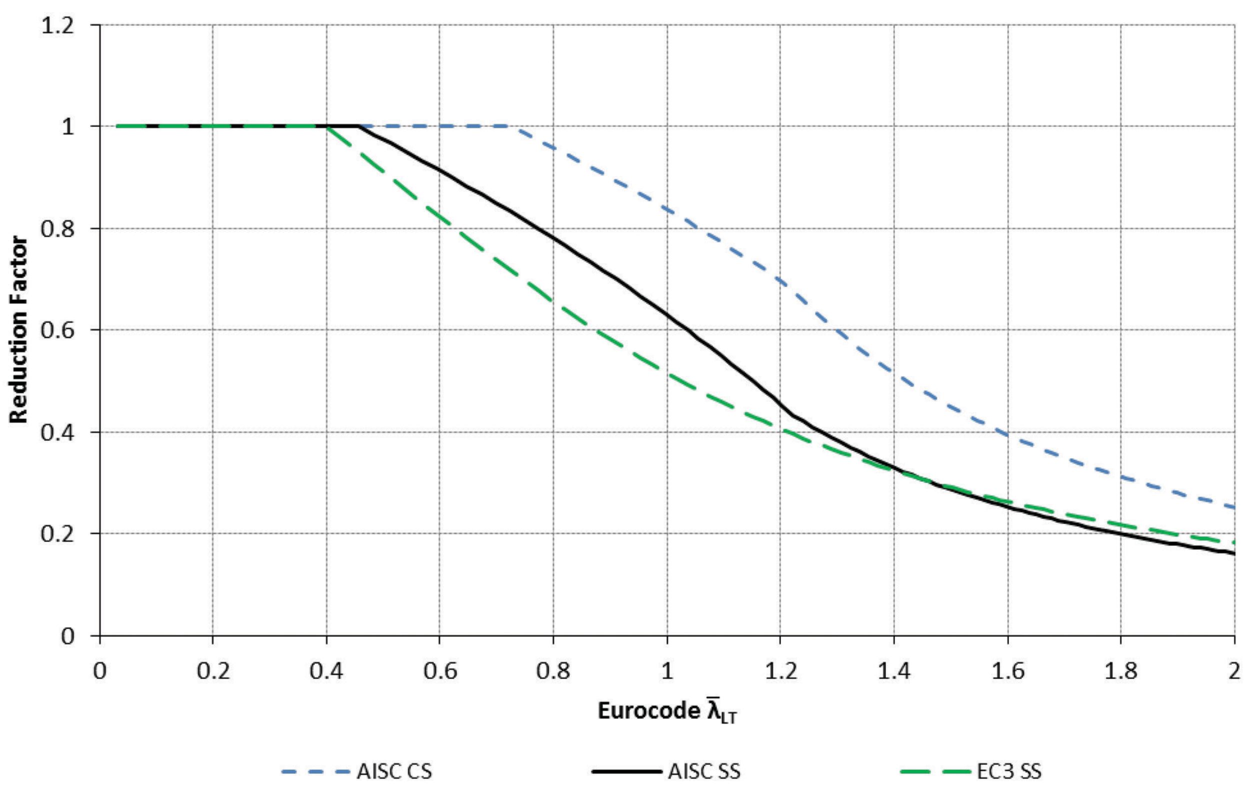



*Fig. B-13. Lateral-torsional buckling curves: AISC carbon steel, EC3 stainless steel, and recommended AISC stainless steel.*

Stainless steel

For $\overline{\lambda}_w < 0.6/\eta$ $\qquad \chi_w = \eta \qquad$ (5.18 of EN 1993-1-4)

For $\overline{\lambda}_w > 0.60/\eta$

$$\chi_w = 0.11 + \frac{0.64}{\overline{\lambda}_w} - \frac{0.05}{\overline{\lambda}_w^{\,2}} \qquad \text{(5.19 of EN 1993-1-4)}$$

The recommended value of $\eta = 1.2$ is for both carbon steel and stainless steel. Note that no guidance is given for stainless steel in the less onerous case of a rigid end post due to a lack of test data.

### B.7.2 The AISC *Specification* Methodology for Carbon Steel

The AISC *Specification* gives two methods for calculating the shear strength of a web; the method in Section G2 does not utilize the post-buckling strength of the web (i.e., tension field action) whereas the method in Section G3 does. In this Design Guide, the method in Section G2 is adopted for stainless steel. No guidance is given on a method which utilizes the post-buckling strength for stainless steel plate girders. It is suggested that this topic is researched and guidance added later.

The nominal shear strength, $V_n$, of unstiffened or stiffened webs according to the limits of shear yielding and shear buckling is given by:

$$V_n = 0.6F_yA_wC_v \qquad (\textit{Spec.}\ \text{Eq. G2-1})$$

where

$A_w$ = shear area, taken as the overall depth of the section multiplied by the web thickness

$C_v$ = web shear coefficient

= 1.0 for webs of rolled I-shaped members with

$$\frac{h}{t_w} \le 2.24\sqrt{\frac{E}{F_y}}$$

Elsewhere, $C_v$ is calculated as follows:

When $\dfrac{h}{t_w} \le 1.10\sqrt{\dfrac{k_vE}{F_y}}$

$$C_v = 1.0 \qquad (\textit{Spec.}\ \text{Eq. G2-3})$$

When $1.10\sqrt{\dfrac{k_vE}{F_y}} < \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_vE}{F_y}}$

$$C_v = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w} \qquad (\textit{Spec.}\ \text{Eq. G2-4})$$

When $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_vE}{F_y}}$

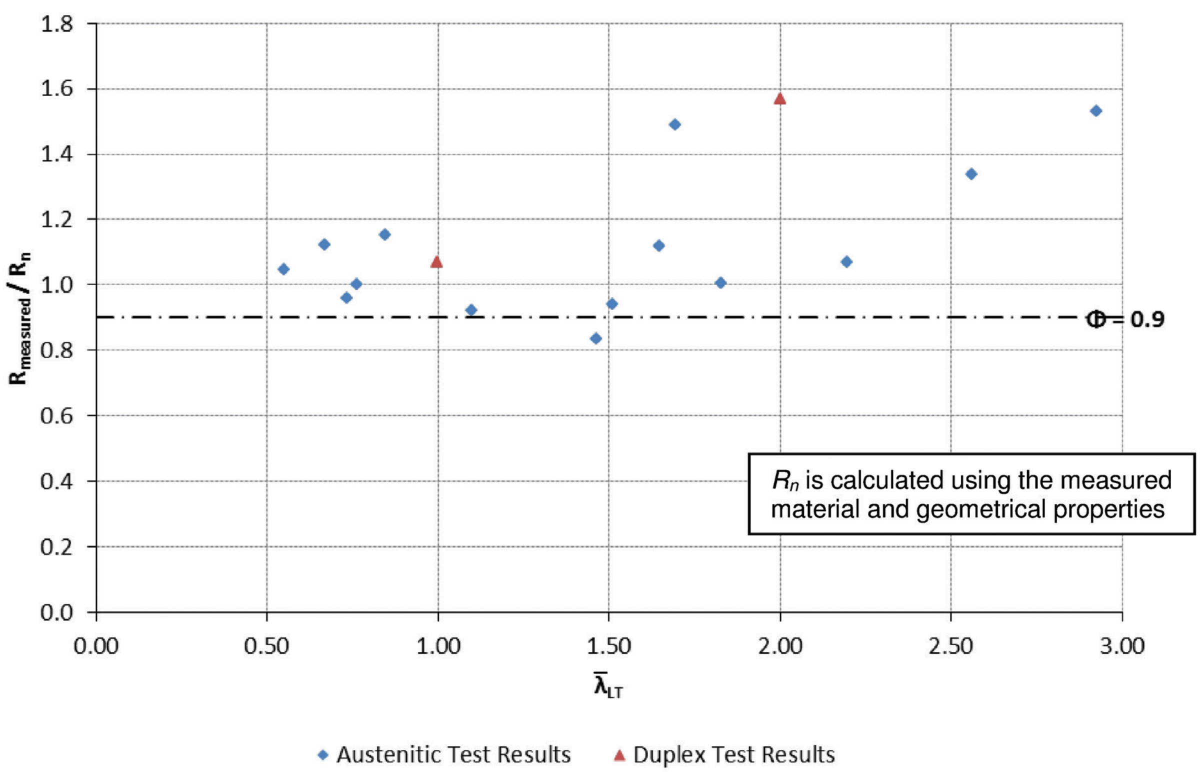



*Fig. B-14. Measured/predicted strengths for laterally unrestrained members against slenderness.*

$$C_v = \frac{1.51Ek_v}{\left(h/t_w\right)^2 F_y} \qquad (\textit{Spec}. \text{ Eq. G2-5})$$

For webs without transverse stiffeners and with $h/t_w < 260$, $k_v = 5$.

### B.7.3 Recommendations for the AISC Design Guide

In order to compare the approaches, it is necessary to plot the buckling curves nondimensionally as the ratio $E/F_y$ is different for stainless steel and carbon steel. Figure B-15 shows the Eurocode web shear buckling curves and AISC web shear buckling curve [carbon steel (CS) and stainless steel (SS)]. Note that $\chi_w$ in Eurocode 3 is not exactly equivalent to $C_v$ in the AISC *Specification* because of the different shear areas assumed in the standards. To enable the approaches to be compared, η in Eurocode 3 is taken as 1.0.

For carbon steel, $\bar{\lambda}_w = \dfrac{h_w}{t}\dfrac{1}{86.4}\sqrt{\dfrac{f_y}{235}}$, and $\dfrac{h_w}{t} = \dfrac{1{,}324}{\sqrt{f_y}}\bar{\lambda}_w$.

Therefore, assuming $E$ = 29,000 ksi (200 000 MPa) and η = 1.0, the Eurocode and AISC expressions can be rewritten as:

Carbon steel—Eurocode 3

For $\bar{\lambda}_w \le 0.83$ $\qquad \chi_w = 1.0$

For $\bar{\lambda}_w > 0.83$ $\qquad \chi_w = 0.83/\bar{\lambda}_w$

Stainless steel—Eurocode 3

For $\bar{\lambda}_w \le 0.6$ $\qquad \chi_w = 1.0$

For $\bar{\lambda}_w > 0.6$ $\qquad \chi_w = 0.11 + \dfrac{0.64}{\bar{\lambda}_w} - \dfrac{0.05}{\bar{\lambda}_w^{\,2}}$

Carbon steel—AISC

For $\bar{\lambda}_w \le 0.831$ $\qquad C_v = 1.0$

For $0.831 < \bar{\lambda}_w \le 1.035$ $\qquad C_v = \dfrac{0.831}{\bar{\lambda}_w}$

For $\bar{\lambda}_w > 1.035$ $\qquad C_v = \dfrac{0.861}{\bar{\lambda}_w^{\,2}}$

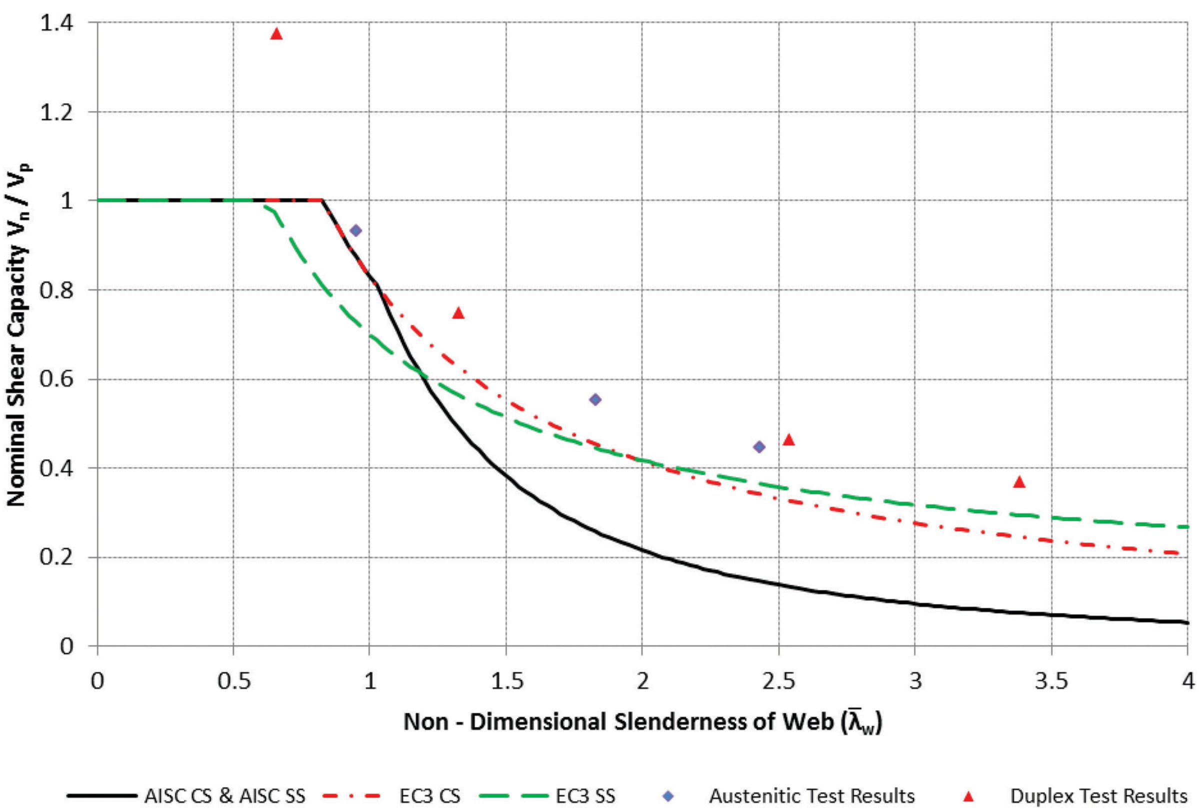



*Fig. B-15. Shear buckling test data and design curves.*

Test data are also shown in Figure B-15, which are reported in the Commentary to the European *Design Manual* (Euro Inox and SCI, 2006b), Real et al. (2003), Estrada et al. (2008), Unosson and Olsson (2003), Estrada et al. (2007), and Real (2001). Comparison of the test data with design curves indicates that AISC guidance for carbon steel given in Section G2 may be adopted, without modifications, for stainless steel. Note that the AISC guidance in Section G3 based on tension field action gives much less conservative values for medium to high $h/t_w$ ratios than the guidance in Section G2; however, a comparison of this with stainless steel test data is beyond the scope of this Design Guide.

### B.7.4 Determination of Resistance Factor

No test results were available for plastic shear failure of cross sections. A resistance factor of 0.9 is therefore considered appropriate.

By comparing the design model with test data for shear buckling, values of $P_m = 1.116$ and $V_p = 0.108$ for austenitic stainless steel were calculated. A resistance factor of 1.137 can therefore be derived in accordance with the procedure described in Section B.2. For duplex stainless steel, values of $P_m = 1.169$ and $V_p = 0.092$ were calculated, although only two results were available for calibration. A resistance factor of 1.024 can be derived. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended for both.

Figure B-16 shows the ratio of measured-to-predicted strengths versus web slenderness, $\bar{\lambda}_w$.

## B.8 DESIGN OF MEMBERS FOR COMBINED FORCES

### B.8.1 Eurocode 3 Methodology for Carbon Steel and Stainless Steel

The resistance of a member subjected to combined axial compression and bending is determined using interaction formulae which assess the buckling resistance of the member. In addition, the resistance of the cross section must be checked at each end of the member. In Eurocode 3, the approach is similar for carbon steel and stainless steel, although there are some differences in the calculation of the interaction factors.

For stainless steel members subject to axial compression and biaxial moments, EN 1993-1-4 requires that all members satisfy both the following interaction formulae, which relate to in-plane and out-of-plane buckling, respectively:

$$\frac{N_{Ed}}{(N_{b,Rd})_{min}} + k_y\left(\frac{M_{y,Ed} + N_{Ed}\,e_{Ny}}{\beta_{W,y}\,W_{pl,y}\,f_y/\gamma_{M1}}\right) + k_z\left(\frac{M_{z,Ed} + N_{Ed}\,e_{Nz}}{\beta_{W,z}\,W_{pl,z}\,f_y/\gamma_{M1}}\right) \le 1 \qquad \text{(5.16 of EN 1993-1-4)}$$

Members potentially subject to lateral-torsional buckling should also satisfy:

$$\frac{N_{Ed}}{(N_{b,Rd})_{min1}} + k_{LT}\left(\frac{M_{y,Ed} + N_{Ed}\,e_{Ny}}{M_{b,Rd}}\right) + k_z\left(\frac{M_{z,Ed} + N_{Ed}\,e_{Nz}}{\beta_{W,z}\,W_{pl,z}\,f_y/\gamma_{M1}}\right) \le 1 \qquad \text{(5.17 of EN 1993-1-4)}$$

In the above expressions:

$e_{Ny}$ and $e_{Nz}$ are the shifts in the neutral axes when the cross section is subject to uniform compression

$N_{Ed}$, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the maximum moments about the *y*- and *z*- axis along the member, respectively

$(N_{b,Rd})_{min}$ is the smallest value of $N_{b,Rd}$ for the following four buckling modes: flexural buckling about the *y*-axis, flexural buckling about the *z*-axis, torsional buckling, and flexural-torsional buckling

$(N_{b,Rd})_{min1}$ is the smallest value of $N_{b,Rd}$ for the following three buckling modes: flexural buckling about the *z*-axis, torsional buckling, and flexural-torsional buckling

$\beta_{W,y}$ and $\beta_{W,z}$ are the values of $\beta_W$ determined for the *y*- and *z*-axes, respectively, where

$\beta_W = 1$ for Class 1 or 2 cross sections

$= W_{el}/W_{pl}$ for Class 3 cross sections

$= W_{eff}/W_{pl}$ for Class 4 cross sections

$W_{pl,y}$ and $W_{pl,z}$ are the plastic moduli for the *y*- and *z*-axes, respectively

$M_{b,Rd}$ is the lateral-torsional buckling resistance

$k_y$, $k_z$, $k_{LT}$ are the interaction factors, as follows:

$$k_y = 1.0 + 2\left(\bar{\lambda}_y - 0.5\right)\frac{N_{Ed}}{N_{b,Rd,y}}$$

where

$$1.2 \le k_y \le 1.2 + 2\frac{N_{Ed}}{N_{b,Rd,y}}$$

$$k_z = 1.0 + 2\left(\bar{\lambda}_z - 0.5\right)\frac{N_{Ed}}{\left(N_{b,Rd}\right)_{min1}}$$

where

$$1.2 \le k_z \le 1.2 + 2\frac{N_{Ed}}{\left(N_{b,Rd}\right)_{min1}}$$

$k_{LT} = 1.0$

Note that in the Eurocodes the *y*-axis is the major axis and the *z*-axis is the minor axis.

### B.8.2 The AISC *Specification* Methodology for Carbon Steel

The AISC *Specification* gives a simple bilinear interaction formula to assess all the possible modes of failure, namely cross-section resistance and member buckling resistance:

(a) When $\dfrac{P_r}{P_c} \ge 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad (\textit{Spec}.\ \text{Eq. H1-1a})$$

(b) When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad (\textit{Spec}.\ \text{Eq. H1-1b})$$

where

$P_r$ = required axial strength using LRFD or ASD load combinations, kips (N)

$P_c$ = available axial strength, kips (N)

$M_r$ = required flexural strength using LRFD or ASD combinations, kip-in. (N-mm)

$M_c$ = available flexural strength, kip-in. (N-mm)

### B.8.3 Recommendations for the AISC Design Guide

Yong et al. (2006) has compared the Eurocode 3 approach with the AISC *Specification* approach and demonstrates that the two standards may appreciably disagree, with differences in predicted resistances as high as 25% for the interaction formulae. For uniform moment distribution with a member slenderness of 0.5, the AISC *Specification* is more conservative than Eurocode 3, but it is the opposite with a

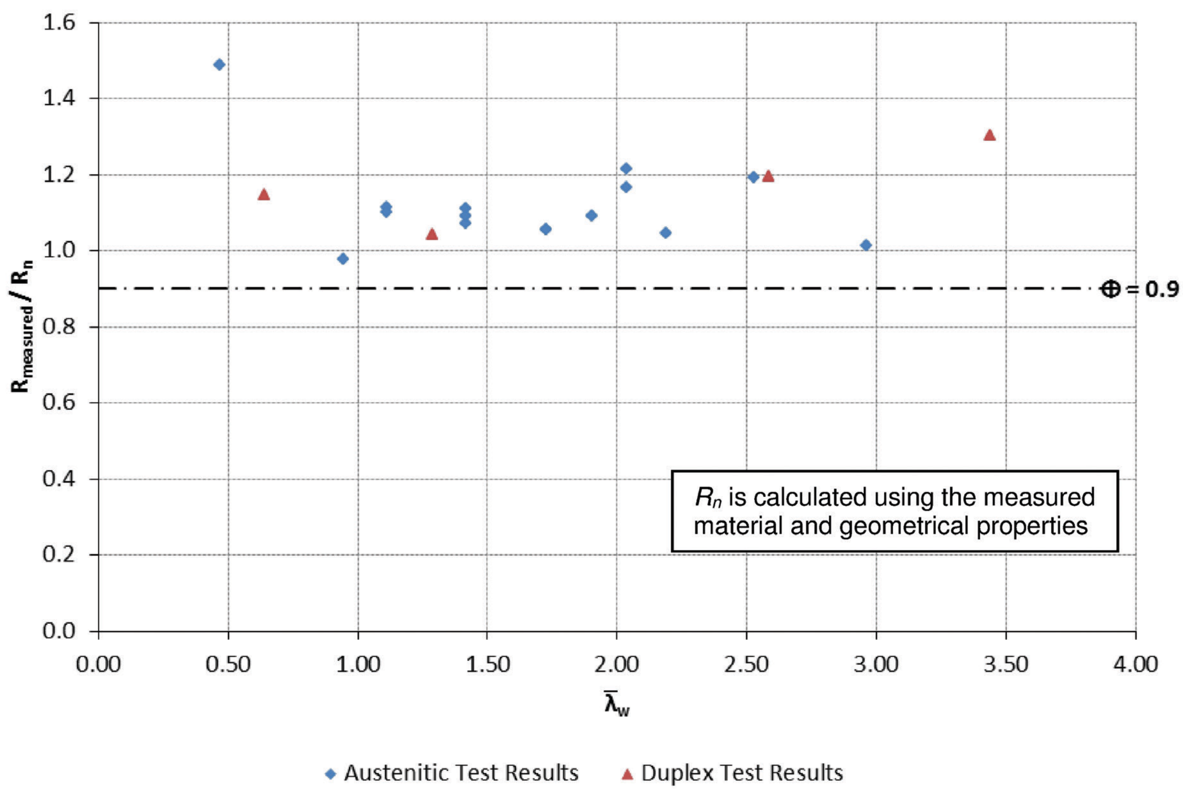



*Fig. B-16. Measured/predicted strengths versus web slenderness for webs subject to shear forces.*

member slenderness of 1.5. For linear moment distribution with reverse end moments, the AISC *Specification* is more conservative.

It is considered that the AISC *Specification* approach is sufficiently accurate to obtain reasonable results in most cases and, given the small number of stainless steel test results, it is recommended that this approach be adopted in Chapter 8 of this Design Guide.

### B.8.4 Determination of Resistance Factor

Test results on round HSS and I-shaped members are reported in the Commentary to the European *Design Manual* (Euro Inox and SCI, 2006b). Additional test results on rectangular HSS are reported in Young and Hartono (2002). The test results have been compared against the AISC *Specification* interaction curve and lie above the curve in all cases.

By comparing the design model with test data, values of $P_m$ = 1.570 and $V_p$ = 0.341 were calculated for austenitic stainless steel. A resistance factor of 1.052 can therefore be derived in accordance with the procedure described in Section B.2. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.9 is recommended. No data were available for duplex stainless steel, therefore, a resistance factor of 0.90 is assumed based upon the results for austenitic stainless steel.

Figure B-17 shows the ratio of measured-to-predicted strength versus member slenderness.

## B.9 DESIGN OF CONNECTIONS

### B.9.1 Design of Welded Connections

#### B.9.1.1 Eurocode 3 and the AISC *Specification* Methodology

In both Eurocode 3, EN 1993-1-8 (CEN, 2005b) and the AISC *Specification*, the fillet weld resistance is determined by using an effective area of weld which is the effective throat thickness multiplied by the effective length of the weld. The guidance on determining this is the same in both codes. Both Eurocode 3 and the AISC *Specification* give a simplified and directional method for calculating the strength of fillet welds. In the simplified method, the main difference between the codes is that the AISC *Specification* considers the angle of loading, whereas Eurocode 3 does not take into account the direction of loading. For the directional methods, the AISC *Specification* uses elastic vector analysis but Eurocode 3 uses the von Mises yield criterion to determine the stress occurring during the load application.

For the design of carbon steel fillet welds to Eurocode 3, the shear strength is determined from

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} \qquad \text{(4.4 of EN 1993-1-8)}$$

where $\beta_w$ is a correlation factor which varies from 0.8 for Grade S235 steel to 0.9 for Grade S355 steel to 1.0 for Grade S420 steel. For stainless steel, it is taken as 1.0 for all stainless steels, unless a lower value is justified by tests. There does not appear to be an equivalent factor in the AISC *Specification*.

#### B.9.1.2 Recommendations for the AISC Design Guide

The AISC approach can be adopted without modification for stainless steel as presented in Chapter 9 of this Design Guide.

#### B.9.1.3 Determination of Resistance Factors for Welded Connections

Two test programs have investigated structural stainless steel welded connections. The first set was carried out at Imperial College in 1991 (SCI, 1991). These tests included 15 specimens in various configurations, including both groove and fillet welds loaded in various directions. However, no mechanical property data were available for these tests, meaning it was impossible to calculate $P_m$ and $V_p$. This test program was, therefore, excluded from any further analysis. The only other data available for welded connections was obtained from Stangenberg (2000b). With the loss of the Imperial College test results, no test data are available for groove welds. The AISC *Specification* resistance factors of 0.75 and 0.80 given in Table J2.5 were therefore adopted.

Fillet welds were analyzed in two populations: welds loaded in the longitudinal direction and welds loaded in the transverse direction. By comparing the design model with test data, and utilizing AISC *Specification* Equation J2-5 to increase the strength in the longitudinal direction, a minimum value for $P_m$ of 0.941 (transverse loading) and an associated $V_p$ equal to 0.033 was calculated for austenitic stainless steel. A resistance factor of 0.571 can therefore be derived in accordance with the procedure described in Section B.2. For duplex stainless steel, values of $P_m$ = 1.017 and $V_p$ = 0.025 were derived. This leads to a resistance factor of 0.619. The resistance factor recommended for this Design Guide is therefore 0.55 for austenitic and 0.60 for duplex stainless steel, as opposed to the higher value of 0.75 for carbon steel in the AISC *Specification*.

Figure B-18 shows the ratio of measured-to-predicted strengths versus measured strength for welded connections in various configurations. It should be noted that there is a higher target reliability index for connections (see Table B-1).

### B.9.2 Design of Bolted Connections

#### B.9.2.1 End Distance, Edge Distance and Spacing

The guidance for carbon steel in EN 1993-1-8 is applicable to stainless steel also. The AISC *Specification* uses different formulae for calculating these distances that lead to larger values of minimum end distance, edge distance, and spacing. Therefore, this guidance can be safely adopted in this Design Guide, as discussed in Section 9.3.

### B.9.2.2 Tension and Shear Strength

The expressions for tension and shear strength of bolts given in Eurocode 3 and the AISC *Specification* are shown in Table B-6.

For M12 to M36 bolts, the ratio of $A_s/A_b$ lies between 0.72 and 0.80. In the table, a value of 0.75 has been assumed. The table shows that Eurocode 3 predicts lower strengths in tension and shear (shear plane through the threaded portion of the bolt, carbon steel bolt Classes 4.8, 5.8, 6.8, 10.9 and stainless steel bolts) than the AISC *Specification*. However, Eurocode 3 predicts higher values of strength in shear (shear plane through the unthreaded portion of the bolt). The codes give the same expression for shear (shear plane through the threaded portion of the bolt) for carbon steel bolt Classes 4.6, 5.6 and 8.8.

The interaction formulae in both codes give similar results. In the interest of simplicity, it is recommended that the AISC rules for carbon steel bolts are applied to stainless steel, with an appropriate resistance factor based on test data, as discussed in Section 9.3.

### B.9.2.3 Bearing Strength at Bolt Holes

The bearing strength predicted by Eurocode 3 is:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}} \qquad \text{(Table 3.4, EN 1993-1-8)}$$

For carbon steel, $f_u$ is the ultimate tensile strength of the connected ply. However, for stainless steel, a reduced value is used in the above expression to limit deformation, $f_{u,red}$ :

$$f_{u,red} = 0.5 f_y + 0.6 f_u \text{ but} \le f_u \qquad \text{(6.1 of EN 1993-1-4)}$$

where

$d$ = bolt diameter

$f_u$ = ultimate tensile strength of the connected ply

$t$ = ply thickness

$\alpha_b$ is the smallest of $\alpha_d$, $f_{ub}/f_{u,red}$, or 1.0

$\alpha_d = e_1/3d_h$ for end bolts in the direction of load transfer

$\alpha_d = \dfrac{p_1}{3d_h} - \dfrac{1}{4}$ for inner bolts in the direction of load transfer

$k_1$ is the smaller of $2.8\dfrac{e_2}{d_h} - 1.7$ or $1.4\dfrac{p_2}{d_h} - 1.7$ or 2.5 for edge bolts perpendicular to direction of load transfer

$k_1$ is the smaller of $1.4\dfrac{p_2}{d_h} - 1.7$ or 2.5 for inner bolts

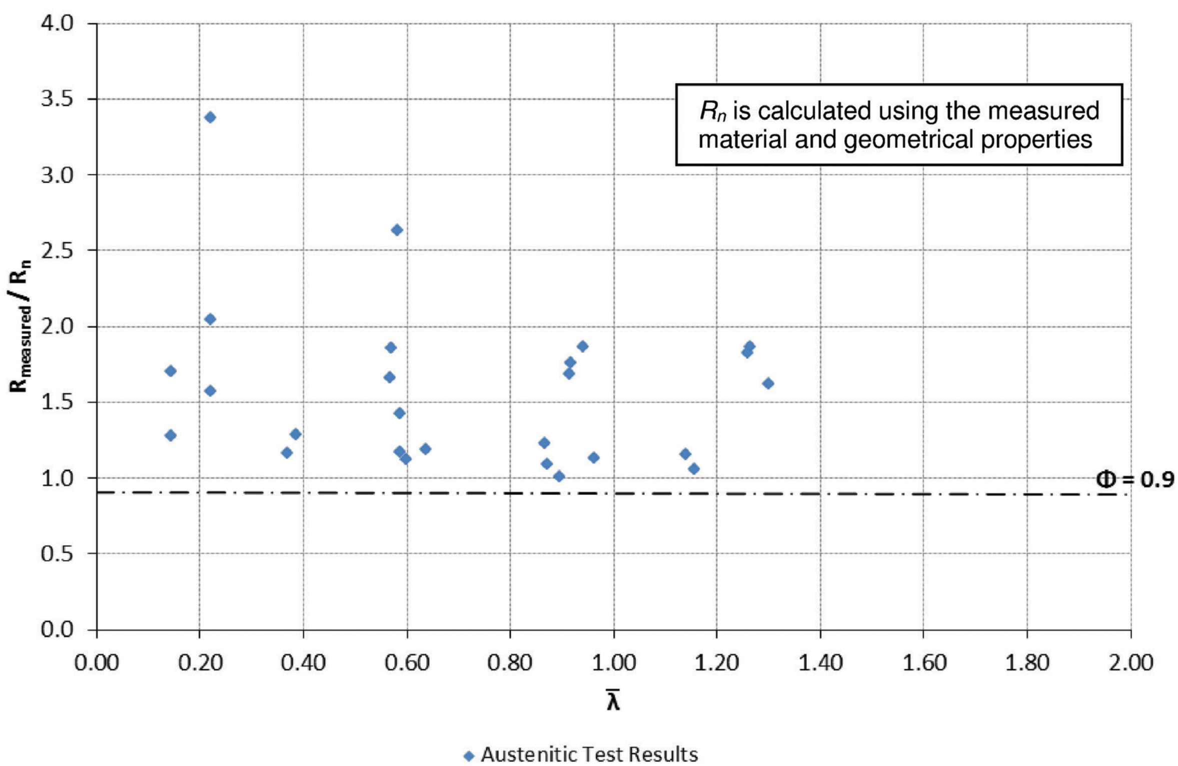



*Fig. B-17. Measured/predicted strengths versus member slenderness for sections subject to combined bending and compression.*

| Table B-6. Expressions for the Strength of Bolts in Shear and Tension | | |
|---|---|---|
| | **Eurocode 3 for Carbon Steel and Stainless Steel Bolts** | **AISC *Specification* for Carbon Steel Bolts** |
| Tension resistance | $R_{nt} = 0.9f_{ub}A_s = 0.68f_{ub}A_b$[a] | $R_{nt} = 0.75F_{ub}A_b$ |
| Shear resistance if the shear plane passes through the unthreaded portion of the bolt | $R_{nv} = 0.6f_{ub}A_b$ | $R_{nv} = 0.55F_{ub}A_b$ |
| Shear resistance if the shear plane passes through the threaded portion of the bolt | Carbon steel bolt Classes 4.6, 5.6, 8.8<br>$R_{nv} = 0.6f_{ub}A_s = 0.45f_{ub}A_b$[a]<br>Carbon steel bolt Classes 4.8, 5.8, 6.8, 10.9 and stainless steel<br>$R_{nv} = 0.5f_{ub}A_s = 0.38f_{ub}A_b$[a] | $R_{nv} = 0.45F_{ub}A_b$ |
| Combined shear and tension | $\frac{V_r}{R_{nv}} + \frac{P_r}{1.4R_{nt}} \leq 1.0$ | $F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}$<br>(by setting $F'_{nt} = f_{rt}$, this expression can be rewritten as:<br>$\frac{f_{rt}}{F_{nt}} + \frac{f_{rv}}{\phi F_{nv}} \leq 1.3$) |

[a] Assuming $A_s \approx 0.75A_b$

Notes:

$A_b$ = nominal unthreaded body area of bolt or threaded part, in.$^2$ (mm$^2$)
$A_s$ = tensile stress area of bolt or threaded part, in.$^2$ (mm$^2$)
$F_{ub}$ = specified minimum ultimate tensile strength of the bolt given in relevant ASTM standard
$F_{nt}$ = available tension strength = $0.75F_{ub}$
$F_{nv}$ = available shear strength = $0.55F_{ub}$ or $0.45F_{ub}$
$f_{rt}$ = required tensile strength (using LRFD and ASD load combinations)
$f_{rv}$ = required shear strength (using LRFD or ASD load combinations)
$f_{ub}$ = specified minimum ultimate tensile strength of the bolt (Eurocode 3)

perpendicular to the direction of load transfer

$f_{ub}$ = specified minimum ultimate tensile strength of the bolt (Eurocode 3)

And the other terms are defined in Section 9.3.6.

Since the Eurocode rules were written, further work has been carried out investigating the bearing strength of stainless steel connections (Salih et al., 2011). Whereas the load deformation curve for carbon steel connections flattens off after the initiation and spreading of yielding, for stainless steel connections this curve continues to rise significantly owing to strain hardening. For this reason, greater clarity in defining bearing capacity than has previously been used when considering carbon steel connections was necessary. Different failure definitions were devised for stainless steel connections, and bearing design equations for both thick and thin material that cover two cases (one restricting and one ignoring serviceability deformations) were proposed. These equations define the bearing capacity in terms of the material ultimate strength instead of the so-called reduced ultimate strength, $f_{u,red}$, and therefore are consistent with the provisions for carbon steel connections.

The proposed equations provide a modest enhancement in capacity compared to the current Eurocode approach, as well as being simpler to use. The recommendations for thick material given in Salih et al. (2011) have been included in Section 9.3.6 of this Design Guide, and are also very likely to be introduced into EN 1993-1-4 in the next revision.

Figures B-19 and B-20 compare the expressions in Section 9.3.6a (i) and (ii), respectively, with the results of numerical analyses. In these figures the hole diameter is denoted as $d_0$.

### B.9.2.4 Determination of Resistance Factors for Bolted Connections

Data for this analysis was obtained from Ryan (2000) and Salih (2010).

Taking into account all of the possible failure mechanisms for bolted connections (i.e., bolt failure in tension, bolt failure in shear, bearing failure, and net section failure) comparisons of the design models against the test data were carried out. The analyses demonstrated that a resistance factor of 0.75 was applicable, consistent with the AISC *Specification.*

For duplex stainless steel, the only test results available concern bolt shear and tension rupture; this is not comprehensive enough for a full analysis. A resistance factor of 0.75 is therefore specified, based on the results for austenitic stainless steel.

Figure B-21 shows the ratio of measured-to-predicted strength versus measured strength for various configurations of bolted connections.

### B.9.3 Affected Elements of Members and Connecting Elements

In EN 1993-1-8, the design value for block shear (termed "block tearing" in the standard) for symmetric bolt groups subject to concentric loading, is determined from the equation:

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\frac{f_y A_{nv}}{\gamma_{M0}} \qquad \text{(3.9 of EN 1993-1-8)}$$

where

$A_{nt}$ = net area subject to tension

$A_{nv}$ = net area subject to shear

The equation used in the AISC *Specification* for the available strength for the limit state of block shear rupture along a shear failure path or path(s) and a perpendicular tension failure path is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \qquad \text{(}Spec.\text{ Eq. J4-5)}$$

where $U_{bs}$ = 1.0 when the tension stress is uniform (angles, gusset plates, and most coped beams), and $U_{bs}$ = 0.5 when the tension stress is nonuniform.

There are no specific data for stainless steel in shear rupture and the omission of special rules in EN 1993-1-4 implies that the carbon steel guidance in EN 1993-1-8 applies. Therefore, the AISC carbon steel provisions for block shear strength in Section J4.3 may be adopted for stainless steel as stipulated in Section 9.4.

### B.9.4 Bearing Strength

In the absence of test data for stainless steel, the guidance for carbon steel pins in the AISC *Specification* Section J7 may be applied to stainless steel for determining the strength of surfaces in contact, as stipulated in Section 9.5 (finished surfaces, pins in reamed, drilled or bored holes, and ends of fitted bearing stiffeners).

### B.9.5 Flanges and Webs with Concentrated Forces

The European *Design Manual* (Euro Inox and SCI, 2006a) describes the tests carried out on austenitic stainless steel plate girders to study the resistance of webs to concentrated forces (Selen, 1999). The work confirmed that the guidance

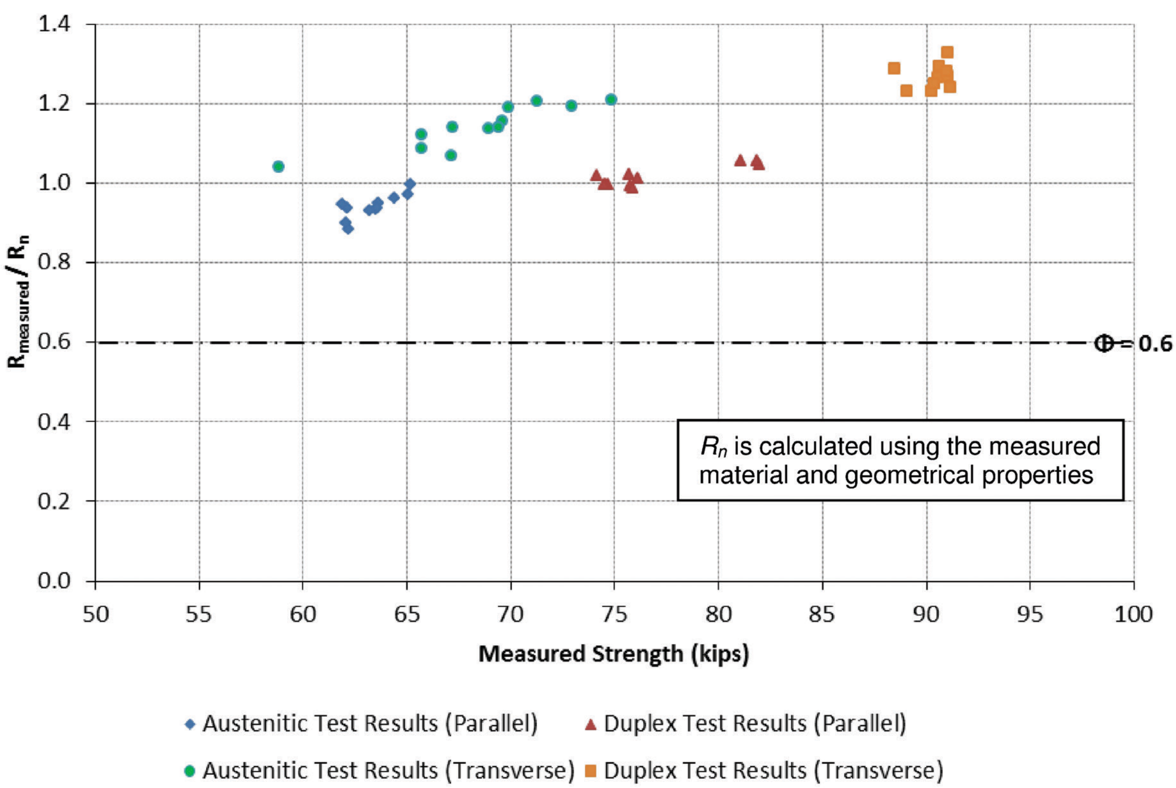



*Fig. B-18. Measured/predicted strengths versus measured strength for welded connections.*

for carbon steel plate girders in EN 1993-1-5 (CEN, 2006b) can be applied to stainless steel also. Comparing the test data with the AISC provisions also showed that the AISC carbon steel rules and resistance factors may be adopted for stainless steel.

Figure B-22 shows the ratio of measured-to-predicted strengths versus web depth-to-thickness, $h/t_w$, for flanges and webs with concentrated forces.

## B.10 STRUCTURAL DESIGN FOR FIRE CONDITIONS

### B.10.1 Mechanical and Thermal Properties at Elevated Temperatures

Annex C of the Eurocode for structural fire resistance, EN 1993-1-2 (CEN, 2005c), extends the rules for carbon steel to cover stainless steel, and gives some supporting material and mechanical properties at elevated temperatures. The data for the specific heat and emissivity in this Design Guide is taken from EN 1993-1-2 Annex C. However, the data for the thermal elongation of austenitic and duplex stainless steels in this Design Guide is taken from SEW (1992) (the expression in EN 1993-1-2 covers austenitic stainless steels only, and contains an error). The data for the precipitation hardening stainless steel is taken from Part 1 of EN 10088 (CEN, 2005d).

Since the Eurocode was prepared, many further isothermal and anisothermal tensile tests have been carried out on a wide range of stainless steels. Gardner et al. (2010) analyzes all available data and proposes six sets of strength retention factors that apply to different groups of stainless steels. It is expected that these factors will be included in the next version of EN 1993-1-2. The data given in Table 10-2 to Table 10-5 are taken from Gardner et al. (2010).

### B.10.2 Compression Members

#### B.10.2.1 European Design Manual for Structural Stainless Steel Methodology

As mentioned in Section B.1.3, the guidance on fire resistant design for stainless steel in EN 1993-1-2 is conservative, and the European *Design Manual for Structural Stainless*

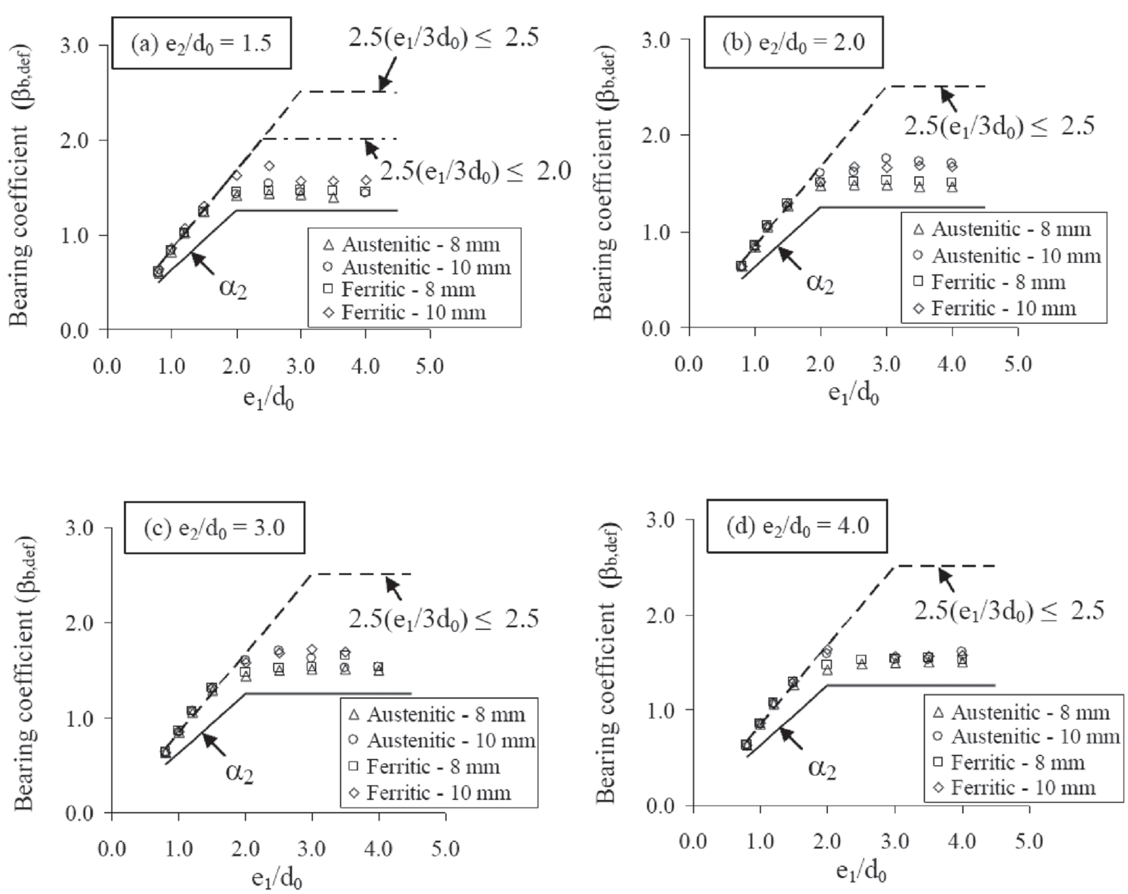



*Fig. B-19. Bearing coefficient for thick plates (8 and 10 mm) (5⁄16 and 3⁄8 in.) from parametric studies when deformation at the bolt hole at service load is a design consideration.*

*Steel* gives less conservative design rules, based on a larger body of test data (Euro Inox and SCI, 2006b). The *Design Manual* gives an expression for calculating the compression resistance of a member in fire that is based on the room temperature flexural buckling curve rather than a special fire buckling curve, which is the approach taken in EN 1993-1-2 for carbon and stainless steel. The *Design Manual* states that the design buckling resistance, $N_{b,fi,tRd}$, at time $t$ of a compression member with a uniform temperature, $\theta_a$, for nonslender sections is given by:

$$N_{b,fi,tRd} = \frac{\chi_{fi} A k_{0.2proof,\theta} f_y}{\gamma_{M,fi}}$$

(European *Design Manual* Eq. 7.8)

where

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}} \leq 1.0$$

(European *Design Manual* Eq. 7.10)

$$\varphi_\theta = 0.5\left[1 + \alpha\left(\bar{\lambda}_\theta - \bar{\lambda}_0\right) + \bar{\lambda}_\theta^2\right]$$

(European *Design Manual* Eq. 7.11)

$$\bar{\lambda}_\theta = \sqrt{\frac{Af_y}{N_{cr}}} \times \left[\frac{k_{0.2proof,\theta}}{k_{E,\theta}}\right]^{0.5}$$

(European *Design Manual* Eq. 7.12)

The imperfection factor, $\alpha$, and limiting slenderness factor, $\bar{\lambda}_0$, are those for room temperature design given in Section B.5.1.1.

**B.10.2.2 The AISC *Specification* Methodology for Carbon Steel**

The nominal compressive strength at a temperature $T$, $P_n(T)$, is determined as:

$$P_n(T) = F_{cr}(T) A_g$$

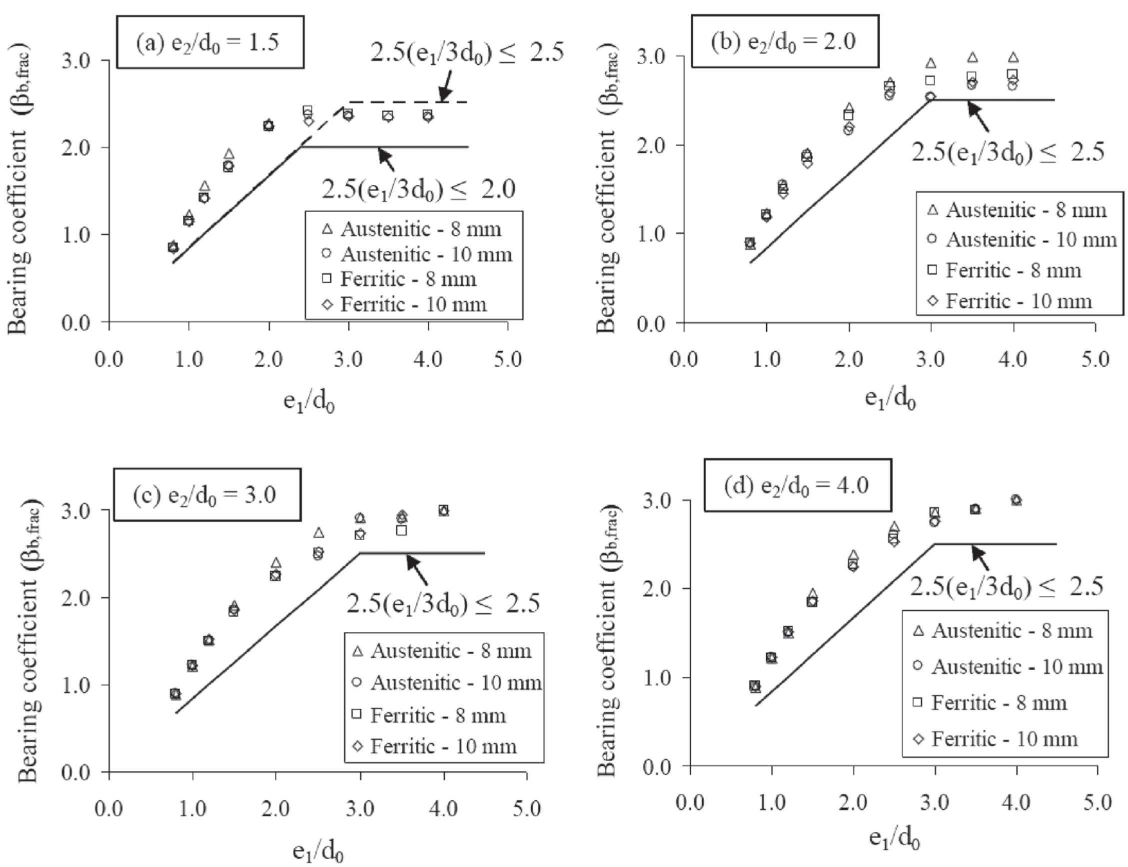



*Fig. B-20. Bearing coefficient for thick plates (8 and 10 mm) (5⁄16 and 3⁄8 in.) from parametric studies when deformation at the bolt hole at service load is not a design consideration.*

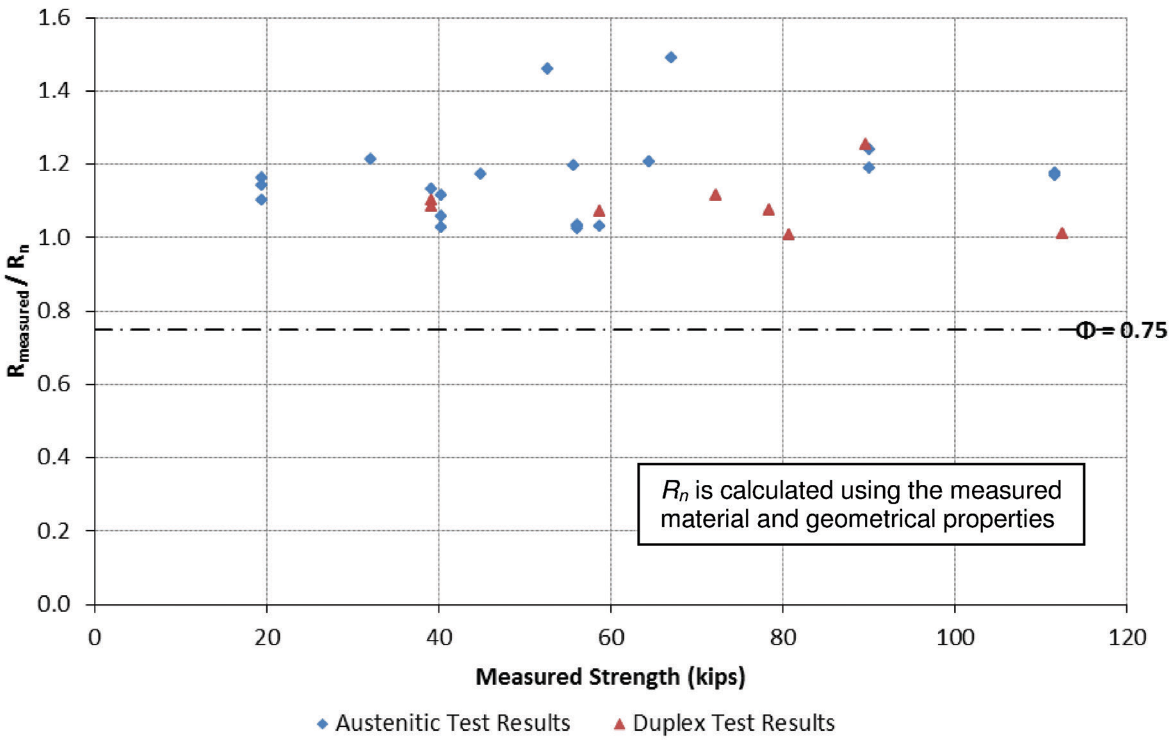



*Fig. B-21. Measured/predicted strengths versus measured strength for bolted connections.*

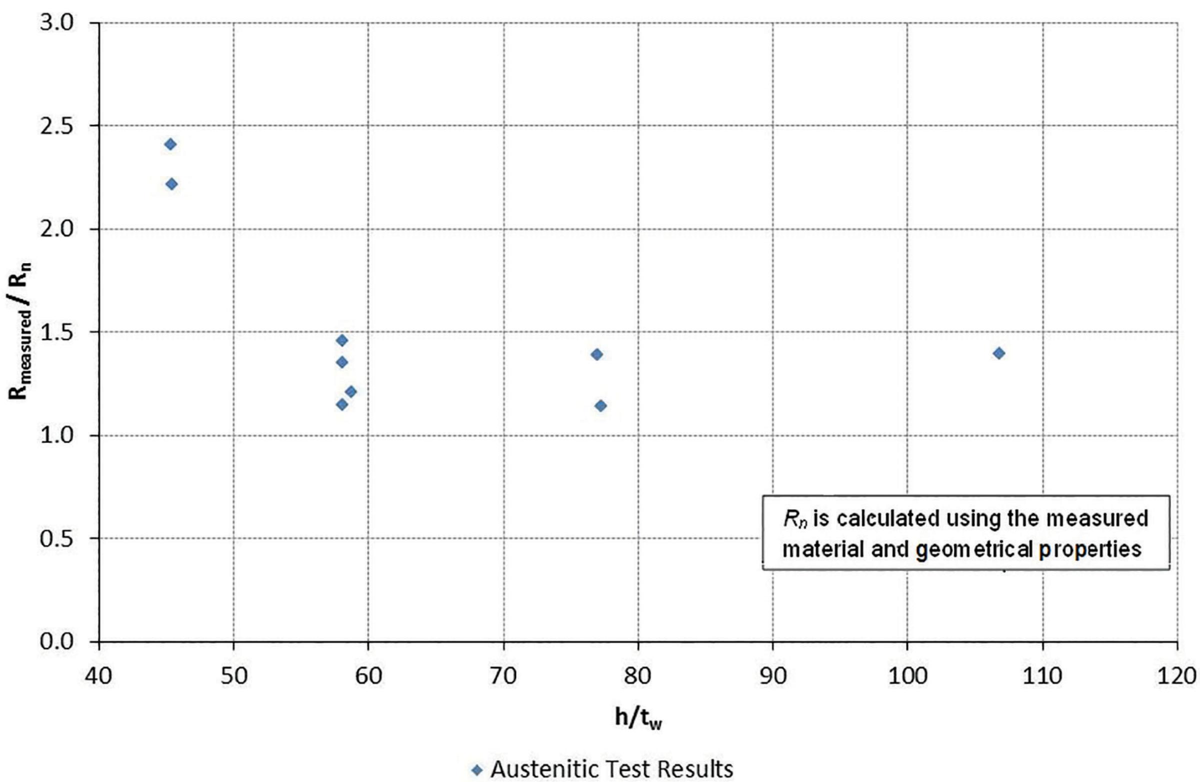



*Fig. B-22. Measured/predicted strength versus* $h/t_w$ *for flanges and webs with concentrated forces.*

where

$$F_{cr}(T) = \left[0.42^{\sqrt{F_y(T)/F_e(T)}}\right] F_y(T) \qquad (Spec.\ \text{Eq. A-4-2})$$

where $F_y(T)$ is the yield stress at elevated temperature and $F_e(T)$ is the critical elastic buckling stress calculated from Equation E3-4 of the AISC *Specification* with the elastic modulus $E(T)$ at elevated temperature.

### B.10.2.3 Recommendations for the AISC Design Guide

Figure B-23 compares the strength retention of stainless steels, tabulated in Table 10-2 to Table 10-5 of this Design Guide, with those for carbon steel given in Table A-4.2.1 of the AISC *Specification.* Stainless steel loses strength to a greater extent at temperatures up to about 1,100 °F (600 °C) but the austenitic stainless steels retain strength better than carbon steel above this limit. Austenitic and duplex stainless steels retain stiffness better than carbon steel at all temperatures above 400 °F (200 °C), as shown in Figure B-24.

As temperature rises, the strength-to-stiffness ratio for stainless steel decreases, whereas for carbon steel this ratio increases with temperature. The impact of this is that the expression $0.42[F_y(T)/F_e(T)]^{0.5}$ increases for stainless steel with increasing temperature but decreases for carbon steel (Figure B-25).

The variation of $F_{cr}(T)$ with temperature in AISC *Specification* Equation A-4-2 for stainless and carbon steel is shown in Figure B-26, which demonstrates the combined effect of the variation of strength-to-stiffness ratio $0.42[F_y(T)/F_e(T)]^{0.5}$ multiplied by the strength $F_y(T)$ with temperature. In the graph, $F_{cr}(T)$ is calculated for a carbon steel and stainless steel column of identical cross section and length, with the carbon steel column made from steel with $F_y = 36$ ksi and the stainless steel column made from Type S30400. Even though the carbon steel column is stronger at room temperature, at temperatures above approximately 1,000 °F (550 °C), the stainless steel column retains a higher compressive flexural buckling strength.

In the process of deriving guidance for this Design Guide, two buckling curves for calculating the strength of stainless steel columns in fire were compared:

- The fire flexural buckling curve (Equation A-4-2 in the AISC *Specification*)
- The room temperature AISC flexural buckling curve for stainless steel (modified AISC *Specification* Equations E3-2 and E3-3)

In both cases the elevated temperature properties for stainless steel were used.

Figures B-27 to B-30 show the variation of the ratio $F_{cr}(T)/F_y(T)$ with temperature at 600 °F (315 °C), 1,000 °F (538 °C), 1,400 °F (760 °C), and 1,800 °F (982 °C), respectively, for these approaches. The fire buckling curves in the European *Design Manual for Structural Stainless Steel* are shown for comparison purposes and Table B-7 compares the failure temperatures predicted by these two approaches with tests by Baddoo and Gardner (2000) and Ala-Outinen (1996). The AISC fire buckling curve for carbon steel gave very conservative results. It was therefore decided to recommend that for stainless steel columns in fire, the flexural buckling strength is calculated from the room temperature buckling curves, using the strength and stiffness at elevated temperatures.

It should be noted that the columns tested in Ala-Outinen (1996) were made from cold-worked stainless steel, with yield strengths of nearly 600 MPa (87 ksi). The strength retention characteristics of cold-worked stainless steel differ from those for annealed stainless steel; the strength is retained up to about 400 °C (750 °F) and then there is a steep reduction in strength. Therefore, the use of the strength reduction factors in Tables 10-2 to 10-5 is not really applicable in this case. However, the results are included for comparative purposes in Table B-7.

## B.10.3 Flexural Members

### B.10.3.1 European *Design Manual* for Structural Stainless Steel Methodology

The European *Design Manual* determines the buckling resistance moment from the room temperature buckling curve with the material factors at elevated temperatures in the same way that the room temperature flexural buckling curve is used for columns in fire.

The design buckling resistance moment, $M_{b,fi,t,Rd}$, at time $t$ of a laterally unrestrained beam should be determined from:

For Class 1 and 2 sections

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} k_{0.2proof,\theta} f_y / \gamma_{M,fi}$$

(European *Design Manual* Eq. 7.18)

For Class 3 sections

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{el,y} k_{0.2proof,\theta} f_y / \gamma_{M,fi}$$

(European *Design Manual* Eq. 7.19)

For Class 4 sections

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{eff,y} k_{0.2proof,\theta} f_y / \gamma_{M,fi}$$

(European *Design Manual* Eq. 7.20)

where

$\chi_{LT,fi}$ = reduction factor for lateral-torsional buckling in fire

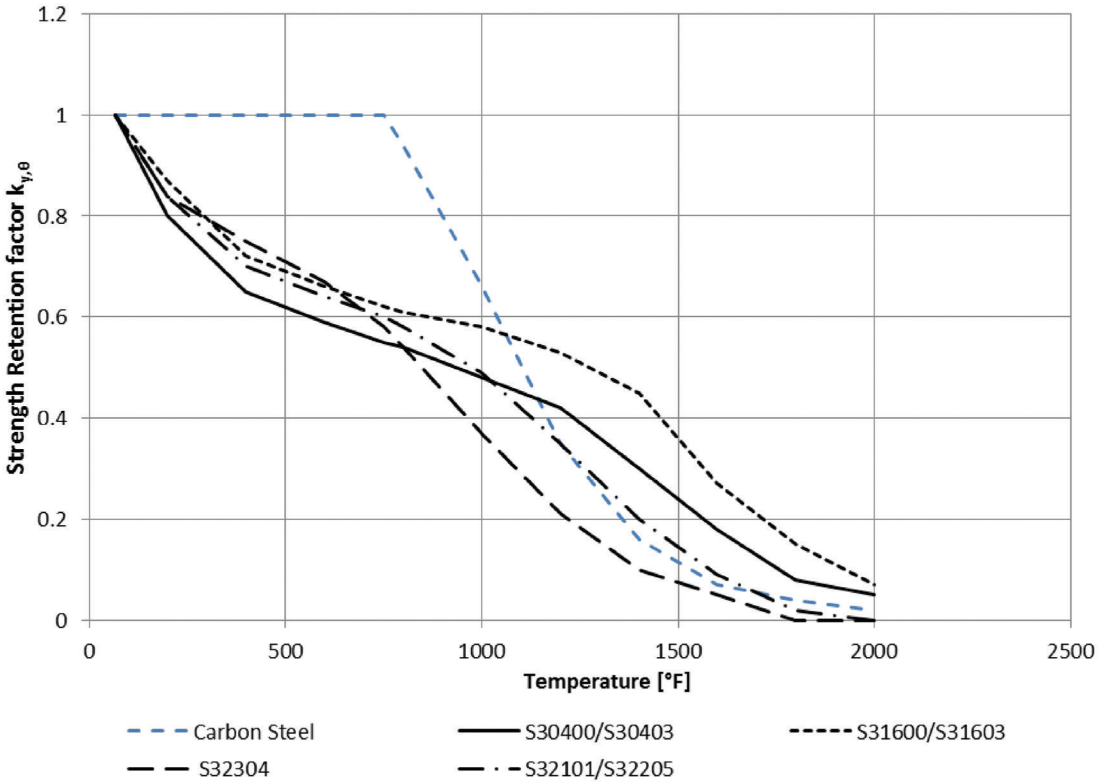



*Fig. B-23. Comparison of stainless steel and carbon steel strength retention factors.*

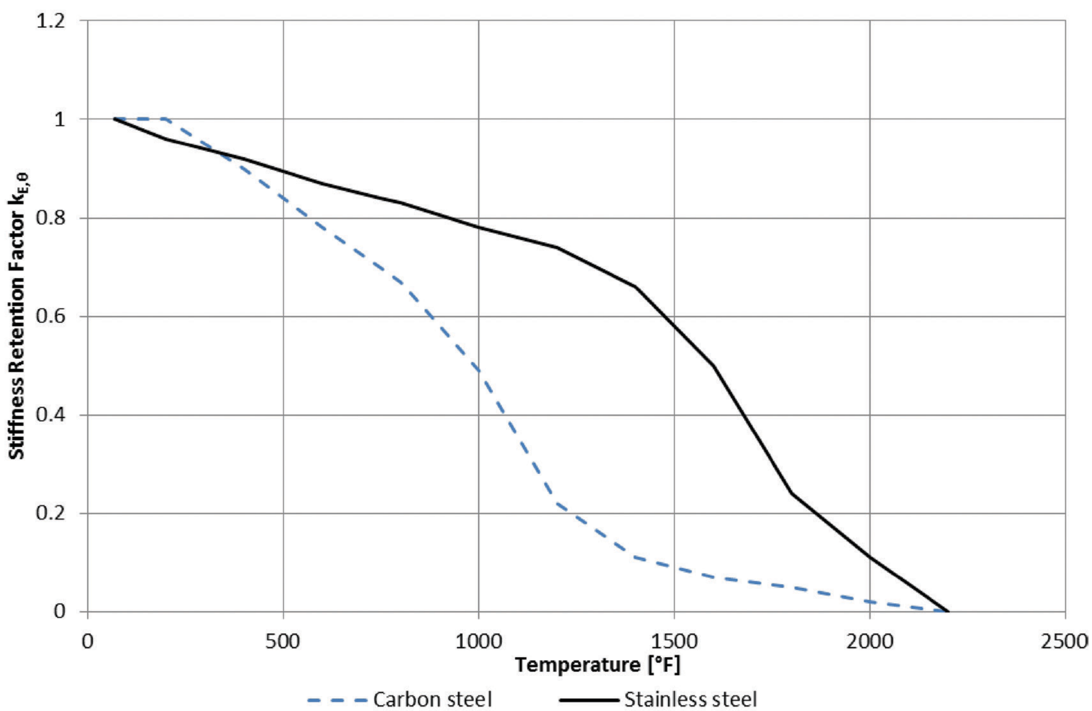



*Fig. B-24. Comparison of stainless steel and carbon steel stiffness retention factors.*

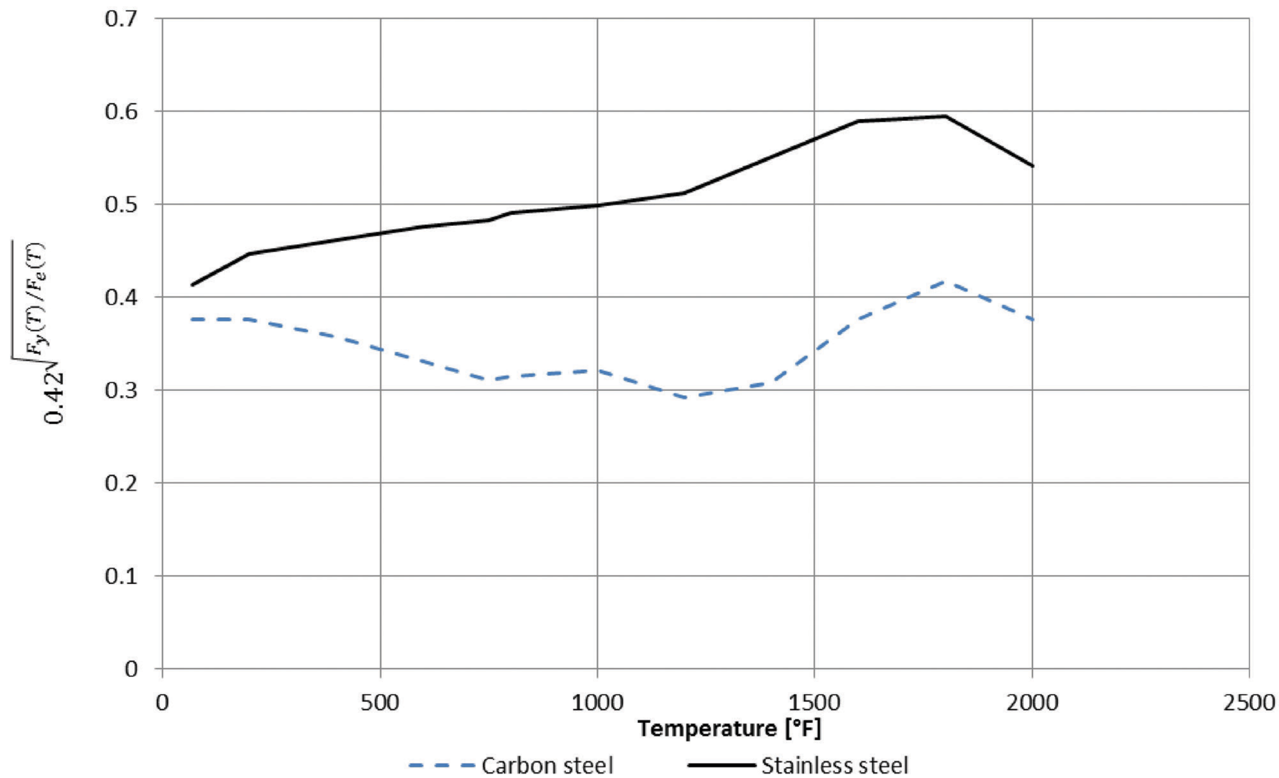



*Fig. B-25. Comparison of* $0.42[F_y(T)/F_e(T)]^{0.5}$ *factor for carbon and stainless steel.*

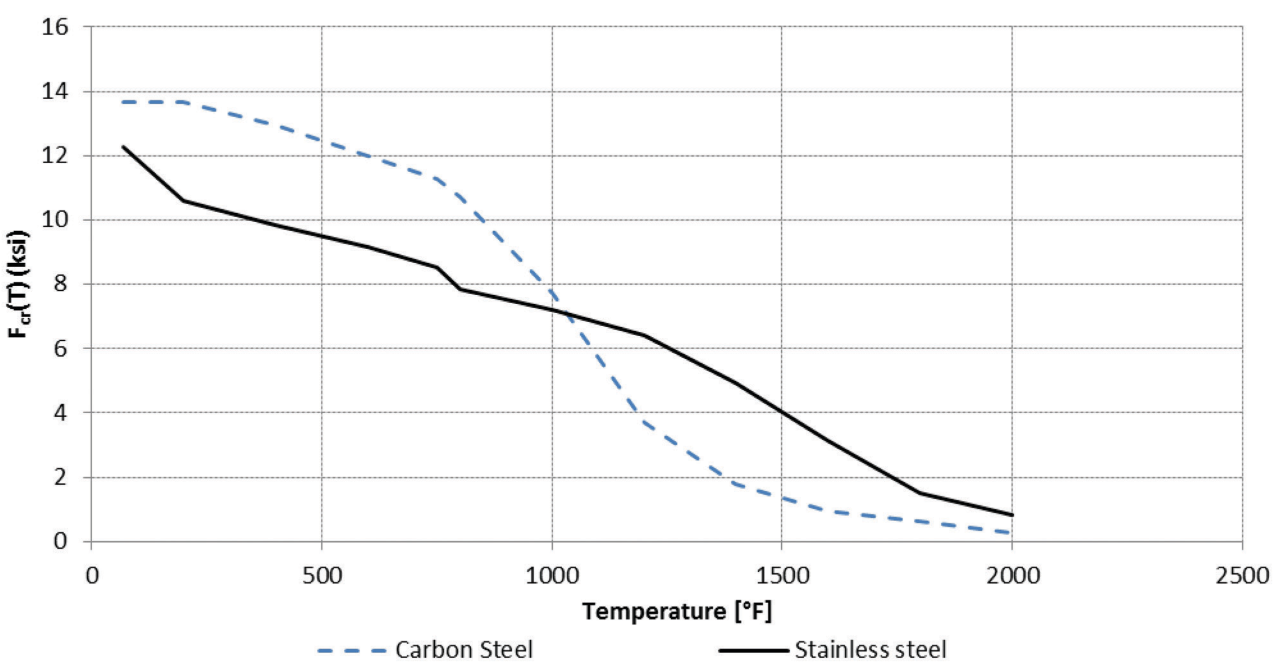



*Figure B-26. Comparison of* $F_{cr}(T)$ *for carbon and stainless steel.*

| Table B-7. Comparison Between Proposed Method and Test Data for Columns in Fire Conditions | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Test Results: Baddoo and Gardner (2000)** | | | | | | | | | | |
| Section Type | Section | End Conditions | $K$ | Area | $F_{y,amb}$ | Length | Applied Load | $T_{test}$ | $T_{pred}$[a] | $T_{pred}$[b] |
| | | | | in.$^2$ | ksi | in. | kips | °F | °F | °F |
| HSS | 150x100x6 | Fix | 0.5 | 4.42 | 37.99 | 133.86 | 60.25 | 1473 | 800 | 1200 |
| HSS | 150x75x6 | Fix | 0.5 | 3.96 | 37.99 | 133.86 | 31.47 | 1621 | 1200 | 1400 |
| HSS | 100x75x6 | Fix | 0.5 | 3.06 | 37.99 | 133.86 | 35.07 | 1482 | 1000 | 1200 |
| HSS | 100x100x4 | Pin | 1.0 | 2.27 | 43.21 | 157.09 | 17.98 | 1535 | 1200 | 1400 |
| **Test Results: Ala-Outinen (1996)** | | | | | | | | | | |
| Section Type | Section | End Conditions | $K$ | Area | $F_{y,amb}$ | Length | Applied Load | $T_{test}$ | $T_{pred}$[a] | $T_{pred}$[b] |
| | | | | in.$^2$ | ksi | in. | kips | °F | °F | °F |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.98 | 10.12 | 1603 | 1400 | 1400 |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.98 | 29.00 | 1074 | <200 | 200 |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.96 | 25.63 | 1200 | 200 | 400 |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.96 | 21.36 | 1310 | 200 | 1000 |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.96 | 12.36 | 1529 | 1200 | 1400 |
| HSS | 40x40x4 | Pin | 1.0 | 0.87 | 85.84 | 34.98 | 16.86 | 1410 | 1000 | 1200 |

[a] $T_{pred}$ determined from AISC carbon steel fire flexural buckling curve, with stainless steel $F_y(T)$ and $F_e(T)$
[b] $T_{pred}$ determined from AISC room temperature buckling curve for stainless steel, with stainless steel $F_y(T)$ and $F_e(T)$

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta} + \sqrt{\phi_{LT,\theta}^2 - \bar{\lambda}_{LT,\theta}^2}} \quad \text{but } \chi_{LT,fi} \le 1$$

(European *Design Manual* Eq. 7.21)

$$\phi_{LT,\theta} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT,\theta} - 0.4\right) + \bar{\lambda}_{LT,\theta}^2\right]$$

(European *Design Manual* Eq. 7.22)

$\alpha_{LT}$ = room temperature imperfection factor given in Section B.6.2.1

$k_{0.2proof,\theta}$ = retention factor at the maximum temperature, θ, reached anywhere in the section

The nondimensional slenderness, $\bar{\lambda}_{LT,\theta}$, at temperature θ is given by:

$$\bar{\lambda}_{LT,\theta} = \bar{\lambda}_{LT}\left[\frac{k_{0.2\,proof,\theta}}{k_{E,\theta}}\right]^{0.5}$$

(European *Design Manual* Eq. 7.23)

where all the terms are described in the European *Design Manual.*

### B.10.3.2 The AISC *Specification* Methodology for Carbon Steel

The AISC methodology for the design of laterally unbraced, doubly symmetric members for fire conditions is as follows:

(a) When $L_b \le L_r(T)$

$$M_n(T) = C_b\left\{M_r(T) + \left[M_p(T) - M_r(T)\right]\left[1 - \frac{L_b}{L_r(T)}\right]^{c_x}\right\}$$

(*Spec*. Eq. A-4-3)

(b) When $L_b > L_r(T)$

$$M_n(T) = F_{cr}(T)\,S_x \qquad \text{(Spec. Eq. A-4-4)}$$

where

$$F_{cr}(T) = \frac{C_b\pi^2 E(T)}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

(*Spec*. Eq. A-4-5)

$$L_r(T) = 1.95 r_{ts}\frac{E(T)}{F_L(T)}\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left[\frac{F_L(T)}{E(T)}\right]^2}}$$

(*Spec*. Eq. A-4-6)

$$M_r(T) = S_x F_L(T) \qquad \text{(Spec. Eq. A-4-7)}$$

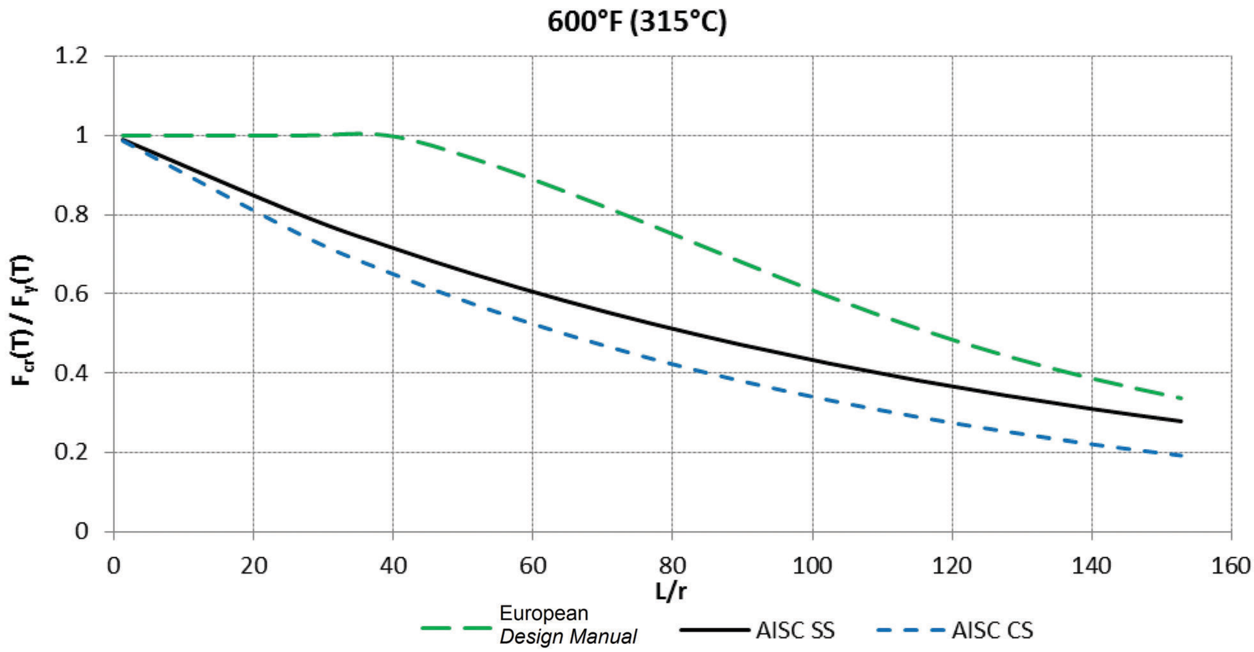



*Fig. B-27. AISC flexural buckling curves at 600 °F (315 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve.*

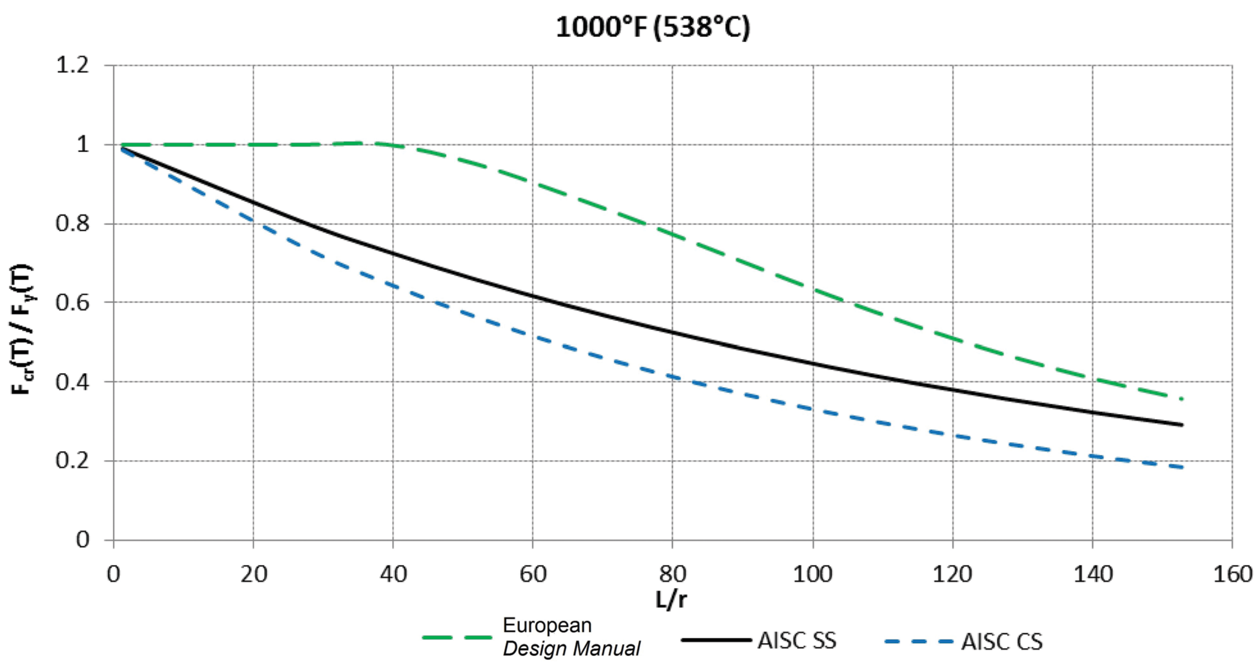



*Fig. B-28. AISC flexural buckling curves at 1,000 °F (538 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve.*

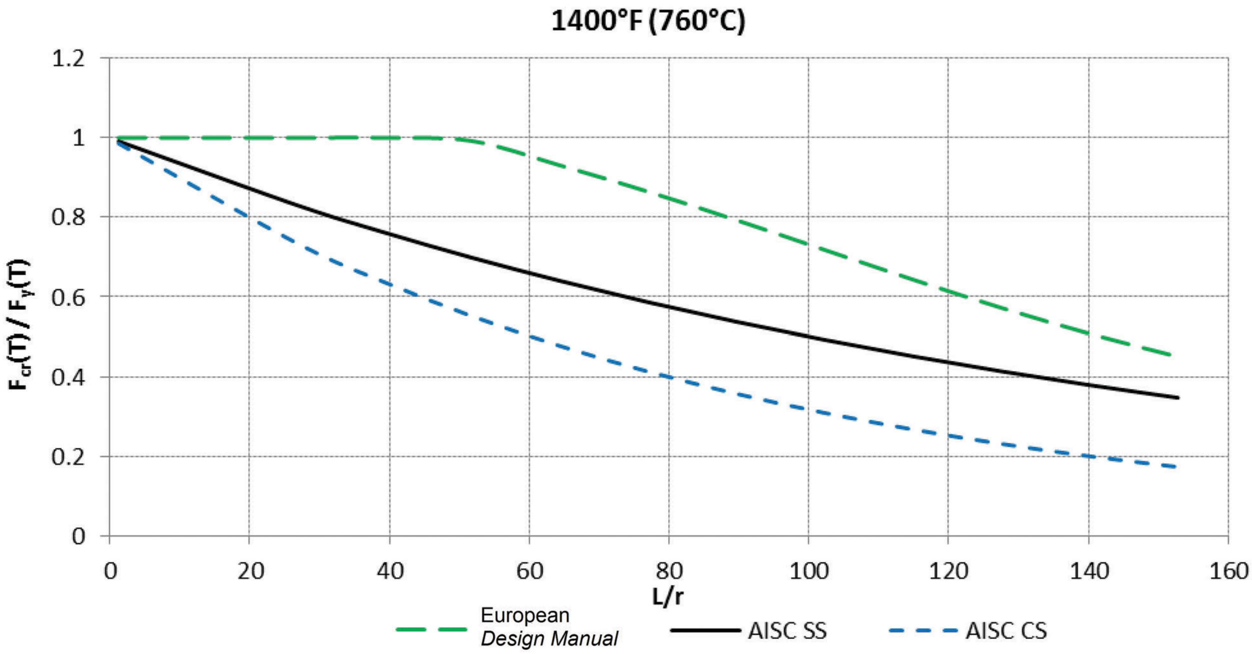



*Fig. B-29. AISC flexural buckling curves at 1,400 °F (760 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve.*

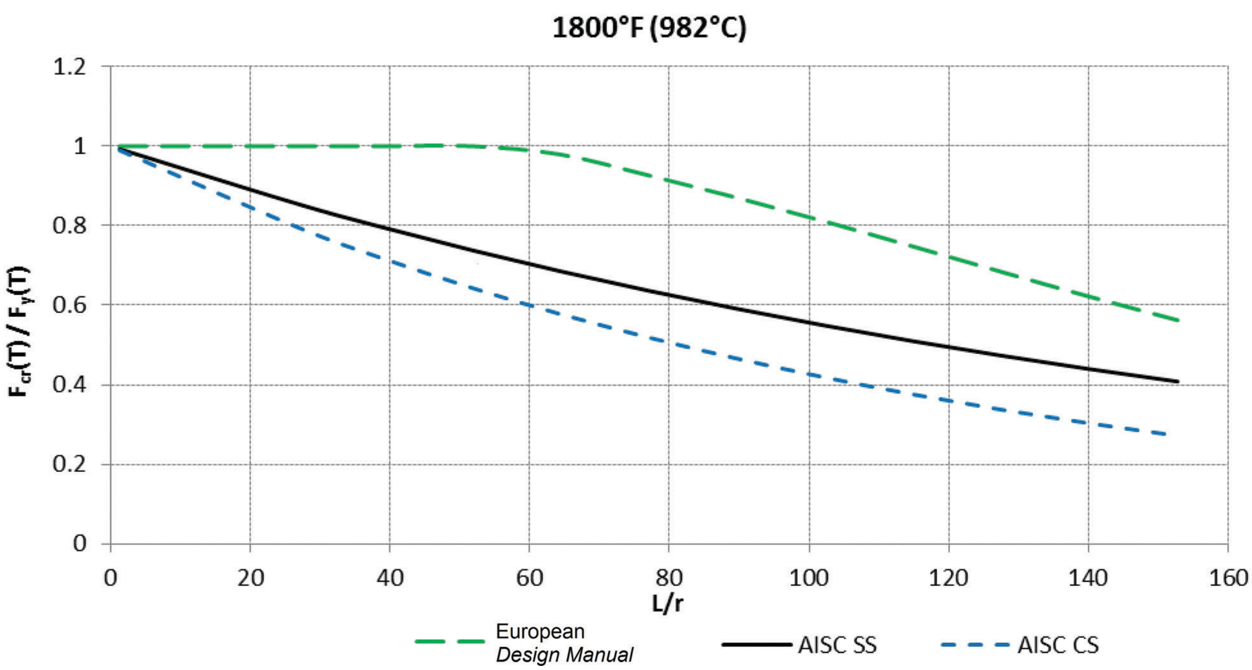



*Fig. B-30. AISC flexural buckling curves at 1,800 °F (982 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve.*

$$F_L(T) = F_y(k_p - 0.3k_y) \qquad (\textit{Spec}.\text{ Eq. A-4-8})$$

$$M_p(T) = Z_x F_y(T) \qquad (\textit{Spec}.\text{ Eq. A-4-9})$$

$$c_x = 0.53 + \frac{T}{450} \le 3.0 \quad \text{where } T \text{ is in °F}$$

$$(\textit{Spec}.\text{ Eq. A-4-10})$$

where the terms are all defined in the AISC *Specification*.

#### B.10.3.3 Recommendations for this Design Guide

There is a lack of experimental data relating to the behavior of unrestrained stainless steel beams under fire conditions, but numerical analysis has been carried out by Vila Real et al. (2008) and on the basis of this data, modified expressions for stainless steel have been derived for inclusion in this Design Guide. Note that the parameter, $F_p(T)$, the proportional limit at elevated temperatures, is not available for stainless steel and so the strength parameter, $F_L$, cannot be calculated. The stainless steel expressions use $F_y$ instead of $F_L$.

(a) When $L_b \le L_r(T)$, use AISC *Specification* Equation A-4-3

(b) When $L_b > L_r(T)$

$$M_n(T) = 0.4F_{cr}(T)S_x \qquad (\text{modified } \textit{Spec}.\text{ Eq. A-4-4})$$

where

$F_{cr}(T)$ is given by AISC *Specification* Equation A-4-5

$$L_r(T) = 1.95 r_{ts}\frac{E(T)}{F_y(T)} \times \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left[\frac{F_y(T)}{E(T)}\right]^2}}$$

$$(\text{modified } \textit{Spec}.\text{ Eq. A-4-6})$$

$$M_r(T) = 0.4F_y(T)S_x \qquad (\text{modified } \textit{Spec}.\text{ Eq. A-4-7})$$

$M_p(T)$ is given by AISC *Specification* Equation A-4-9 and $c_x$ is given by AISC *Specification* Equation A-4-10.

Figure B-31, Figure B-32 and Figure B-33 show these modified AISC design expressions for stainless steel compared with the European *Design Manual* (EC SS) expressions and the available numerical data at 752 °F, 1,112 °F and 1,292 °F, respectively.

## B.11 CONTINUOUS STRENGTH METHOD

A new design approach called the continuous strength method (CSM) has been developed over a number of years, and provisions are given in Appendix A. It is a deformation-based design approach that incorporates strain hardening, and is applicable to I-shaped members and rectangular HSS of low member slenderness. When compared with test results on stainless steel stub columns and beams, the predictions from the CSM offer improved mean resistance and reduced scatter compared to the design provisions of Sections 5 and 6 of this Design Guide, which are known to be rather conservative. The relevant technical background is provided in detail in Gardner (2008), Ashraf et al. (2008), Gardner and Theofanous (2008), and Gardner et al. (2011). The technical basis for the method has been thoroughly assessed in the reviews of these papers and within the Evolution Group for EN 1993-1-4. It is very likely that this method will be introduced into EN 1993-1-4 as an alternative design method in a future revision.

### B.11.1 Determination of Resistance Factors for Continuous Strength Method (Compression Members)

A separate analysis was required to determine the reliability of the CSM for predicting compressive strength, in accordance with the method described in Appendix A, Section A.4. The data used in this analysis were a subset of the data referred to in Section B.5.1.3 that met the conditions of applicability of the CSM.

Comparison of the CSM design model against these test data indicates a value of $P_m$ = 1.115 and $V_p$ = 0.104 for austenitic stainless steel. In accordance with the procedure described in B.2, a resistance factor of 1.13 can therefore be calculated. For duplex stainless steel, a value of $P_m$ = 1.089 and $V_p$ = 0.103 were determined. A resistance factor of 0.98 can therefore be calculated. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended for both cases.

Figure B-34 plots the ratio of measured-to-predicted strengths versus cross-section slenderness. One test point falls beneath the $\phi_c$ = 0.90 line, which is statistically acceptable.

### B.11.2 Determination of Resistance Factors for Continuous Strength Method (Flexural Members)

A separate analysis was required to determine the reliability of the CSM for predicting flexural strength, in accordance with the method described in Appendix A, Section A.5. The data used in this analysis were a subset of the data referred to in Section B.6.1.4 that met the conditions of applicability of the CSM.

Comparison of the CSM design model against these test data indicates a value of $P_m$ = 1.175 and $V_p$ = 0.103 for austenitic stainless steel. In accordance with the procedure

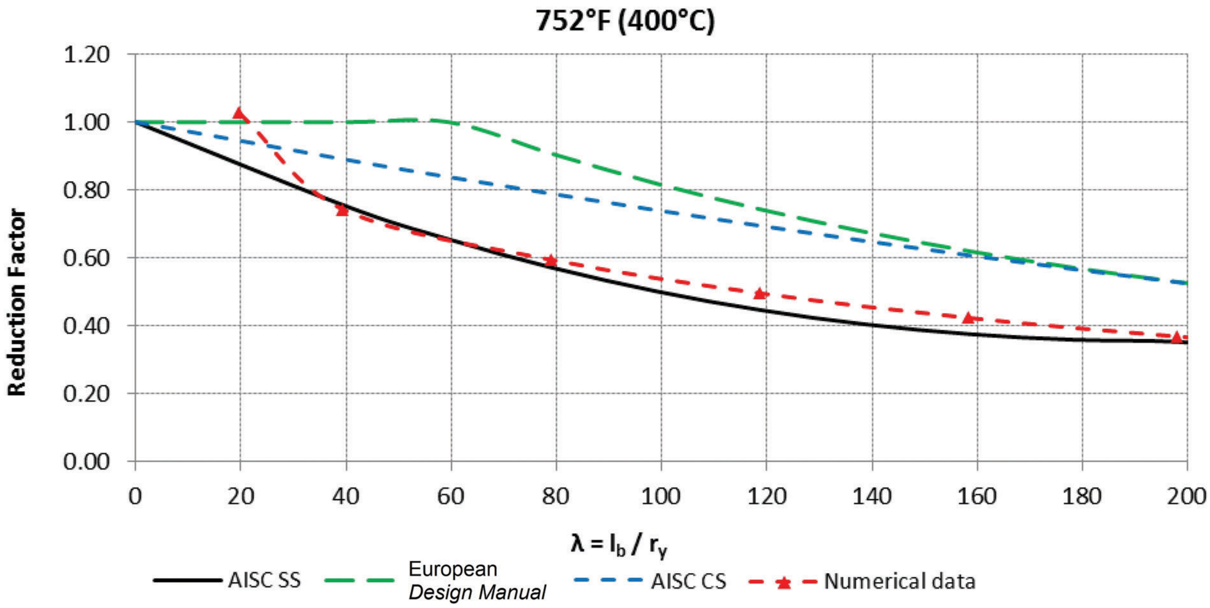



*Fig. B-31. AISC lateral-torsional buckling curves at 752 °F (400 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve, and numerical data (Vila Real et al., 2008).*

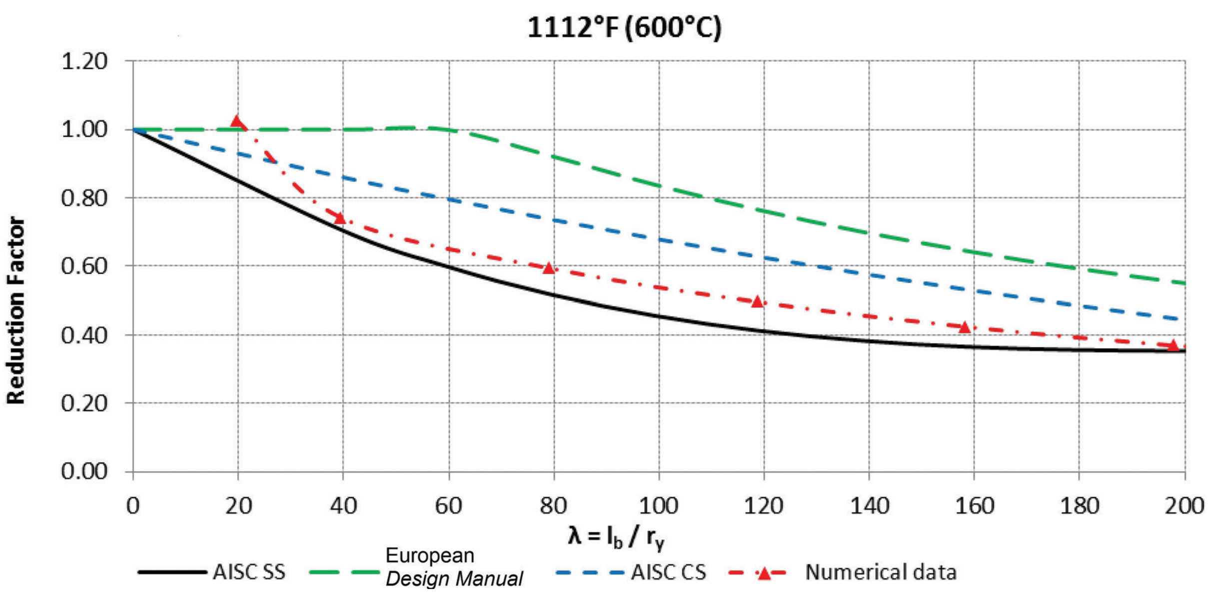



*Fig. B-32. AISC lateral-torsional buckling curves at 1,112 °F (600 °C) for carbon steel and stainless steel, compared with the European* Design Manual for Structural Stainless Steel *curve and numerical data (Vila Real et al., 2008).*

described in Section B.2, a resistance factor of 1.19 can therefore be derived. For duplex stainless steel, a value of $P_m$ = 1.135 and $V_p$ = 0.070 were determined. A resistance factor of 1.00 can therefore be derived. In order to maintain consistency with the AISC *Specification*, a resistance factor of 0.90 is recommended.

Figure B-35 plots the ratio of measured-to-predicted strengths versus cross-section slenderness.

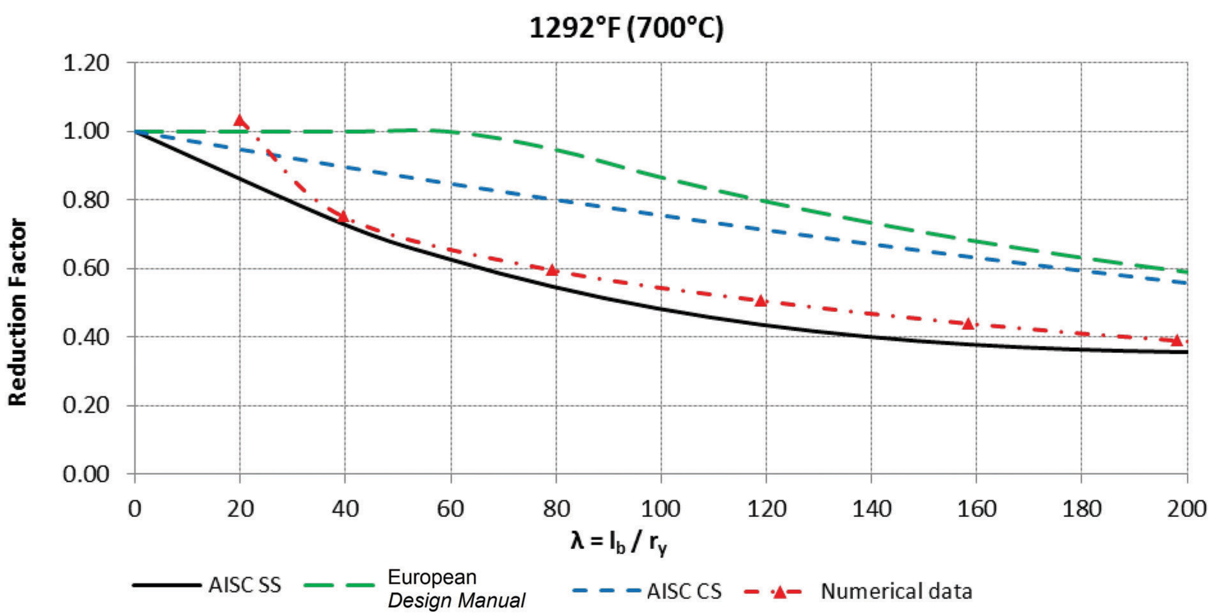



*Fig. B-33. AISC lateral-torsional buckling curves at 1,292 °F (700 °C) for carbon steel and stainless steel, compared with European* Design Manual for Structural Stainless Steel *curve and numerical data (Vila Real et al., 2008).*

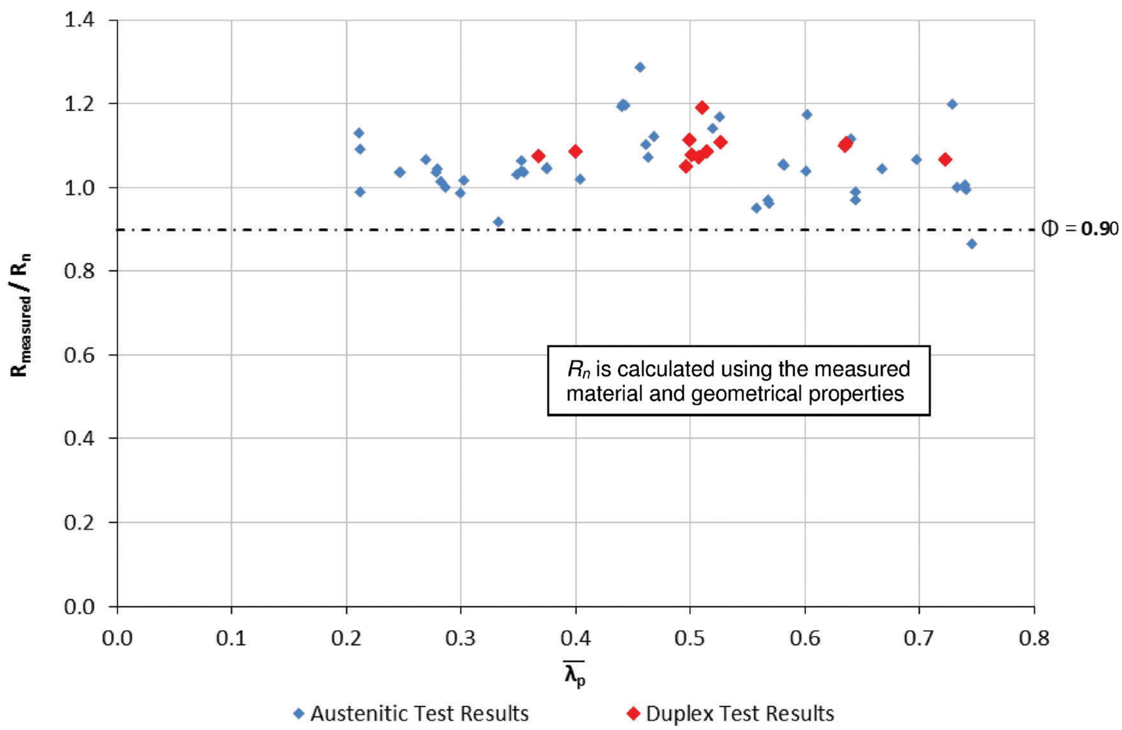



*Fig. B-34. Measured/predicted strengths versus cross-section slenderness for members subject to compression, using the continuous strength method.*

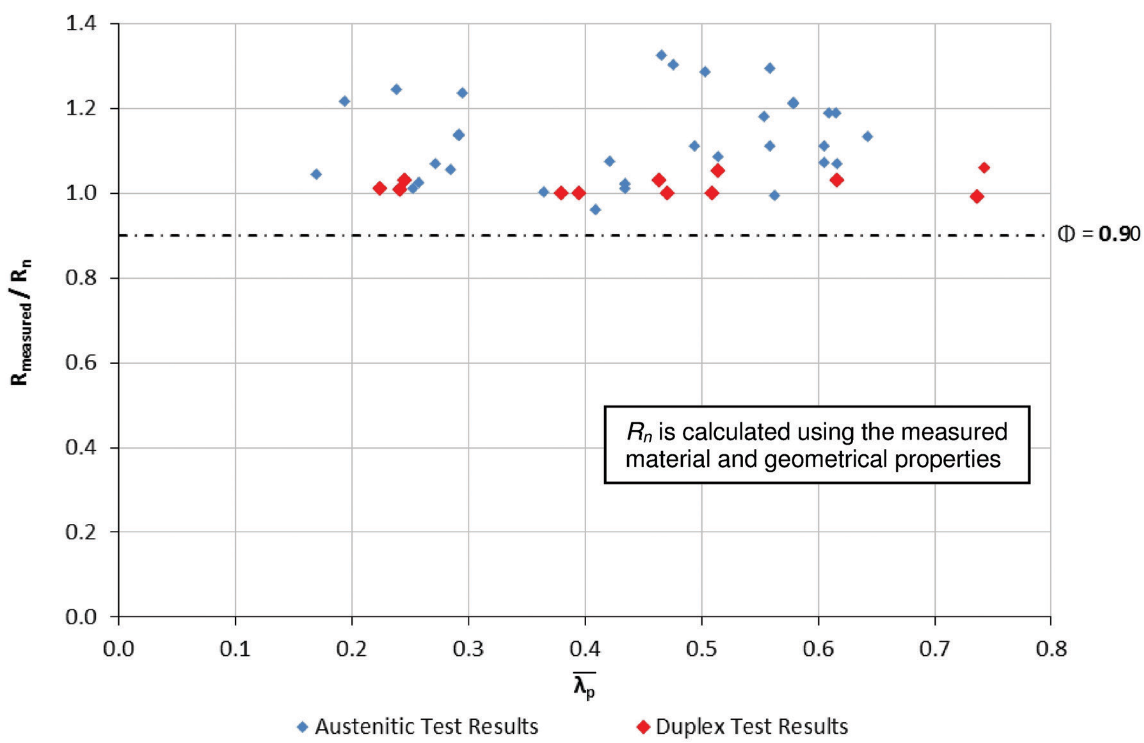



*Fig. B-35. Measured/predicted bending strengths versus cross-section slenderness for members subject to bending, using the continuous strength method.*

# DESIGN EXAMPLES

This section includes six design examples that illustrate the application of the design provisions presented here. The examples are:

**Design Example 1—**A round HSS subject to axial compression

**Design Example 2—**A square HSS with a slender cross section subject to axial compression

**Design Example 3—**W-shape subject to compression and bi-axial bending

**Design Example 4—**C-shape member subject to bending about the major axis

**Design Example 5—**Flexible end-plate connection

**Design Example 6—**A round HSS subject to axial compression in a fire

### Example 1—Round HSS Subject to Axial Compression

**Given:**

Determine the available compressive strength of a round HSS 6.625×0.280, Type S30400 stainless steel column as an interior column in a multi-story building. The column is pinned at both ends. The story height is 11 ft.

**Solution:**

From Table 2-2 and Table 2-9, the material properties are as follows:

Type S30400 stainless steel
$F_y = 30$ ksi
$E = 28{,}000$ ksi

For stainless steel HSS sections, geometric properties can be calculated directly; a reduced thickness is not applicable as it is to carbon steel sections:

$D = 6.625$ in.
$t = 0.280$ in.
$A_g = 5.58$ in.$^2$
$r = 2.25$ in.
$\lambda = \dfrac{D}{t} = 23.7$

*Check slenderness*

Calculate the limiting width-to-thickness ratio, $\lambda_r$, from Table 3-1 Case 7 for round HSS.

$$\lambda_r = 0.10\frac{E}{F_y}$$
$$= 0.10\left(\frac{28{,}000\text{ ksi}}{30\text{ ksi}}\right)$$
$$= 93.3$$

$\lambda < \lambda_r$; therefore the round HSS is nonslender

*Determine the available compressive strength*

Using Chapter 5, determine the available compressive strength. From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K = 1.0$; therefore the slenderness ratio is:

$$\frac{KL}{r} = \frac{(1.0)(11.0 \text{ ft})(12 \text{ in./ft})}{2.25 \text{ in.}}$$
$$= 58.7$$

Determine the applicable equation for critical stress, $F_{cr}$, from Section 5.3, as follows:

$$3.77\sqrt{\frac{E}{F_y}} = 3.77\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}}$$

= 115 > 58.7, therefore modified AISC *Specification* Equation E3-2 from Section 5.3 applies

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E3-4)}$$
$$= \frac{\pi^2 E}{(58.7)^2}$$
$$= 80.2 \text{ ksi}$$

$$F_{cr} = \left(0.50^{\frac{F_y}{F_e}}\right)F_y \qquad \text{(modified }\textit{Spec}\text{. Eq. E3-2)}$$
$$= \left(0.50^{\frac{30 \text{ ksi}}{80.2 \text{ ksi}}}\right)(30 \text{ ksi})$$
$$= 23.1 \text{ ksi}$$

The nominal compressive strength is:

$$P_n = F_{cr}A_g \qquad \text{(}\textit{Spec}\text{. Eq. E3-1)}$$
$$= 23.1 \text{ ksi } (5.58 \text{ in.}^2)$$
$$= 129 \text{ kips}$$

From Section 5.1, the available compressive strength is:

| LRFD | ASD |
|---|---|
| $\phi_c = 0.85$<br>$\phi_c P_n = 0.85(129 \text{ kips})$<br>$= 110 \text{ kips}$ | $\Omega_c = 1.76$<br>$\frac{P_n}{\Omega_c} = \frac{129 \text{ kips}}{1.76}$<br>$= 73.3 \text{ kips}$ |

**Example 2—Square HSS (with Slender Cross Section) Subject to Axial Compression**

**Given:**

Determine the available compressive strength of a square HSS 5.9×5.9×0.157, Type S32101 stainless steel column as an interior column in a multi-story building. The column is pinned at both ends. The story height is 11 ft.

**Solution:**

From Table 2-2 and Table 2-9, the material properties are as follows:

Type S32101 stainless steel
$F_y = 77$ ksi
$E = 29{,}000$ ksi

For stainless steel HSS sections, geometric properties can be calculated directly. Note that for determining the width-to-thickness ratio, $b$ is taken as the outside dimension minus three times the design wall thickness according to Section 3.3.1. Stainless steel sections use the actual wall thickness rather than a reduced thickness for design, as would be applicable for carbon steel sections. Therefore, the width, $b$, is determined as follows:

$$b = 5.90 \text{ in.} - 3(0.157 \text{ in.}) = 5.43 \text{ in.}$$

$t = 0.157$ in.
$A_g = 3.54$ in.$^2$
$I = 19.3$ in.$^4$
$r = 2.33$ in.

$$\lambda = \frac{b}{t} = 34.6$$

*Check slenderness*

Calculate the limiting width-to-thickness ratio, $\lambda_r$, from Table 3-1 Case 5 for square HSS walls.

$$\lambda_r = 1.24\sqrt{\frac{E}{F_y}} = 1.24\sqrt{\frac{29{,}000 \text{ ksi}}{77 \text{ ksi}}} = 24.1$$

$\lambda > \lambda_r$; therefore the square HSS is slender

*Determine the available compressive strength*

Section 5.6 is used to determine the nominal compressive strength, $P_n$, for an HSS member with slender elements. For HSS, $P_n$ is determined based upon the limit state of flexural buckling. Torsional buckling will not govern for HSS unless the torsional unbraced length greatly exceeds the controlling flexural unbraced length.

The effective area, $A_e$, must be determined in order to determine the reduction factor, $Q_a$:

$$Q_a = \frac{A_e}{A_g} \qquad \text{(\textit{Spec}. Eq. E7-16)}$$

where
$A_e$ = summation of the effective areas of the cross section based on the reduced effective width, $b_e$

For flanges of square and rectangular slender-element sections of uniform thickness, with $\frac{b}{t} \geq 1.24\sqrt{\frac{E}{f}}$:

$$b_e = 1.468t\sqrt{\frac{E}{f}}\left[1 - \frac{0.194}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(modified \textit{Spec}. Eq. E7-17)}$$

where
$f = P_n/A_e$, but can conservatively be taken as $F_y$ according to Section 5.6.2

With $f = F_y$, the effective width is:

$$b_e = 1.468t\sqrt{\frac{E}{F_y}}\left[1 - \frac{0.194}{(b/t)}\sqrt{\frac{E}{F_y}}\right] \leq b$$

$$= 1.468(0.157 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{77 \text{ ksi}}}\left[1 - \frac{0.194}{(34.6)}\sqrt{\frac{29{,}000 \text{ ksi}}{77 \text{ ksi}}}\right]$$

$$= 3.99 \text{ in.} \leq 5.43 \text{ in.}$$

$$\text{Length that is ineffective} = b - b_e = 5.43\text{ in.} - 3.99\text{ in.} = 1.44\text{ in.}$$

$$A_e = 3.54\text{ in.}^2 - 4(1.44\text{ in.})(0.157\text{ in.}) = 2.64\text{ in.}^2$$

For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). Therefore the reduction factor is:

$$Q = Q_a = \frac{A_e}{A_g} = \frac{2.64\text{ in.}^2}{3.54\text{ in.}^2} = 0.746$$

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K = 1.0$, and the slenderness ratio is:

$$\frac{KL}{r} = \frac{(1)(11.0\text{ ft})(12\text{ in./ft})}{2.33\text{ in.}} = 56.7$$

Determine the applicable equation for critical stress, $F_{cr}$, from Section 5.6, as follows:

$$3.77\sqrt{\frac{E}{QF_y}} = 3.77\sqrt{\frac{29{,}000\text{ ksi}}{0.746(77\text{ ksi})}}$$

$= 84.7 > 56.7$, therefore modified *Specification* Equation E7-2 from Section 5.6 applies

For the limit state of flexural buckling:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E3-4)}$$

$$= \frac{\pi^2(29{,}000\text{ ksi})}{(56.7)^2} = 89.0\text{ ksi}$$

$$F_{cr} = Q\left(0.50^{\frac{QF_y}{F_e}}\right)F_y \qquad \text{(modified }\textit{Spec}\text{. Eq. E7-2)}$$

$$= 0.746\left(0.50^{\frac{0.746(77\text{ ksi})}{89.0\text{ ksi}}}\right)(77\text{ ksi}) = 36.7\text{ ksi}$$

The nominal compressive strength is:

$$P_n = F_{cr}A_g \qquad \text{(}\textit{Spec}\text{. Eq. E7-1)}$$

$$= (36.7\text{ ksi})(3.54\text{ in.}^2) = 130\text{ kips}$$

From Section 5.1, the available compressive strength is:

| LRFD | ASD |
|---|---|
| $\phi_c = 0.90$<br>$\phi_c P_n = 0.90(130 \text{ kips})$<br>$= 117 \text{ kips}$ | $\Omega c = 1.67$<br>$\dfrac{P_n}{\Omega_c} = \dfrac{130 \text{ kips}}{1.67}$<br>$= 77.8 \text{ kips}$ |

**Example 3—W-Shape Subject to Compression and Bi-Axial Bending**

**Given:**

Determine if a 9-ft-long W6×16, laser fused, Type S31600 stainless steel section in a symmetric braced frame has sufficient strength to support the following required strengths:

| LRFD | ASD |
|---|---|
| $P_u = 7.00 \text{ kips}$<br>$M_{ux} = 3.00 \text{ kip-ft}$<br>$M_{uy} = 3.00 \text{ kip-ft}$ | $P_a = 4.70 \text{ kips}$<br>$M_{ax} = 2.00 \text{ kip-ft}$<br>$M_{ay} = 2.00 \text{ kip-ft}$ |

Assume the column has adequate restraint to prevent lateral-torsional buckling.

**Solution:**

From Table 2-2 and Table 2-9, the material properties are as follows:

Type S31600 stainless steel
$F_y = 30$ ksi
$E = 28{,}000$ ksi

From AISC *Manual* Table 1-1 (AISC, 2010d), the geometric properties are as follows (assume these values are conservative for a laser-fused section):

W6×16
$A = 4.74 \text{ in.}^2$
$I_x = 32.1 \text{ in.}^4$
$I_y = 4.43 \text{ in.}^4$
$Z_x = 11.7 \text{ in.}^3$
$Z_y = 3.39 \text{ in.}^3$
$S_y = 2.20 \text{ in.}^3$
$r_x = 2.60$ in.
$r_y = 0.967$ in.
$b_f = 4.03$ in.
$t_f = 0.405$ in.
$t_w = 0.260$ in.
$d = 6.28$ in.
$h_o = 5.88$ in.
$\dfrac{h}{t_w} = 19.1$
$\dfrac{b_f}{2t_f} = 4.98$

*Check element slenderness*

From Table 3-2, for a rolled I-shaped section subject to major or minor axis bending, an unstiffened compression element, such as the flanges, may be considered compact if:

$$\frac{b}{t} \le 0.33\sqrt{\frac{E}{F_y}}$$
$$\le 0.33\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}}$$
$$\le 10.1$$

$$\frac{b}{t} = \frac{b_f}{2t_f} = 4.98 \le 10.1, \text{ therefore the flange is compact.}$$

The web of a I-shaped section is considered compact if:

$$\frac{h}{t_w} \le 2.54\sqrt{\frac{E}{F_y}}$$
$$\le 2.54\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}}$$
$$\le 77.6$$

$$\frac{h}{t_w} = 19.1 \le 77.6, \text{ therefore, the web is compact.}$$

For a stiffened element, the more stringent limit of a stiffened element subject to a compression load (Table 3-1) governs over the limit of a stiffened element subject to flexure (Table 3-2):

$$\frac{h}{t_w} \le 1.24\sqrt{\frac{E}{F_y}}$$
$$\le 1.24\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}}$$
$$\le 37.9$$

$$\frac{h}{t_w} \le 37.9 \text{ therefore the section is nonslender under axial compression}$$

*Determine the available compressive strength*

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K = 1.0$; therefore, the slenderness ratio is:

$$\frac{KL}{r} = \frac{(1.0)(9.00 \text{ ft})(12 \text{ in./ft})}{0.967 \text{ in.}}$$
$$= 112$$

Determine the applicable equation for critical stress, $F_{cr}$, from Section 5.3 as follows:

$$3.77\sqrt{\frac{E}{F_y}} = 3.77\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}}$$

= 115 > 112, therefore modified AISC *Specification* Equation E3-2 from Section 5.3 applies

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(\textit{Spec}. Eq. E3-4)}$$

$$= \frac{\pi^2 (28{,}000 \text{ ksi})}{(112)^2}$$
$$= 22.0 \text{ ksi}$$

$$F_{cr} = \left( 0.50^{\frac{F_y}{F_e}} \right) F_y \qquad \text{(modified } \textit{Spec.} \text{ Eq. E3-2)}$$

$$= \left( 0.50^{\frac{30 \text{ ksi}}{22.0 \text{ ksi}}} \right) (30 \text{ ksi})$$
$$= 11.7 \text{ ksi}$$

The nominal compressive strength is:

$$P_n = F_{cr} A_g \qquad (\textit{Spec.} \text{ Eq. E3-1})$$
$$= 11.7 \text{ ksi } (4.74 \text{ in.}^2)$$
$$= 55.5 \text{ kips}$$

From Section 5.1, the available compressive strength is:

| LRFD | ASD |
|---|---|
| $\phi_c P_n = 0.90(55.5 \text{ kips})$<br>$= 50.0 \text{ kips}$ | $\dfrac{P_n}{\Omega_c} = \dfrac{55.5 \text{ kips}}{1.67}$<br>$= 33.2 \text{ kips}$ |

*Determine the available flexural strength*

For a compact section bent about the major axis, the limit states of yielding and lateral-torsional buckling apply. This member was said to have adequate restraint to prevent lateral-torsional buckling; therefore the limit state of yielding will control. As discussed in Section 6.2, the AISC *Specification* Section F2 applies to stainless steel, where the nominal flexural strength for yielding is defined as follows:

$$M_{nx} = M_p \qquad (\textit{Spec.} \text{ Eq. F2-1})$$
$$= F_y Z_x$$
$$= (30 \text{ ksi})(11.7 \text{ in.}^3) / (12 \text{ in./ft})$$
$$= 29.3 \text{ kip-ft}$$

From Section 6.1, the available flexural strength about the major axis is:

| LRFD | ASD |
|---|---|
| $\phi_b M_{nx} = 0.90(29.3 \text{ kip-ft})$<br>$= 26.4 \text{ kip-ft}$ | $\dfrac{M_{nx}}{\Omega_b} = \dfrac{29.3 \text{ kip-ft}}{1.67}$<br>$= 17.5 \text{ kip-ft}$ |

As discussed in Section 6.2, the AISC *Specification* Section F6 applies to stainless steel I-shaped members bent about their minor axis and the limit states of yielding and flange local buckling apply; however, for sections with compact flanges the limit state of flange local buckling does not apply. The limit state of yielding will control:

$$
\begin{aligned}
M_{ny} &= F_y Z_y \le 1.6 F_y S_y && \text{(Spec. Eq. F6-1)}\\
&= (30\text{ ksi})(3.39\text{ in.}^3)/(12\text{ in./ft})\\
&= 8.48\text{ kip-ft}\\
1.6F_y S_y &= 1.6(30\text{ ksi})(2.20\text{ in.}^3)/(12\text{ in./ft})\\
&= 8.80\text{ kip-ft}
\end{aligned}
$$

Therefore, $M_{ny} = 8.48$ kip-ft.

From Section 6.1, the available flexural strength about the minor axis is:

| LRFD | ASD |
|---|---|
| $\phi_b M_{ny} = 0.90(8.48\text{ kip-ft})$<br>$= 7.63\text{ kip-ft}$ | $\dfrac{M_{ny}}{\Omega_b} = \dfrac{8.48\text{ kip-ft}}{1.67}$<br>$= 5.08\text{ kip-ft}$ |

As discussed in Section 8.1.1, for doubly symmetric members subject to flexure and compression, the guidance in AISC *Specification* Section H1.1 applies. Determine whether AISC *Specification* Equation H1-1a or Equation H1-1b is applicable in this example:

| LRFD | ASD |
|---|---|
| $\dfrac{P_r}{P_c} = \dfrac{P_u}{\phi_c P_n}$<br>$= \dfrac{7.00\text{ kips}}{50.0\text{ kips}}$<br>$= 0.140 < 0.2$ | $\dfrac{P_r}{P_c} = \dfrac{P_r}{P_n/\Omega_c}$<br>$= \dfrac{4.70\text{ kips}}{33.2\text{ kips}}$<br>$= 0.142 < 0.2$ |

From AISC *Specification* Section H1, check the applicable interaction Equation H1-1b, as follows:

| LRFD | ASD |
|---|---|
| $\dfrac{P_u}{2(\phi_c P_n)} + \left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$<br>$\dfrac{7.00\text{ kips}}{2(50.0\text{ kips})} + \left(\dfrac{3.00\text{ kip-ft}}{26.4\text{ kip-ft}} + \dfrac{3.00\text{ kip-ft}}{7.63\text{ kip-ft}}\right) \le 1.0$<br>$0.577 < 1.0$ **o.k.** | $\dfrac{P_a}{2(P_n/\Omega_b)} + \left(\dfrac{M_{ax}}{M_{nx}/\Omega_b} + \dfrac{M_{ay}}{M_{ny}/\Omega_b}\right) \le 1.0$<br>$\dfrac{4.70\text{ kips}}{2(33.2\text{ kips})} + \left(\dfrac{2.00\text{ kip-ft}}{17.5\text{ kip-ft}} + \dfrac{2.00\text{ kip-ft}}{5.08\text{ kip-ft}}\right) \le 1.0$<br>$0.579 < 1.0$ **o.k.** |

Therefore, the W6×16 is adequate.

**Example 4—Channel Subject to Strong-Axis Bending**

**Given:**

Determine the available flexural and shear strength of a C12×30 Type S30400 stainless steel beam with a simple span of 30 ft. Also, determine the deflection at midspan due to an unfactored dead load of 5 kips applied at the midpoint. The beam is laterally restrained at its midpoint. The self-weight of the beam will be ignored for the purposes of this calculation.

**Solution:**

From Table 2-2 and Table 2-9, the material properties are as follows:

Type S31600 stainless steel

$F_y = 30$ ksi
$E = 28{,}000$ ksi

From AISC *Manual* Table 1-5, the geometric properties are as follows:

C12×30
$A = 8.81$ in.$^2$
$I_x = 162$ in.$^4$
$I_y = 5.12$ in.$^4$
$Z_x = 33.8$ in.$^3$
$S_x = 27.0$ in.$^3$
$r_x = 4.29$ in.
$r_y = 0.762$ in.
$b_f = 3.17$ in.
$t_f = 0.501$ in.
$t_w = 0.510$ in.
$d = 12.0$ in.
$T = 9\frac{3}{4}$ in.
$h_o = 11.5$ in.
$J = 0.861$ in.$^4$
$C_w = 151$ in.$^6$

*Check slenderness*

The flange width-to-thickness ratio is:

$$\begin{aligned}\frac{b}{t} &= \frac{b_f}{t_f} \\ &= \frac{3.17 \text{ in.}}{0.501 \text{ in.}} \\ &= 6.33\end{aligned}$$

From Table 3-2, flanges of channels are considered to be compact if:

$$\begin{aligned}\frac{b}{t} &\le 0.33\sqrt{\frac{E}{F_y}} \\ &\le 0.33\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}} \\ &\le 10.1\end{aligned}$$

$\frac{b}{t} \le 10.1$; therefore the flange is compact.

From Table 3-2, webs of channels are considered to be compact if:

$$\begin{aligned}\frac{h}{t_w} &\le 2.54\sqrt{\frac{E}{F_y}} \\ &\le 2.54\sqrt{\frac{28{,}000 \text{ ksi}}{30 \text{ ksi}}} \\ &\le 77.6\end{aligned}$$

The value of $h$ in the preceeding equation will be taken as the value $T$ given in Table 1-5 of the AISC *Manual*:

$$\frac{h}{t_w} = \frac{T}{t_w}$$

$$= \frac{9\tfrac{3}{4}\text{ in.}}{0.510\text{ in.}}$$

$$= 19.1\text{ in.}$$

$\frac{h}{t_w} \leq 77.6$; therefore the web is compact.

*Determine the available flexural strength*

For a compact channel section bent about the major axis, the limit states of yielding and lateral-torsional buckling apply. As discussed in Section 6.2, AISC *Specification* Section F2 applies to stainless steel, where the nominal flexural strength for yielding is defined as follows:

$$M_n = M_p \qquad \text{(}Spec\text{. Eq. F2-1)}$$

$$= F_y Z_x$$

$$= (30\text{ ksi})(33.8\text{ in.}^3)/(12\text{ in./ft})$$

$$= 84.5\text{ kip-ft}$$

The limit state of lateral-torsional buckling is determined from Section 6.2, as follows:

$$L_p = 0.8 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(modified }Spec\text{. Eq. F2-5)}$$

$$= 0.8(0.762\text{ in.})\sqrt{\frac{28{,}000\text{ ksi}}{30\text{ ksi}}}\Bigg/(12\text{ in./ft})$$

$$= 1.55\text{ ft}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \qquad \text{(}Spec\text{. Eq. F2-6)}$$

For channel sections:

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \qquad \text{(}Spec\text{. Eq. F2-8b)}$$

$$= \frac{11.5\text{ in.}}{2}\sqrt{\frac{5.12\text{ in.}^4}{151\text{ in.}^6}}$$

$$= 1.06$$

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \qquad \text{(}Spec\text{. Eq. F2-7)}$$

$$= \frac{\sqrt{(5.12\text{ in.}^4)(151\text{ in.}^6)}}{27.0\text{ in.}^3}$$

$$= 1.03\text{ in.}^2$$

$$r_{ts} = 1.01\text{ in.}$$

Therefore:

$$L_r = 1.95(1.01 \text{ in.}) \frac{28{,}000 \text{ ksi}}{0.7(30 \text{ ksi})} \sqrt{\frac{(0.861 \text{ in.}^4)(1.06)}{(27.0 \text{ in.}^3)(11.5 \text{ in.})} + \sqrt{\left[\frac{(0.861 \text{ in.}^4)(1.06)}{(27.0 \text{ in.}^3)(11.5 \text{ in.})}\right]^2 + 6.76\left(\frac{0.7(30 \text{ ksi})}{28{,}000 \text{ ksi}}\right)^2}} \Bigg/ (12 \text{ in./ft})$$

$$= 17.6 \text{ ft}$$

The unbraced length, $L_b$, is 15.0 ft. Because $L_p < L_b < L_r$:

$$M_n = C_b\left[M_p - \left(M_p - 0.45F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(modified } \textit{Spec.} \text{ Eq. F2-2)}$$

From the User Note in AISC *Specification* Section F1, $C_b = 1.67$ when one end moment equals zero in the unbraced segment.

$$M_n = 1.67\left\{84.5 \text{ kip-ft} - \left[84.5 \text{ kip-ft} - 0.45(30 \text{ ksi})(27.0 \text{ in.}^3)\big/(12 \text{ in./ft})\right]\left(\frac{15.0 \text{ ft} - 1.55 \text{ ft}}{17.6 \text{ ft} - 1.55 \text{ ft}}\right)\right\}$$

$$= 65.4 \text{ kip-ft} \le 84.5 \text{ kip-ft}$$

From Section 6.1, the available flexural strength is:

| LRFD | ASD |
|---|---|
| $\phi_b = 0.90$<br>$\phi_b M_n = 0.90(65.4 \text{ kip-ft})$<br>$= 58.9 \text{ kip-ft}$ | $\Omega_b = 1.67$<br>$\dfrac{M_n}{\Omega_b} = \dfrac{65.4 \text{ kip-ft}}{1.67}$<br>$= 39.2 \text{ kip-ft}$ |

*Determine the available shear strength*

From Section 7, the provisions in AISC *Specification* Chapter G apply. Therefore, the available shear strength is determined as follows:

$$V_n = 0.6F_yA_wC_v \qquad (\textit{Spec.} \text{ Eq. G2-1})$$

where

$$\begin{aligned} A_w &= dt_w \\ &= (12.0 \text{ in})(0.510 \text{ in.}) \\ &= 6.12 \text{ in.}^2 \end{aligned}$$

Determine the value of $C_v$ from AISC *Specification* Section G2.1(b):

When $\dfrac{h}{t_w} \le 1.10\sqrt{\dfrac{k_vE}{F_y}}$, $C_v = 1.0$

$$1.10\sqrt{\frac{5(28{,}000 \text{ ksi})}{30 \text{ ksi}}} = 75.1$$

$\dfrac{h}{t_w} = 19.1 \le 75.1,$ therefore $C_v = 1.0$.

Calculate $V_n$:

$$V_n = 0.6(30\text{ ksi})(6.12\text{ in.}^2)(1.0)$$
$$= 110\text{ kips}$$

From Section 7, the available shear strength is:

| LRFD | ASD |
|---|---|
| $\phi v = 0.90$<br>$\phi_v V_n = 0.90(110\text{ kips})$<br>$= 99.0\text{ kips}$ | $\Omega_v = 1.67$<br>$\dfrac{V_n}{\Omega_v} = \dfrac{110\text{ kips}}{1.67}$<br>$= 65.9\text{ kips}$ |

*Calculate the dead load deflection*

From Section 6.7, the secant modulus of elasticity, $E_S$, should be used instead of the modulus of elasticity, where $E_S$ is given by:

$$E_s = \frac{E}{1 + 0.002\left(\dfrac{E}{F_{ser}}\right)\left(\dfrac{F_{ser}}{F_y}\right)^n} \tag{6-1}$$

where

$n = 5.6$ for Type S30400 from Table 6-1

$$F_{ser} = \frac{M}{S_x}$$
$$= \frac{(5.00\text{ kips})(30.0\text{ ft})(12\text{ in./ft})/4}{27.0\text{ in.}^3}$$
$$= 16.7\text{ ksi}$$

Therefore:

$$E_s = \frac{28{,}000\text{ ksi}}{1 + 0.002\left(\dfrac{28{,}000\text{ ksi}}{16.7\text{ ksi}}\right)\left(\dfrac{16.7\text{ ksi}}{30\text{ ksi}}\right)^{5.6}}$$
$$= 24{,}900\text{ ksi}$$

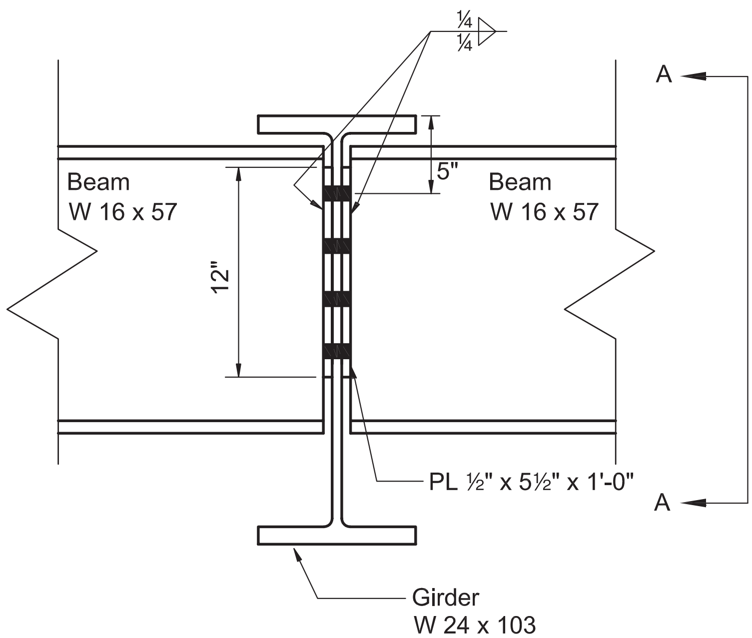
Ignoring the self-weight of the beam, the vertical deflection at the midpoint of the beam is given by the following equation from AISC *Manual* Table 3-23, Case 7:

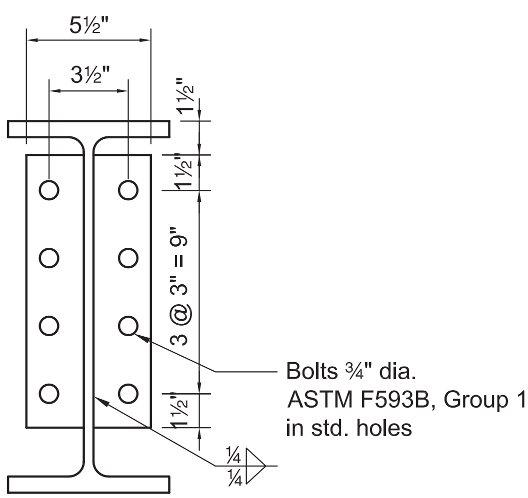
$$\Delta_x = \frac{PL^3}{48E_s I_x}$$
$$= \frac{(5\text{ kips})\left[(30\text{ ft})(12\text{ in./ft})\right]^3}{48(24{,}900\text{ ksi})(162\text{ in.}^4)}$$
$$= 1.20\text{ in.}$$
$$= \frac{L}{300}$$

**Example 5—Shear End-Plate Connection of Beam-to-Girder Web**

**Given:**

Determine the available strength of the shear end-plate connection given in the details shown. Beam sizes and dimensions, and end-plate dimensions are as shown. Both beams and the end plates are Type S30400 stainless steel material. The bolts are ¾-in.-diameter, ASTM F593B (Condition A), Group 1 stainless steel, in standard holes.




*Section A-A*

**Solution:**

From Table 2-2 and Table 2-9, the beam and end-plate material properties are as follows:

Type S30400 stainless steel
$F_y = 30$ ksi
$F_u = 75$ ksi
$E = 28{,}000$ ksi

From Table 2-4, the bolt material properties are as follows:
ASTM F593B (Condition A), Group 1 stainless steel bolts
$F_y = 30$ ksi
$F_u = 75$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×57
$d = 16.4$ in.
$b_f = 7.12$ in.
$t_f = 0.715$ in.
$t_w = 0.430$ in.
$$\frac{h}{t_w} = 33.0$$

Girder
W24×103
$d = 24.5$ in.
$b_f = 9.00$ in.
$t_f = 0.980$ in.
$t_w = 0.550$ in.

*Determine the available end-plate weld strength*

As stipulated in Table 9-1, for welding Type S30400 stainless steel, filler metal Group B is recommended. The minimum tensile strength of the filler metal is $F_{EXX} = 75$ ksi.

As discussed in Section 9.2, the weld strength formulations given in AISC *Specification* Section J2 apply, except for the resistance and safety factors. For a linear weld group with uniform leg size:

$$R_n = F_{nw} A_{we} \qquad \text{(}\textit{Spec}\text{. Eq. J2-4)}$$

where

$$\begin{aligned} F_{nw} &= 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5} \theta) \qquad \text{(}\textit{Spec}\text{. Eq. J2-5)} \\ &= 0.60(75 \text{ ksi})(1.0) \\ &= 45.0 \text{ ksi} \end{aligned}$$

$$\begin{aligned} A_{we} &= (2 \text{ lines of weld})(\text{weld length})(\text{effective throat width}) \\ &= 2(12.0 \text{ in.})\left(\frac{1/4 \text{ in.}}{\sqrt{2}}\right) \\ &= 4.24 \text{ in.}^2 \end{aligned}$$

Therefore:

$$\begin{aligned} R_n &= (45.0 \text{ ksi})(4.24 \text{ in.}^2) \\ &= 191 \text{ kips} \end{aligned}$$

From Section 9.2, the available weld strength is:

| LRFD | ASD |
|---|---|
| $\phi = 0.55$<br>$\phi R_n = 0.55(191\text{ kips})$<br>$= 105\text{ kips}$ | $\Omega = 2.70$<br>$\dfrac{R_n}{\Omega} = \dfrac{191\text{ kips}}{2.70}$<br>$= 70.7\text{ kips}$ |

*Determine the available shear strength of the beam*

From Section 9.4, the provisions in AISC *Specification* Section J4.2 apply for determining the available shear strength of the beam, with the stainless steel resistance and safety factors.

Shear yielding:

$$R_n = 0.60F_yA_{gv} \qquad \text{(\textit{Spec}. Eq. J4-3)}$$

where

$$\begin{aligned} A_{gv} &= dt_w \\ &= (16.4\text{ in.})(0.430\text{ in.}) \\ &= 7.05\text{ in.}^2 \end{aligned}$$

The nominal shear strength is:

$$\begin{aligned} R_n &= 0.60(30\text{ ksi})(7.05\text{ in.}^2) \\ &= 127\text{ kips} \end{aligned}$$

The available shear yielding strength is:

| LRFD | ASD |
|---|---|
| $\phi_v = 0.90$<br>$\phi_v V_n = 0.90(127\text{ kips})$<br>$= 114\text{ kips}$ | $\Omega_v = 1.67$<br>$\dfrac{V_n}{\Omega_v} = \dfrac{127\text{ kips}}{1.67}$<br>$= 76.0\text{ kips}$ |

Shear rupture:

The AISC *Steel Construction Manual* specifies that shear rupture of the beam web must be checked along the length of the weld connecting the shear end-plate to the beam web.

$$R_n = 0.60F_uA_{nv} \qquad \text{(\textit{Spec}. Eq. J4-4)}$$

where

$$\begin{aligned} A_{nv} &= (\text{length of end-plate weld})t_w \\ &= (12\text{ in.})(0.430\text{ in.}) \\ &= 5.16\text{ in.}^2 \end{aligned}$$

The nominal shear strength is:

$$\begin{aligned} R_n &= 0.60(75\text{ ksi})(5.16\text{ in.}^2) \\ &= 232\text{ kips} \end{aligned}$$

The available shear rupture strength is:

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$\phi V_n = 0.75(232 \text{ kips})$<br>$= 174 \text{ kips}$ | $\Omega = 2.00$<br>$\dfrac{V_n}{\Omega} = \dfrac{232 \text{ kips}}{2.00}$<br>$= 116 \text{ kips}$ |

*Determine the available bearing strength of the girder web at bolt holes*

Assume deformation at the bolt hole at the service load level is a design consideration. The available bearing strength of the girder web is determined in accordance with Section 9.3.6. The nominal bearing strength is determined from:

$$R_n = \alpha_d t d F_u \tag{9-1}$$

For end bolts, with bolt hole diameter, $d_h = {}^{13}\!/_{16}$ in., for ¾-in.-diameter bolts in standard holes, from AISC *Specification* Table J3.3:

$$\alpha_d = 1.25\left(\frac{e_1}{2d_h}\right) \le 1.25 \tag{9-2}$$

$$= 1.25\left[\frac{10.50 \text{ in.}}{2({}^{13}\!/_{16} \text{ in.})}\right]$$

$$= 8.08 > 1.25 \text{ therefore } \alpha_d = 1.25$$

The nominal bearing strength on the girder web of a single bolt in the top row is:

$$R_n = 1.25 t d F_u$$

$$= 1.25(0.550 \text{ in.})({}^{3}\!/_{4} \text{ in.})(75 \text{ ksi})$$

$$= 38.7 \text{ kips}$$

For both end bolts, the nominal bearing strength on the girder web is:

$$nR_n = 2(38.7 \text{ kips})$$
$$= 77.4 \text{ kips}$$

For the inner bolts:

$$\alpha_d = 1.25\left(\frac{p_1}{4d_h}\right) \le 1.25 \tag{9-3}$$

$$= 1.25\left[\frac{3.00 \text{ in.}}{4({}^{13}\!/_{16} \text{ in.})}\right]$$

$$= 1.15 < 1.25 \text{ therefore } \alpha_d = 1.15$$

The nominal bearing strength for a single inner bolt is:

$$R_n = 1.15(0.550 \text{ in.})({}^{3}\!/_{4} \text{ in.})(75 \text{ ksi})$$
$$= 35.6 \text{ kips}$$

For the 6 inner bolts, the nominal bearing strength on the girder web is:

$$nR_n = 6(35.6 \text{ kips})$$
$$= 214 \text{ kips}$$

The total nominal bolt bearing strength for the entire bolt group is:

$$(R_n)_{total} = 77.4 \text{ kips} + 214 \text{ kips}$$
$$= 291 \text{ kips}$$

From Section 9.3.6, the available bolt bearing strength on the girder web is:

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$\phi(R_n)_{total} = 0.75(291 \text{ kips})$<br>$= 218 \text{ kips}$ | $\Omega = 2.00$<br>$\dfrac{(R_n)_{total}}{\Omega} = \dfrac{291 \text{ kips}}{2.00}$<br>$= 146 \text{ kips}$ |

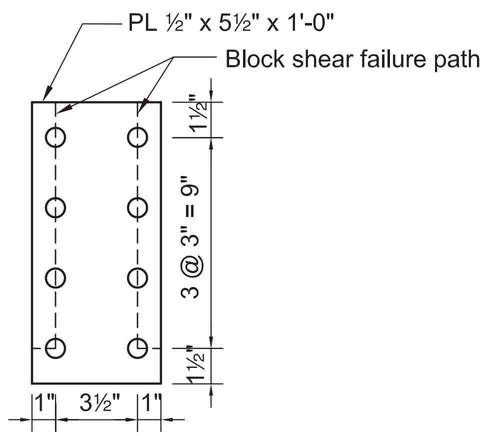
Bolt bearing on the end plate is not checked here, as it would not be a controlling limit state because there are two plates to distribute the load between, but it would be found to give a similar available strength to the girder web.

*Check block shear rupture of end plate*

As discussed in Section 9.4, the available strength due to block shear rupture is determined from AISC *Specification* Section J4.3, where the nominal strength is:

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec}. \text{ Eq. J4-5})$$

The block shear rupture failure path is assumed to occur as shown here:




As stipulated in AISC *Specification* Section B4.3b, the bolt hole is increased by 1⁄16 in. for the net tension and shear areas. When the tension stress is uniform, $U_{bs} = 1.0$.

$$A_{nv} = 2\left[10.5 \text{ in.} - (3.5)\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{1}{2} \text{ in.}\right)$$
$$= 7.44 \text{ in.}^2$$
$$A_{nt} = 2\left(1.00 \text{ in.} - \frac{\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}}{2}\right)\left(\tfrac{1}{2} \text{ in.}\right)$$
$$= 0.563 \text{ in.}^2$$
$$A_{gv} = 2(10.5 \text{ in.})\left(\tfrac{1}{2} \text{ in.}\right)$$
$$= 10.5 \text{ in.}^2$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \\
&= 0.60(75\text{ ksi})(7.44\text{ in.}^2) + (1.0)(75\text{ ksi})(0.563\text{ in.}^2) \le 0.60(30\text{ ksi})(10.5\text{ in.}^2) + (1.0)(75\text{ ksi})(0.563\text{ in.}^2) \\
&= 377\text{ kips} > 231\text{ kips} \quad \text{therefore } R_n = 231\text{ kips}
\end{aligned}
$$

From AISC *Specification* Section J4.3, the available block shear strength is:

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$\phi R_n = (0.75)(231\text{ kips})$<br>$= 173\text{ kips}$ | $\Omega = 2.00$<br>$\dfrac{R_n}{\Omega} = \dfrac{231\text{ kips}}{2.00}$<br>$= 116\text{ kips}$ |

*Available bolt shear strength*

From Section 9.3.4, the nominal shear rupture strength of the bolts is determined as follows:

$$R_n = F_nA_b \qquad (\textit{Spec}.\text{ Eq. J3-1})$$

where

$$
\begin{aligned}
F_{nv} &= 0.45F_u \quad \text{assuming that the threads are in the shear plane} \\
&= 0.45(75\text{ ksi}) \\
&= 33.8\text{ ksi}
\end{aligned}
$$

$$
\begin{aligned}
A_b &= \frac{\pi(0.75\text{ in.})^2}{4} \\
&= 0.442\text{ in.}^2
\end{aligned}
$$

Therefore:

$$
\begin{aligned}
R_n &= (33.8\text{ ksi})(0.442\text{ in.}^2) \\
&= 14.9\text{ kips/bolt}
\end{aligned}
$$

From Section 9.3.4, the available bolt shear strength for 8 bolts in double shear is:

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$2(n)(\phi R_n) = 2(8)(0.75)(14.9\text{ kips/bolt})$<br>$= 179\text{ kips}$ | $\Omega = 2.00$<br>$2(n)\left(\dfrac{R_n}{\Omega}\right) = 2(8)\left(\dfrac{14.9\text{ kips/bolt}}{2.00}\right)$<br>$= 119\text{ kips}$ |

The shear yielding and shear rupture strengths of the end plate and girder are not checked in this example because these checks are similar to those for the supported beam, and would not govern because the end plate is thicker than the beam web. Block shear rupture on the girder web should also be checked, but is not included in this example.

*Summary*

The required shear strength for a single beam connected on one side of the girder should be less than the values presented in the following table:

| Limit State | LRFD (kips) | ASD (kips) |
|---|---|---|
| Available weld strength | 105 | 70.7 |
| Available beam shear strength | 114 | 76.0 |
| Available bearing strength (girder web) | 109 | 73.0 |
| Available end-plate block shear strength | 173 | 116 |
| Available bolt shear strength | **89.5** | **59.5** |

As shown in this summary, the available bolt shear strength is the controlling limit state for this connection.

**Example 6—Round HSS Subject to Axial Compression in a Fire**

**Given:**

Determine the design strength of the round HSS column in Example 1 subjected to a standard ASTM E119 fire for 30 minutes.

**Solution:**

From Table 2-2 and Table 2-9, the material properties are as follows:

Type S30400 stainless steel
$F_y = 30$ ksi
$E = 28{,}000$ ksi

For Example 1, the geometric properties are:

$D = 6.625$ in.
$t = 0.280$ in.
$A_g = 5.58$ in.$^2$
$r = 2.25$ in.

The temperature rise, $\Delta T_s$, of an unprotected steel section in a short time period, $\Delta t$, is determined by:

$$\Delta T_s = \frac{a}{c_s\left(\dfrac{W}{D}\right)}\left(T_F - T_s\right)\Delta t \qquad \text{(Comm. } \textit{Spec}\text{. Eq. C-A-4-2)}$$

where

$W$ = weight (mass) per unit length
= 19.0 lb/ft
$D$ = heat perimeter
= 20.8 in.

From the Commentary on the AISC *Specification* Appendix 4:

$$a = a_c + a_r \qquad \text{(Comm. } \textit{Spec}\text{. Eq. C-A-4-3)}$$

$$a_c = 4.4 \text{ Btu}/(\text{ft}^2\text{-hr-°F})$$

$$a_r = \frac{5.67\text{x}10^{-8}\varepsilon_F}{T_F - T_s}\left(T_F{}^4 - T_s^4\right) \qquad \text{(Comm. } \textit{Spec}\text{. Eq. C-A-4-4)}$$

From Section 10.2.4, $\varepsilon_F = 0.4$

$$c_s = 0.107 + 0.372\times10^{-5}(T_s - 32) - 2.15\times10^{-8}(T_s - 32)^2 - 5.49\text{x}10^{-12}(T_s - 32)^3 \text{ BTU/(lb-°F)} \qquad (10\text{-}1)$$

For accuracy reasons, the AISC *Specification* Commentary suggests a maximum limit for the time step, $\Delta t$, of 5 s. For the purpose of this design example, a time step of 2 s was used.

Using Equations C-A-4-2, C-A-4-3 and C-A-4-4 with the time-temperature curve in ASTM E119 (ASTM, 2012j), the temperature of the steel at 30 min was calculated. It should be noted that the temperature curve given in ASTM E119 is only defined at 5-min intervals. For the purpose of calculating the temperature of the steel at 2-s time steps, linear interpolation was used. The steel temperature obtained from using the above equation with a time step of 2 s was 1,505 °F (818 °C), which is almost equal to the fire exposure temperature. The column is unprotected and hence, the steel material will approach the exposure temperature in potentially a relatively short time, depending on the thermal mass of the member.

Table 10-2 gives reduction factors for Type S30400 stainless steel:

| Steel Temperature | $k_E(T) = \dfrac{E(T)}{E}$ | $k_y(T) = \dfrac{F_y(T)}{F_y}$ |
|---|---|---|
| 1,400 °F (760 °C) | 0.66 | 0.30 |
| 1,600 °F (871 °C) | 0.50 | 0.18 |

Interpolating for a temperature of 1,504 °F gives $k_E = 0.58$ and $k_y = 0.24$:

$$\begin{aligned} F_y(1{,}504\ °\text{F}) &= k_y(1{,}504\ °\text{F})F_y \\ &= 0.24\ (30\ \text{ksi}) \\ &= 7.20\ \text{ksi} \end{aligned}$$

$$\begin{aligned} E(1{,}504\ °\text{F}) &= k_E(1{,}504\ °\text{F})E \\ &= 0.58(28{,}000\ \text{ksi}) \\ &= 16{,}200\ \text{ksi} \end{aligned}$$

*Determine the available compressive strength*

The critical stress, $F_{cr}$, is determined as follows. From Section 10.3.1, the elastic buckling stress is:

$$F_e(1{,}504\ °\text{F}) = \frac{\pi^2 E(T)}{\left(\dfrac{KL}{r}\right)^2} \tag{10-4}$$

$$= \frac{\pi^2\left(16{,}200\ \text{ksi}\right)}{\left[\dfrac{(1.0)(11\ \text{ft})(12\ \text{in./ft})}{2.25\ \text{in.}}\right]^2}$$

$$= 46.5\ \text{ksi}$$

$$\frac{F_y}{F_e} = \frac{7.20\ \text{ksi}}{46.5\ \text{ksi}}$$

$$= 0.155$$

When $\dfrac{F_y}{F_e} \le 1.44$

$$F_{cr}(1{,}504\ °\text{F}) = \left(0.50^{\frac{F_y(T)}{F_e(T)}}\right)F_y(T) \tag{10-2}$$

$$= \left(0.50^{\frac{7.20\ \text{ksi}}{46.5\ \text{ksi}}}\right)\left(7.20\ \text{ksi}\right)$$

$$= 6.47\ \text{ksi}$$

The nominal compressive strength is:

$$P_n = F_{cr}A_g \qquad (Spec.\text{ Eq. E3-1})$$
$$= (6.47\text{ ksi})(5.58\text{ in.}^2)$$
$$= 36.1\text{ kips}$$

From AISC *Specification* Appendix 4, Section 4.2.4.4, the design compressive strength is:

$$\phi_c = 0.85$$
$$\phi_c P_n = 0.85\,(36.1\text{ kips})$$
$$= 30.7\text{ kips}$$

Note that allowable strength design is not permitted for structural design for fire conditions by analysis, as stipulated in AISC *Specification* Appendix 4, Section 4.1.2.

# SYMBOLS

Some definitions in the list below have been simplified in the interest of brevity. Symbols without text definitions, used only in one location and defined at that location, are omitted in some cases. The section or table number in the right-hand column refers to the section where the symbol is first used.

| Symbol | Definition | Section |
|---|---|---|
| $A_b$ | Nominal unthreaded body area of bolt or threaded part, in.$^2$ (mm$^2$) | 9.3 |
| $A_e$ | Effective net area, in.$^2$ (mm$^2$) | 4.1 |
| $A_e$ | Summation of the effective areas of the cross section based on the reduced effective width, $b_e$, in.$^2$ (mm$^2$) | 5.6 |
| $A_g$ | Gross (total) cross-sectional area of member, in.$^2$ (mm$^2$) | 3.3 |
| $A_n$ | Net area of member, in.$^2$ (mm$^2$) | 3.3 |
| $C_b$ | Lateral-torsional buckling modification factor for nonuniform moment diagrams | 6.2 |
| $D$ | Outside diameter of round HSS, in. (mm) | Table 3-1 |
| $E$ | Modulus of elasticity of steel = 29,000 ksi (200 000 MPa) | Table 3-1 |
| $E(T)$ | Elastic modulus of elasticity of steel at elevated temperature, ksi (MPa) | 10.2 |
| $E_s$ | Secant modulus of elasticity, ksi (MPa) | 6.7 |
| $F_{cr}$ | Critical stress, ksi (MPa) | 5.3 |
| $F_{cr}(T)$ | Critical stress at high temperatures, ksi (MPa) | 10.3 |
| $F_{cry}$ | Critical stress about the axis of symmetry, ksi (MPa) | 5.4 |
| $F_e$ | Elastic buckling stress, ksi (MPa) | 5.3 |
| $F_e(T)$ | Critical elastic buckling stress with the elastic modulus $E(T)$ at elevated temperature, ksi (MPa) | 10.3 |
| $F_{EXX}$ | Filler metal classification strength, ksi (MPa) | 9.2 |
| $F_n$ | Nominal tensile stress $F_{nt}$, or shear stress, $F_{nv}$ | 9.3 |
| $F_{nt}$ | Nominal tensile stress, ksi (MPa) | 9.3 |
| $F_{nv}$ | Nominal shear stress, ksi (MPa) | 9.3 |
| $F_p(T)$ | Proportional limit at elevated temperatures, ksi (MPa) | B.10.3.3 |
| $F_{ser}$ | Maximum serviceability design stress, ksi (MPa) | 6.7 |
| $F_u$ | Specified minimum tensile strength, ksi (MPa) | 2.3 |
| $F_u(T)$ | Minimum tensile strength at elevated temperature, ksi (MPa) | 10.2 |
| $F_y$ | Specified minimum yield stress, ksi (Mpa) | 2.3 |
| $F_y(T)$ | Yield stress at elevated temperature, ksi (MPa) | 10.2 |
| $G$ | Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa) | Table 2-9 |
| $K$ | Effective length factor | 5.2 |
| $L$ | Laterally unbraced length of member, in. (mm) | 5.2 |
| $L_b$ | Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm) | 6.2 |

| | | |
|---|---|---|
| $L_r$ | Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm) | 6.2 |
| $M_n$ | Nominal flexural strength, kip-in. (N-mm) | 6.1 |
| $M_p$ | Plastic bending moment, kip-in. (N-mm) | 6.2 |
| $P_n$ | Nominal axial strength, kips (N) | 4.1 |
| $P_n$ | Nominal compressive strength, kips (N) | 5.1 |
| $Q$ | Net reduction factor accounting for all slender compression elements | 5.6 |
| $Q_a$ | Reduction factor for slender stiffened elements | 5.6 |
| $Q_s$ | Reduction factor for slender unstiffened elements | 5.6 |
| $R_a$ | Required strength using ASD load combinations | 3.2 |
| $R_n$ | Nominal strength, specified in Chapters 4 to 9 | 3.2 |
| $R_u$ | Required strength using LRFD load combinations | 3.2 |
| $S_e$ | Effective section modulus about major axis, in.$^3$ (mm$^3$) | 6.3 |
| $S_x$ | Elastic section modulus taken about the $x$-axis, in.$^3$ (mm$^3$) | 6.2 |
| $V_n$ | Nominal shear strength, kips (N) | 7 |
| $Z$ | Plastic section modulus about the axis of bending, in.$^3$ (mm$^3$) | 6.3 |
| $Z_x$ | Plastic section modulus about the $x$-axis, in.$^3$ (mm$^3$) | 6.2 |
| $b$ | Width of unstiffened compression element; width of stiffened compression element, in. (mm) | 3.3 |
| $b_e$ | Reduced effective width, in. (mm) | 5.6 |
| $b_f$ | Width of flange, in. (mm) | 3.3 |
| $d$ | Nominal bolt diameter, in. (mm) | 9.3 |
| $d$ | Full nominal depth of the section, in. (mm) | 3.3 |
| $d_h$ | Hole diameter, in. (mm) | 9.3 |
| $h$ | Width of stiffened compression element, in. (mm) | 3.3 |
| $r$ | Radius of gyration, in. (mm) | 5.2 |
| $t$ | Thickness of connected material, in. (mm) | 9.5 |
| $t_w$ | Thickness of web, in. (mm) | Table 3-1 |
| $\lambda$ | Slenderness parameter | 6.2 |
| $\lambda_p$ | Limiting width-to-thickness ratio for compact element | 3.3 |
| $\lambda_{pf}$ | Limiting width-to-thickness ratio for compact flange | 6.2 |
| $\lambda_{pw}$ | Limiting width-to-thickness ratio for compact web | 6.2 |
| $\lambda_r$ | Limiting width-to-thickness ratio for noncompact element | 3.3 |
| $\lambda_{rf}$ | Limiting width-to-thickness ratio for noncompact flange | 6.2 |
| $\lambda_{rw}$ | Limiting width-to-thickness ratio for noncompact web | 6.2 |
| $\phi$ | Resistance factor, specified in Chapters 4 to 9 | 3.2 |
| $\phi_b$ | Resistance factor for flexure | 6.1 |
| $\phi_c$ | Resistance factor for compression | 5.1 |

$\phi_{ph}$ Resistance factor for precipitation hardening fasteners . . . . . . . . . . 9.3

$\phi_t$ Resistance factor for tension . . . . . . . . . . 4.1

$\phi_{tph}$ Resistance factor of an unthreaded tension rod with precipitation hardening . . . . . . . . . . 4.2

$\phi_v$ Resistance factor for shear . . . . . . . . . . 7

$\Omega$ Safety factor, specified in Chapters 4 to 9. . . . . . . . . . . 3.2

$\Omega_b$ Safety factor for flexure . . . . . . . . . . 6.1

$\Omega_c$ Safety factor for compression . . . . . . . . . . 5.1

$\Omega_{ph}$ Safety factor for precipitation hardening fastners. . . . . . . . . . . 9.3

$\Omega_t$ Safety factor for tension . . . . . . . . . . 4.1

$\Omega_{tph}$ Safety factor for an unthreaded tension rod with precipitation hardening . . . . . . . . . . 4.2

$\Omega_v$ Safety factor for shear . . . . . . . . . . 7

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# SOURCES OF ADDITIONAL INFORMATION

**International Molybdenum Association**
www.imoa.info

*For queries concerning material selection, corrosion and end uses of molybdenum-containing stainless steels.*

**International Stainless Steel Forum**
www.worldstainless.org

*For queries concerning technical information, statistics and training resources on the use of stainless steel.*

**Nickel Institute**
www.nickelinstitute.org (which hosts www.stainlessarchitecture.org and www.stainlesswater.org )

*For queries concerning material selection, corrosion and end uses.*

**Online Information Centre for Stainless Steels in Construction**
www.stainlessconstruction.com

*A website giving technical guidance, design software, design data, case studies and research papers about the design, specification, fabrication and installation of stainless steel in construction.*

**Outokumpu**
www.outokumpu.com/Products

*A website that provides guidance on selection for corrosive environments, stainless steel properties, welding and other fabrication, and other topics related to stainless steel selection, fabrication and use in end-use applications.*

**Specialty Steel Industry of North America (SSINA)**
www.ssina.com

*For queries concerning material selection, corrosion, product forms and availability in the U.S.*

**Stainless SteelCAL**
www.steel-stainless.org/steelcal

*Computer aided learning in stainless steel design for engineers and architects (available in English, Portuguese and Spanish).*

**Stainless Steel Design Software**
www.steel-stainless.org/software

*Online software for designing structural stainless steel sections, containing a database of laser fused and hot rolled sections.*

**Stainless Structurals**
www.sss.us.com

*Section property and other design data for laser fused and hot rolled sections.*