DUPLEX STAINLESS STEEL COMPOSITE BRIDGES

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DUPLEX STAINLESS STEEL COMPOSITE BRIDGES

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The first pair of bridge girders being lifted into place for the 66 m span Kolbäcksån Bridge in Hallstahammar Municipality, 2024

The main structure comprises five composite beams fabricated from lean duplex stainless steel, making it Sweden's largest stainless steel composite road bridge.

DUPLEX STAINLESS STEEL COMPOSITE BRIDGES

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SUMMARY

Stainless steels are inherently corrosion resistant. In the presence of oxygen, a tightly adherent protective layer of chromium oxide spontaneously forms on their surface, which means they can perform satisfactorily in a wide range of environments without protective coatings. This intrinsic characteristic of stainless steel is particularly important for bridges, which often need a long service life with minimum maintenance in aggressive environments.

Design lives of 120 years can be anticipated for duplex stainless steel composite bridges with very low maintenance requirements for the stainless steel girders, even in demanding environmental circumstances such as coastal locations or where de-icing salts are used.

This Design Guide gives provisions which extend and modify the application of the Eurocode design rules for carbon steel to cover I-girder composite bridges in which the steelwork is made of duplex stainless steel and the concrete deck slab is reinforced using austenitic, duplex or carbon steel reinforcing bars. The provisions given in this Design Guide are focused on the multi-girder and ladder deck forms of construction, but the principles can be extended to other forms of bridge construction.

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INTRODUCTION

1.1 Why use stainless steel for composite beams

Stainless steels are inherently corrosion resistant. In the presence of oxygen, a tightly adherent protective layer of chromium oxide spontaneously forms on their surface, which means they can perform satisfactorily in a wide range of environments without protective coatings. This intrinsic characteristic of stainless steel is particularly important for bridges, which often need a long service life with minimum maintenance in aggressive environments.

There is a wide range of stainless steels with varying levels of corrosion resistance and strength. Design lives of 120 years can be anticipated for bridges with very low maintenance requirements for the stainless steel girders, even in demanding environmental circumstances such as coastal locations or where de-icing salts are used.

The two families of stainless steel most widely used in bridges are austenitic and duplex stainless steels:

Austenitic stainless steels have a yield strength of around 220 N/mm^2 and can be strengthened by cold working. They have very high ductility and are readily weldable. They exhibit excellent toughness even down to very low temperatures.

Duplex stainless steels, sometimes called austenitic-ferritic stainless steels, have a mixed microstructure of austenite and ferrite. They are stronger than austenitic stainless steels, with a yield strength of around 460 N/mm^2 and are suitable for a broad range of corrosive environments. They can also be strengthened by cold working. They have excellent toughness and good weldability.

Both austenitic and duplex stainless steels exhibit fatigue resistance which is at least as good as that of carbon steel.

Duplex stainless steels are the most widely used for the structural components of a bridge due to their superior strength and excellent corrosion resistance, while austenitic stainless steels are mainly used for non-structural components on bridges. Austenitic stainless steel may also be used for the shear connectors or as reinforcing bars for the concrete deck slab.

Duplex stainless steels can be categorised as lean (i.e. with lower alloying additions), standard and super duplex stainless steels (i.e., with higher alloying additions). All duplex stainless steel grades have very similar mechanical and physical properties. Lean and standard duplex stainless steels have been used for the structural components of several bridges in Europe.

Duplex stainless steel mill finishes are dull, with low reflectivity, although a range of special finishes can be applied

The initial material cost of stainless steel is considerably higher than that of carbon steel. However, there may be some initial cost savings associated with not needing corrosion resistant coatings. Moreover, the superior strength of duplex grades over typical S355 carbon steel may lead to weight savings, which could help to offset the higher material cost. A reduction in weight may also lead to other benefits such as faster welding and easier handling and installation.

Eliminating the need for coating maintenance or component replacement due to corrosion leads to long term maintenance cost savings that can far exceed the initial cost difference between stainless steel and carbon steel. This is particularly important for a bridge over a railway, in busy traffic areas or low water crossings, where access to maintenance is limited or where the cost associated with the temporary closure of the line under the bridge is high[1]. Indirect benefits of reduced maintenance for vehicular bridges include less traffic disruption and a reduction in the greenhouse gas particulate emissions associated with standing traffic, as well as reduced maintenance worker hazards. The greater the area to be painted (i.e. the higher the surface area/tonne), the more cost-effective stainless steel becomes. Pedestrian bridges generally have higher surface area/tonne than road bridges.

Although weathering steel also offers low maintenance advantages, an additional benefit of stainless steel is that graffiti can be cleaned off more easily from the surface of stainless steel than from weathering steel.

Different rules are needed for designing stainless steels because they have no definite yield point, show an early departure from linear elastic behaviour, and exhibit pronounced strain hardening. These nonlinear stress strain characteristics impact certain aspects of the structural behaviour, for example local and global buckling responses differ from those of carbon steel. For this reason, it is not correct to apply certain carbon steel design provisions directly to stainless steel members.

Since stainless steel is more costly than carbon steel, the balance between the cost of material versus cost of fabrication is different, hence some optimization in design to reduce weight (and hence material cost) may be worthwhile.

1.2 Scope

This Design Guide gives provisions which extend and modify the application of the Eurocode design rules for carbon steel to cover duplex stainless steel I-girder composite bridges, focusing on the multi-girder and ladder deck forms of construction such as those shown in Figure 1.1, in which the shear connection between the steel beams and the concrete slab is achieved through welded headed studs. However, the principles given in this Design Guide can be extended to other forms of bridge construction. The Design Guide also gives provisions for designing welded and bolted connections made with austenitic or duplex stainless steel bolting components, and in which all connected elements are made of duplex stainless or duplex stainless steel connected to a dissimilar metal, such as carbon steel.

The provisions are based on the rules in EN 1993 (Eurocode 3) for steel beams, and EN 1994 (Eurocode 4) for composite beams. Where complementary information is given because the scope of EN 1994 is insufficient, design equations are based on the principles of the Eurocodes. As a result, this publication may be used as NCCI (non-contradictory, complementary information) as part of a design 'in accordance with the Eurocodes'. It is the expectation of the authors that the principles of this guidance will be included in a future version of Eurocode 4.

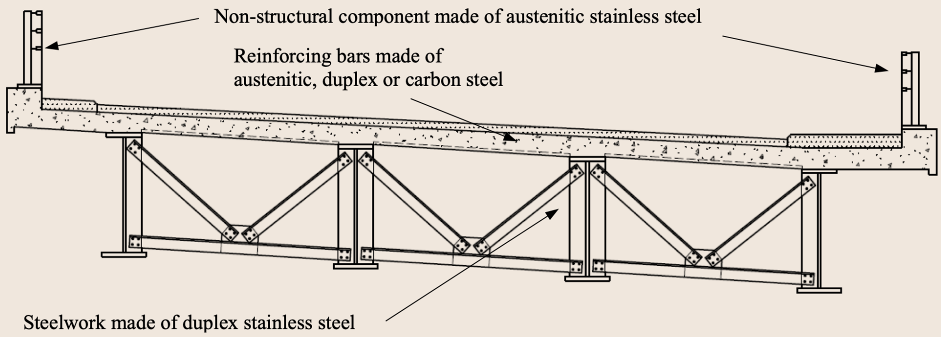
At the time of writing this Design Guide, some Second Generation Eurocodes have already been published. However, they should not yet be used as their National Annexes have not yet been published. Additionally, cross-references to other Eurocodes may relate to Second Generation documents that are not yet available. It is expected that the First Generation Eurocodes will be withdrawn on 30 March 2028.

Unless otherwise indicated, all Eurocode references in this Design Guide are to the versions of the Eurocode (First Generation) that were current at the time of writing. Changes to these rules in the Second Generation Eurocodes are included in grey boxes.

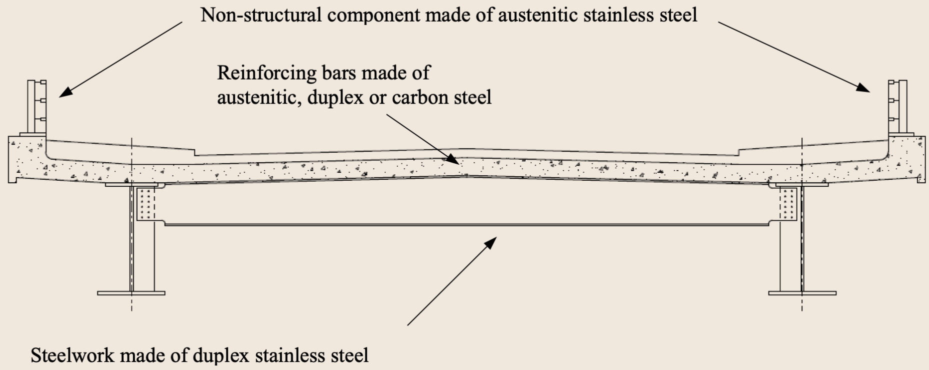
There are some discrepancies between the terminology and symbols used in different Eurocode parts. When this occurs, this Design Guide generally adopts the terminology or symbol used in EN 1994-2.

This Design Guide covers the design of conventionally welded or laser welded members. Hot-rolled sections are not covered because these types of sections are not currently available in duplex stainless steel.

The design of composite columns is outside the scope of this Design Guide.



(a) Cross section of a typical multi-girder bridge



(b) Cross section of a typical ladder deck bridge

Figure 1.1 Typical duplex stainless steel composite bridges

At the time of writing this Design Guide, a new form of composite construction for bridges has emerged in which the shear connection between the steel beam and the concrete slab is achieved through encased composite dowels cut from hot rolled steel plates, sections or bars, and a CEN Technical Specification (CEN/TS 1994-1-102[2]) for the design of this type of shear connection is being developed. Although not covered in this Design Guide, this form of construction is also expected to be suitable for duplex stainless steel composite bridges.

BASIS OF DESIGN

According to EN 1994-2[3], the design of steel-concrete composite bridges should be carried out in accordance with the requirements of EN 1990[4]. The basic requirements of EN 1990 are deemed to be satisfied for composite structures when the following are applied together:

- limit state design in conjunction with the partial factor method in accordance with EN 1990;
- actions in accordance with EN 1991[5];
- combination of actions in accordance with EN 1990 and;
- resistances, durability and serviceability in accordance with EN 1994-2.

The requirements in EN 1994-2 are also applicable to duplex stainless steel composite bridges, with the additions or modifications indicated in this Design Guide.

2.1 Verification by the partial factor method

Partial factors needed for the design of duplex stainless steel composite bridges are given in Table 2.1. The UK National Annexes adopt the values recommended in the Eurocodes.

For the design of shear connections made with austenitic stainless steel studs, it is recommended to use the same value for γ_V recommended for carbon steel studs (i.e. $\gamma_V = 1.25$).

The design of stainless steel reinforcement used for the concrete deck slab should be based on the same partial factor used for carbon steel reinforcement ($\gamma_S = 1.15$).

EN 1993-1-4[6] does not cover the design of stainless steel slip resistant bolted connections. However, design rules are given in the Second Generation of EN 1993-1-4[7] (see Section 10.1), in which the partial factors are the same as those recommended in EN 1993-1-8[8] for carbon steel slip resistant bolted connections.

Different Eurocode parts use the partial factors in slightly different ways: for example, while in Eurocode 3, the partial factors are generally applied to nominal resistances to obtain design resistances, in Eurocode 2 and Eurocode 4 these are generally applied to characteristic or nominal strengths to obtain design strengths. Thus, based on the nomenclature used in EN 1994-2, the design strength of the structural materials covered in this Design Guide are as follows:

- the design strength of the duplex stainless steel plate material (for the steel section) is given by:

$$f_{\mathrm{yd}} = \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{Mi}}}$$

  i = 0, 1 or 2 depending on the failure mode
- the design strength of the reinforcing bars is given by:

$$f_{\mathrm{sd}} = \frac{f_{\mathrm{sk}}}{\gamma_{\mathrm{S}}}$$

  the design strength of the concrete is given by:

$$f_{\mathrm{cd}} = \frac{f_{\mathrm{ck}}}{\gamma_{\mathrm{C}}}$$

Where:

f_{y} is the nominal yield strength of the duplex stainless steel plate material

f_{sk} is the characteristic yield strength of the stainless or carbon steel reinforcing bars

f_{ck} is the characteristic cylinder strength of the normal or lightweight concrete

Note that for the shear studs and the bolts, design strengths are not given because in these cases, the partial factors are applied to the resistance.

Table 2.1 Recommended values of partial factors for the design of duplex stainless steel composite bridges

<table>
<tr><th colspan="2">Material</th><th>Symbol</th><th>Recommended value</th><th>Recommended in:</th></tr>
<tr><td colspan="2">Concrete</td><td>γ_{C}</td><td>1.50</td><td>EN 1992-1-1</td></tr>
<tr><td rowspan="3">Structural stainless steel</td><td>Resistance of cross-sections to excessive yielding including local buckling</td><td>γ_{M0}</td><td>1.10</td><td>EN 1993-1-4</td></tr>
<tr><td>Resistance of members to instability assessed by member checks</td><td>γ_{M1}</td><td>1.10</td><td>EN 1993-1-4</td></tr>
<tr><td>Resistance of cross-sections in tension to fracture</td><td>γ_{M2}</td><td>1.25</td><td>EN 1993-1-4</td></tr>
<tr><td colspan="2">Steel reinforcement (reinforcing bars)</td><td>γ_{S}</td><td>1.15</td><td>EN 1992-1-1</td></tr>
<tr><td colspan="2">Shear connection</td><td>γ_{V}</td><td>1.25</td><td>EN 1994-2</td></tr>
<tr><td colspan="2">Stainless steel welded connection</td><td>γ_{M2}</td><td>1.25</td><td>EN 1993-1-4</td></tr>
<tr><td colspan="2">Stainless steel bearing type bolted connection</td><td>γ_{M2}</td><td>1.25</td><td>EN 1993-1-4</td></tr>
<tr><td rowspan="2">Stainless steel slip resistant bolted connections</td><td>Ultimate limit state</td><td>γ_{M3}</td><td>1.25</td><td>EN 1993-1-4 Gen 2</td></tr>
<tr><td>Serviceability limit state</td><td>$\gamma_{\mathrm{M3,ser}}$</td><td>1.10</td><td>EN 1993-1-4 Gen 2</td></tr>
</table>

MATERIALS

3.1 Duplex stainless steel plate material

3.1.1 Stress-strain characteristics

Duplex stainless steels exhibit more rounded stress-strain characteristics than carbon steel, with no well-defined yield strength. For this reason, the 'yield' strength of duplex stainless steels is conventionally quoted in terms of a proof strength defined as the 0.2% proof strength, also referred to as the 0.2% offset yield strength. This is illustrated in Figure 3.1 which compares the stress-strain characteristic (for strains of up to 0.75%) of a representative duplex stainless steel grade and an S355 carbon steel grade. The full range of stress-strain characteristics of these steels is compared in Figure 3.2, which shows that due to the lack of a yield plateau in duplex stainless steels, strain hardening manifests gradually from the onset of yielding.

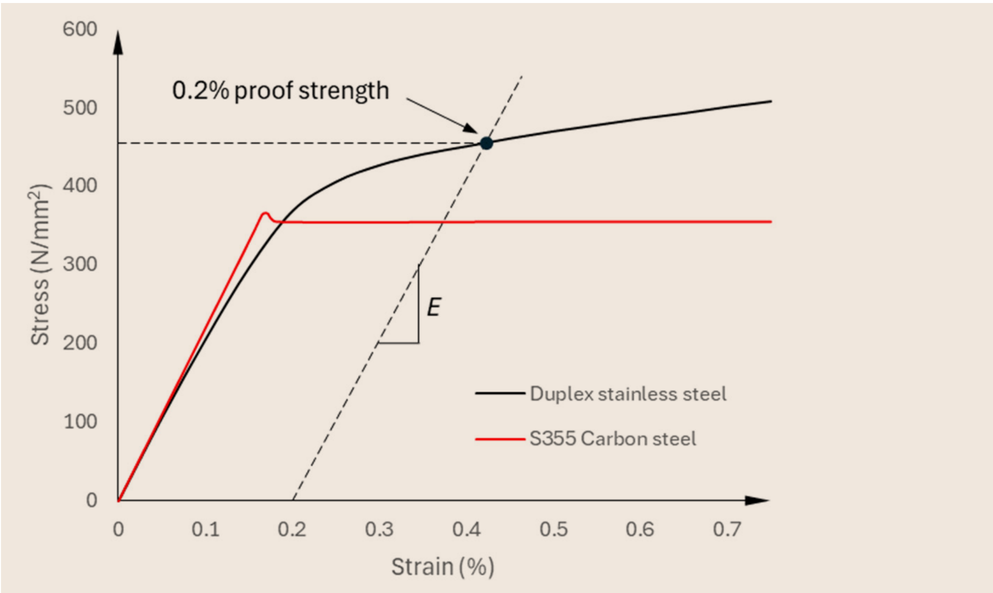


Figure 3.1 Stress-strain curves for typical duplex stainless steel and carbon steel (low strain range)

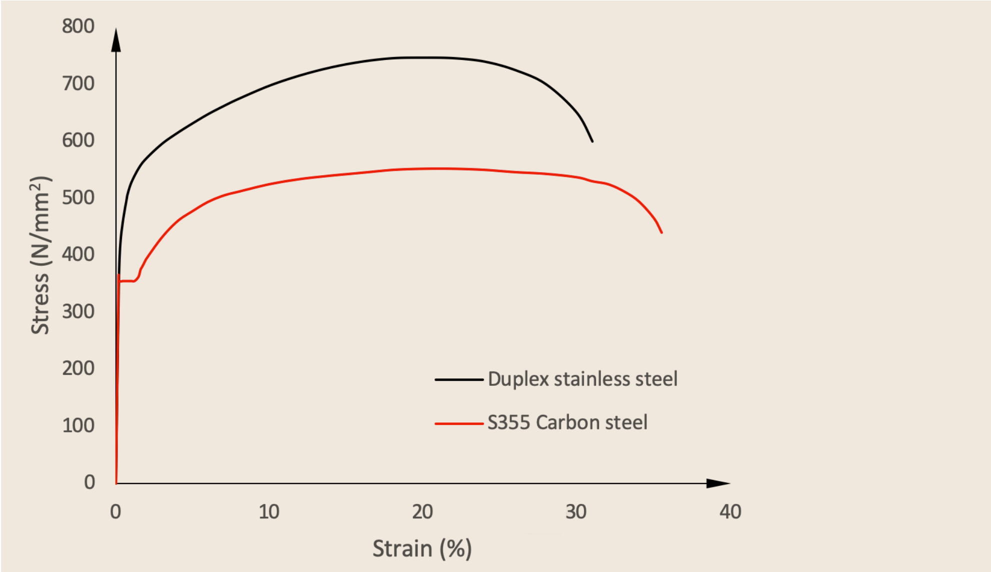


Figure 3.2
Full range stress-strain curves for typical duplex stainless steel and carbon steel

3.1.2 Strength

Table 3.1 gives the strengths of the duplex stainless steels which are covered in EN 1993-1-4[6]. The most likely grades for use in bridge girders are the lean and standard duplex grades.

Table 3.1
Nominal values of the yield strength f_y and the ultimate tensile strength f_u for duplex stainless steel grades used in bridges, in accordance with EN 10088 (N/mm²)

Grade		Product form			
		Hot rolled strip		Hot rolled plate	
		Nominal thickness t $t \le 13.5$ mm		Nominal thickness t $t \le 75$ mm	
		f_y	f_u	f_y	f_u
Lean duplex	1.4062	480 a	680 a	450	650
	1.4162	480 a	680 a	450	650
	1.4362	400	650	400	630
	1.4482	480 a	660 a	450	650
	1.4662	550 b	750 b	480	680
Standard duplex	1.4462	460	700	460	640
Super duplex	1.4410	530	750	530	730
	1.4501	-	-	530	730
	1.4507	530	750	530	730

The nominal values of f_y and f_u in this table may be used in design without taking special account of anisotropy or strain hardening effects.

1.4482, 1.4062 and 1.4662 are only covered in EN 10088-2[10] and -3[11] .

a $t \le 10$ mm   b $t \le 13$ mm

According to EN 1993-1-4, the nominal value of the yield strength f_y and ultimate tensile strength f_u of hot-rolled strip and plate made from duplex stainless steel are taken as the minimum specified value for the 0.2% proof strength ($R_{p0.2}$) and ultimate tensile strength (R_m) given in EN 10088-4[9]. These values apply to material in the annealed condition. They consider a small level of work hardening introduced during rolling of the strip or plate. This effect is reflected by the different minimum specified strength values given for the different product forms and/or thickness.

3.1.3 Ductility

According to EN 1993-2[12], the steel should meet the ductility requirements specified in EN 1993-1-1[13]. These requirements involve a minimum ratio f_u/f_y of 1.10, a minimum elongation at fracture of 15% and a minimum ultimate strain of $15\varepsilon_y$, where ε_y is the yield strain of the steel given by f_y/E. All these requirements are met by the duplex stainless steels covered in this Design Guide, as indicated by Clause 2.1.2(2) of EN 1993-1-4. This is illustrated in Figure 3.2, which shows that the ductility of duplex stainless steels is similar or better than that of carbon steels. According to EN 10088-4, duplex grades 1.4362, 1.4410 and 1.4507 have a minimum elongation at fracture of 20%, while for the other grades listed in Table 3.1 the minimum elongation at fracture ranges between 25-30%. The minimum f_u/f_y ratio for these duplex grades is 1.36.

In the Second Generation of EN 1993-1-1[14] , the ductility requirements are revised. Requirements vary depending on the type of global analysis that is carried out, as follows:

- For plastic global analysis: $f_u/f_y \geq 1.10$ and elongation at failure not less than 15 %.
- For elastic global analysis: $f_u/f_y \geq 1.05$ and elongation at failure not less than 12 %.

The requirement for uniform elongation is removed.

These ductility requirements are adopted in the Second Generation of EN 1993-1-4 for stainless steel.

3.1.4 Fracture toughness

Steels used in bridges are required to have good toughness properties to avoid brittle fracture under low service temperatures, which in the UK can be as low as -20 °C. The toughness-temperature relationship of duplex stainless steels is similar to that of carbon steels, i.e.: high toughness at ambient temperatures with the material exhibiting a ductile behaviour; low toughness at very low temperatures with the material exhibiting a brittle behaviour; and a transition region in which the toughness decreases with decreasing temperatures and the failure mode changes from ductile to brittle.

Lean duplex stainless steels demonstrate adequate toughness at service temperatures down to -40 °C, while the more highly alloyed duplex grades, such as 1.4462, show even better toughness properties. Contrary to carbon steels, duplex stainless steels are not specified with different sub-grades for different toughness properties. The fracture toughness of most duplex stainless steel grades is comparable to that of carbon steel grade S355K2, and can meet the toughness required for typical bridges in the UK.

According to EN 1993-2[12], steels used in bridges need to satisfy the conditions given in EN 1993-1-10[15], which includes a procedure for determining the maximum allowable plate thickness of the steel based on the lowest service temperature of the structure, the stress in the component, and the fracture toughness required. The maximum allowable thicknesses are presented in EN 1993-1-10 in a tabulated form.

In the Second Generation of EN 1993-1-10[16], the maximum allowable thicknesses are dependent on the execution class. The general procedure in EN 1993-1-10 for determining the maximum allowable thickness of carbon steel plates is also applicable to duplex stainless steels providing the values of maximum thickness in Table 3.2 are used. Table 3.2 is applicable to Execution Class EXC 3 and EXC 4. Execution Class EXC 3 is the most commonly used class for bridges. This table is included in the Second Generation of EN 1993-1-4[7] and applies to elements subject to fatigue loading. It should be noted that duplex stainless steels are classified into different toughness requirement classes (i.e. TR1 to TR4)

Table 3.2 Maximum permissible values of element thickness t in mm for duplex stainless steel for Execution Class EXC3 and EXC4

Grade	f_y N/mm² (hot rolled plate)	Toughness requirement	*KV*		Reference Temperature T_{Ed} [°C]																				
			at T	J_{min}	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
			[°C]	[J]	$\sigma_{Ed} = 0.75f_y\ (t)$							$\sigma_{Ed} = 0.50f_y\ (t)$							$\sigma_{Ed} = 0.25f_y\ (t)$						
1.4062, 1.4162, 1.4482	450	TR1	-20	40	80	65	55	45	35	30	20	130	110	95	80	65	55	45	200	180	155	135	115	100	85
		TR2	-30	40	95	80	65	55	45	35	30	155	130	110	95	80	65	55	200	200	180	155	135	115	100
1.4662	480	TR1	-20	40	75	65	50	40	35	25	20	125	105	90	75	60	50	40	200	175	150	130	110	95	80
		TR2	-30	40	95	75	65	50	40	35	25	150	125	105	90	75	60	50	200	200	175	150	130	110	95
1.4362	400	TR3	-40	40	125	105	90	75	60	50	40	195	165	145	120	105	85	70	200	200	200	190	165	145	125
		TR4	-50	40	145	125	105	90	75	60	50	200	195	165	145	120	105	85	200	200	200	200	190	165	145
1.4462	460	TR3	-40	40	115	95	80	65	55	45	35	175	150	130	110	90	75	65	200	200	200	175	155	130	115
		TR4	-50	40	135	115	95	80	65	55	45	200	175	150	130	110	90	75	200	200	200	200	175	155	130
1.4410, 1.4501, 1.4507	530	TR3	-40	40	105	85	70	60	45	40	30	160	140	115	100	80	70	55	200	200	190	165	145	120	105
		TR4	-50	40	125	105	85	70	60	45	40	190	160	140	115	100	80	70	200	200	200	190	165	145	120

NOTE 1: Linear interpolation can be used. Most applications require σ_{Ed} values between $0.75f_y\ (t)$ and $0.50f_y\ (t)$.

$\sigma_{Ed} = 0.25f_y\ (t)$ is given for interpolation purposes, and for use for elements under compression stress.

NOTE 2: Maximum permissible values of element thickness t are restricted to 200 mm. The values for thickness in this table can exceed the upper thickness limit in the relevant product standards.

3.1.5 Through thickness properties

EN 1993-2 stipulates that steels with improved through-thickness properties conforming to EN 10164[17] should be used where required by EN 1993-1-10. However, there is no evidence that suggests through-thickness lamellar tearing occurs in stainless steels even though these are not covered in EN 10164. It is also worth noting that this type of failure is not critical in typical bridges, and in most cases can be prevented by good design to avoid unusual joint details[18].

The Second Generation of EN 1993-1-4 includes a clause that specifically states that the duplex stainless steels do not need to be specified with improved through-thickness properties because they do not exhibit lamellar tearing.

3.1.6 Design values of other material properties

According to EN 1993-1-4, the following values for the material properties should be used for duplex stainless steels:

- Modulus of elasticity $E = 200 \times 10^3$ N/mm^2
- Shear modulus $G = 77 \times 10^3$ N/mm^2
- Poisson's ratio $\nu = 0.3$

In addition, the Second Generation of EN 1993-1-4 specifies the following property:

- Coefficient of linear thermal expansion $\alpha_T = 13 \times 10^{-6}$ per °C for ($T \leq 100$ °C)

and the material parameter ε is defined as follows:

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{3.1}$$

The definition of ε given by Equation 3.1 differs from the definition used in EN 1993-1-4, in which

$$\varepsilon = \sqrt{\frac{235}{f_y}\frac{E}{210\,000}}$$

Although the Second Generation of EN 1993-1-4 specifies $\alpha_T = 13 \times 10^{-6}$ per °C for ($T \leq 100$ °C), it is recommended to use the coefficient of linear thermal expansion of the concrete (i.e. $\alpha_T = 10 \times 10^{-6}$ per °C) when calculating the change in length of the bridge and carrying out the global analysis for the determination of stresses in the composite structures. This recommendation is aligned with the design approach in EN 1994-2, in which the coefficient of linear thermal expansion of the concrete is also used when performing these calculations.

3.1.7 Material model for duplex stainless steel

EN 1993-1-4 recommends that the stress-strain characteristic of duplex stainless steels is represented using the Ramberg-Osgood material model, given by:

$$\varepsilon = \frac{\sigma}{E} + 0.002\left[\frac{\sigma}{f_y}\right]^n \qquad \text{for } \sigma \leq f_y \tag{3.2}$$

$$\varepsilon = 0.002 + \frac{f_y}{E} + \frac{\sigma - f_y}{E_y} + \varepsilon_u\left[\frac{\sigma - f_y}{f_u - f_y}\right]^m \qquad \text{for } f_y < \sigma \leq f_u \tag{3.3}$$

where:

σ is the engineering stress

ε is the engineering strain

E, f_y and f_u are the nominal elastic modulus, yield strength and ultimate tensile strength

n is the strain hardening parameter which is given as 5 for duplex stainless steel grades 1.4462 and 1.4362

The Second Generation of EN 1993-1-4 recommends a value of n = 8 for all duplex stainless steel grades.

E_y is the tangent modulus of the stress-strain curve at the yield strength defined as:

$$E_y = \frac{E}{1 + 0.002n\left[\frac{E}{f_y}\right]} \tag{3.4}$$

ε_u is the ultimate strain, corresponding to the ultimate strength f_u, which may be obtained from the approximation:

$$\varepsilon_u = 1 - \frac{f_y}{f_u} \tag{3.5}$$

$$m = 1 + 2.8\frac{f_y}{f_u} \tag{3.6}$$

Figure 3.3 illustrates the key parameters in the material model.

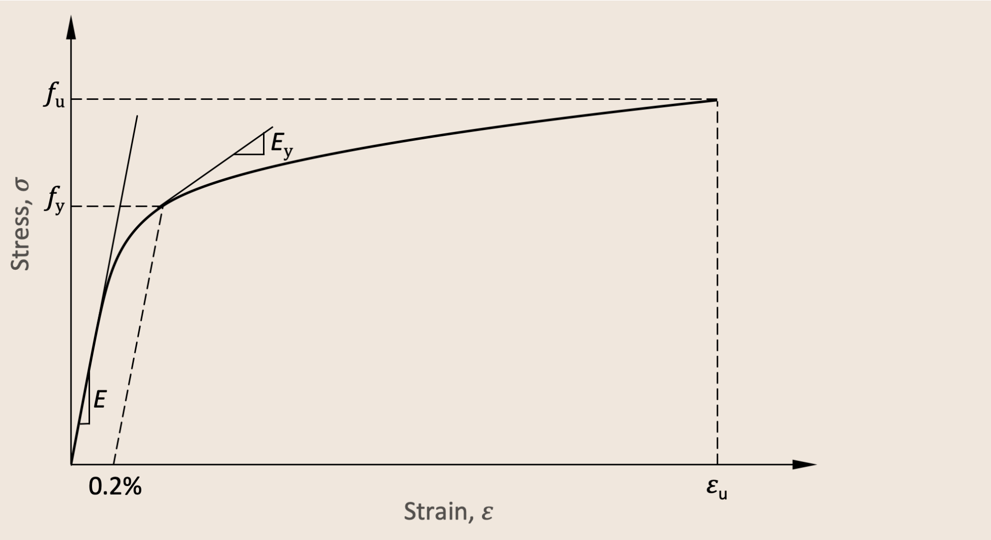


Figure 3.3
Key parameters in material model

3.2 Studs

Based on the study conducted in Ref.[19], it is recommended that the shear studs used with duplex stainless steel composite beams are made of austenitic stainless steel grades 1.4301 or 1.4303. The studs should meet the requirements in ISO 13918 for SD3 studs.

Duplex stainless steel studs are not available from steel stockists and will need to be custom made. Their structural performance (in terms of load-slip behaviour of the shear connection) has not yet been thoroughly investigated. If this type of stud is specified, their structural performance should be demonstrated by testing.

It is not recommended to weld carbon steel studs to a duplex beam due to the difficulty in achieving a reliable weld (see Section 11.6.1 for further information).

3.3 Carbon steel and stainless steel reinforcement

The properties of carbon steel reinforcement should be obtained from EN 1992-1-1[20], which covers reinforcement in five strength classes from B400 (with a characteristic yield strength $f_{sk} \geq 400$ N/mm^2) up to B600 ($f_{sk} \geq 600$ N/mm^2).

EN 1992-1-1 allows the use of stainless steel reinforcing bars. However, it does not give specific requirements for the material properties. Instead, it says that any steel used for the reinforcing bars should meet the requirements for carbon steel reinforcement.

EN 1992-1-1 classifies the reinforcement into three ductility classes based on the tensile to yield strength ratio $(f_t/f_y)_k$ and the strain at maximum force ε_{uk} (i.e. Classes A, B and C). However, for bridges, EN 1992-2 recommends that only reinforcement of Class B or C is used. The UK National Annex to EN 1992-2 also requires that only reinforcement with ductility class B or C be used for bridges.

The Second Generation of EN 1992-1-1[21] introduces the new strength class B700, covering the use of steel reinforcement with $f_{sk} \geq 700$ N/mm^2, and gives specific property requirements for stainless steel reinforcement. The strength and ductility requirements for stainless steel reinforcement are the same as those for carbon steel, but there are some differences in the required minimum relative rib area and the fatigue stress. The strength and ductility properties for stainless steel reinforcement used in bridges are given in Table 3.3 and Table 3.4, respectively. For the design value of the modulus of elasticity of the stainless steel reinforcement, it is possible to assume a value of 200 000 N/mm^2, while for the coefficient of thermal expansion, a value of 10×10^{-6} per K is given, which is the same value used for carbon steel reinforcement and concrete.

Table 3.3 Strength properties of stainless steel reinforcement (as given in the Second Generation of EN 1992-1-1)

Property	Reinforcing steel strength class [a]					
	B400	B450	B500	B550	B600	B700
Yield strength $R_{p0.2k}$ (N/mm²) (5 % quantile) [b,c]	≥ 400	≥ 450	≥ 500	≥ 550	≥ 600	≥ 700
Fatigue stress range $2\sigma_a$ (N/mm²) for $N \geq 5 \times 10^6$ cycles based on a stress ratio $\sigma_{min} / \sigma_{max} = 0.2$ for strength class B500 only (10 % quantile) [b,c]	160 for $\phi \leq 16$ mm 140 for 12 mm $< \phi \leq$ 16 mm 130 for $\phi > 16$ mm					
Minimum relative rib area $f_{R,min}$ (5 % quantile) [c]	0.039 for $\phi \leq 6$ mm 0.045 for 6.5 mm $\leq \phi \leq$ 8.5 mm 0.052 for 9 mm $\leq \phi \leq$ 10.5 mm 0.056 for 11 mm $\leq \phi \leq$ 50 mm					

[a] All strength classes apply unless the National Annex excludes specific classes.

[b] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles that is to be confirmed by testing.

[c] The quantile percentile indicates the maximum % of test results that are allowed to be below the characteristic value.

Table 3.4 Ductility properties of stainless steel reinforcement (as given in the Second Generation of EN 1992-1-1)

Properties for weldable reinforcement	Ductility Class	
	B	C
Stress ratio $R_m / R_{p0.2k}$, where R_m should be limited to the stress at 7% elongation (10 % quantile) [a]	≥ 1.08	≥ 1.15 < 1.35
Tensile yield strength ratio of $R_{e,act} / R_{p0.2,nom}$ (10 % quantile) [a]	≤ 1.3	≤ 1.3
Elongation at maximum load A_{gt} (%) (10 % quantile) [a]	≥ 5.0	≥ 7.5
Bendability	Pass bend and/or rebend test according to relevant standard for stainless steel reinforcing. For rebending of reinforcing stainless steel on site see requirements in EN 13670[22].	

[a] The quantile percentile indicates the maximum % of test results that are allowed to be below the characteristic value.

3.3.1 Material model for reinforcement

The stress-strain behaviour of carbon steel and stainless steel reinforcement may be represented by either of the two material models shown in Figure 3.4. If the material model with an inclined post-elastic branch is used, the maximum strain should be limited to the design strain ε_{ud}, but if the material model with horizontal post-elastic branch is used, no limit is needed on the maximum strain.

The parameter k in Figure 3.4 is taken as the ratio between the characteristic ultimate tensile strength and the characteristic yield strength $(f_t/f_y)_k$. In EN 1992-1-1, the recommended value for ε_{ud} is $0.9\varepsilon_{uk}$.

In the Second Generation of EN 1992-1-1, the design strain is limited to $\varepsilon_{ud} \leq \varepsilon_{uk}/\gamma_S$ both for carbon steel and stainless steel reinforcement.

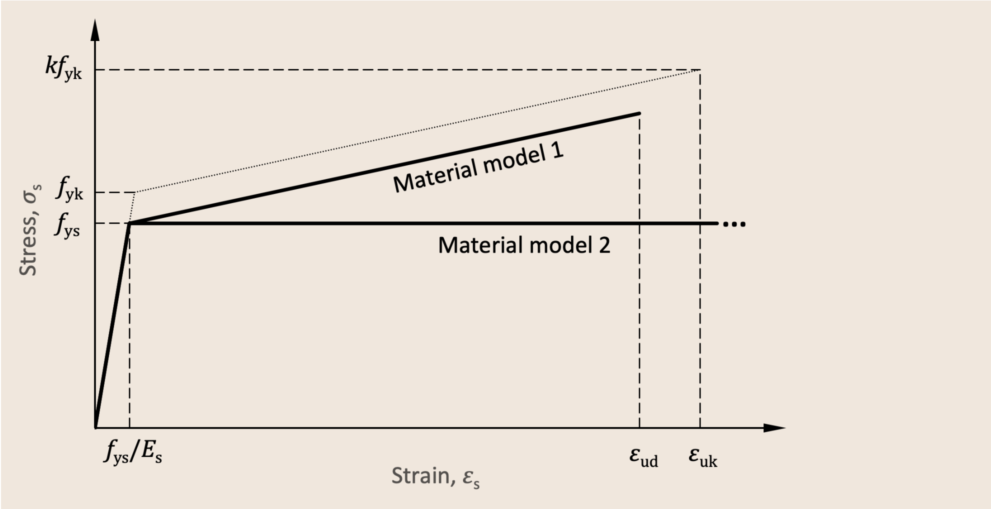


Figure 3.4 Material model for carbon steel and stainless steel reinforcement

3.4 Concrete

The properties of normal concrete and lightweight concrete should be obtained from EN 1992-1-1. This standard covers a wide range of concrete strength classes. However, only concrete strength classes ranging from C20/25 to C60/75 for normal concrete and LC20/22 to LC60/66 for lightweight concretes are permitted in EN 1994-2 for composite bridges.

3.5 Stainless steel bolting assembly

According to EN 1993-1-4, stainless steel bolts and nuts should conform to EN ISO 3506-1[23] and EN ISO 3506-2[24]. Washers should also be made of stainless steel and should conform to EN ISO 3506-7[25], EN ISO 7089[26] or EN ISO 7090[27], as appropriate.

In EN ISO 3506-1 and EN ISO 3506-2, bolts and nuts are designated by their stainless steel grade and their property class. The designation of the stainless steel grade consists of one letter indicating the stainless steel alloy family (e.g. A for austenitic and D for duplex) and a digit which specifies the range of chemical compositions within the stainless steel alloy family. Thus, the digit also gives an indication of the level of corrosion resistance of the bolt (see Table 4.3). The property class is indicated by a number corresponding to 1/10 of the specified minimum tensile strength of the bolt (in N/mm^2). The nominal yield strength f_{yb} and ultimate tensile strength f_{ub} of the stainless steel bolts covered in this Design Guide are given in Table 3.5. For slip resistant connections, only the bolts in Table 3.5 of property classes 80 and 100 may be used.

It is important to use the right lubricant in a stainless steel bolting assembly to reduce the risk of galling, and in preloaded bolted connections, to help reach the required pretension force in the bolt by minimizing the applied torque. Not all lubricants are effective for all types of stainless steel bolting assemblies, even if the lubricant is marketed for use with stainless steel threaded parts. Lubricants containing molybdenum disulfide and polytetrafluoroethylene (PTFE) generally provide successful installation performance, and wax or other lubricants containing silver or copper powders, mica, graphite or talc may also facilitate successful installation.

Table 3.5 Nominal values of f_{yb} and f_{ub} for stainless steel bolts

Stainless steel grade to EN ISO 3506-1	Property class to EN ISO 3506-1	Yield strength f_{yb} N/mm²	Ultimate tensile strength f_{ub} N/mm²
A2, A3, A4	50	210	500
A2, A3, A4, A5, D2, D4, D6, D8	70	450	700
A2, A3, A4, A5, A8, D2, D4, D6, D8	80	600	800
A4, A5, A8, D2, D4, D6, D8	100	800	1000

A: austenitic; D: duplex.

3.6 Welding consumables

In addition to the general requirements for welding consumables in EN 1993-1-8, EN 1993-1-4 states that they should be capable of producing a weld with a corrosion resistance that is adequate for the service environment. This is deemed to be satisfied if the corrosion resistance of the deposited metal and weld metal is equal to, or greater than, that of the material being welded.

The designations of stainless steel welding consumables that may be used in duplex stainless steel composite bridges are given in EN ISO 3581[28], EN ISO 14343[29] and EN ISO 17633[30]. These standards provide two approaches for classifying the consumables (i.e. ‘nominal composition’ and the ‘alloy type’ approaches).

Table 3.6 gives suitable welding consumables for different duplex stainless steel grades. In the table, the classification designation of the welding consumable is based on the nominal composition approach, which uses integers to give an indication of the level of chromium, nickel and molybdenum, and symbols to indicate the presence of other elements in the all-weld metal deposit. Advice on the selection of suitable consumables should be sought from manufacturers of stainless steel and consumables.

Table 3.6 Examples of suitable welding consumables for different duplex stainless steel grades

Duplex grade of parent material	Designation of nominal composition of welding consumable in accordance with EN ISO 3581, EN ISO 14343 or EN ISO 17633
Lean duplex: 1.4482, 1.4162 1.4362, 1.4062, 1.4662	23 7 N L or 22 9 3 N L
Standard duplex: 1.4462	22 9 3 N L
Super duplex: 1.4410, 1.4501, 1.4507	25 9 4 N L

The Second Generation of EN 1993-1-4 covers welded connections between dissimilar metals such as duplex stainless steel and carbon steel. In these types of joints, the filler metal should be over-alloyed to ensure adequate mechanical properties and corrosion resistance of the joint. Over-alloying avoids dilution of the joined elements in the fusion zone. Further information on welding duplex stainless steel to carbon steel is given in Section 11.6.2.

DURABILITY AND GRADE SELECTION

The main reason for using duplex stainless steel for the steelwork of bridges is to take advantage of its excellent corrosion resistance properties, which ensure long-term durability with minimal maintenance.

The corrosion resistance of a given stainless steel grade is predominantly dependent on its constituent elements, which means that each grade has a slightly different response when exposed to a corrosive environment. Care is therefore needed to select the most appropriate grade of stainless steel for a given application. Generally, the higher the level of corrosion resistance required, the greater the cost of the material.

The most common reasons for a metal to fail to perform as well as expected regarding corrosion resistance are:

- incorrect assessment of the environment or exposure to unexpected conditions, e.g. unsuspected contamination by chloride ions or higher than expected surface accumulations.
- 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.

Even when surface staining or corrosion occur, it is unlikely that structural integrity will be compromised. However, the user may still regard unsightly rust staining on external surfaces as a failure. As well as careful material grade selection, good detailing and workmanship can significantly reduce the likelihood of staining and corrosion; practical guidance is given in Section 11. 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 grades of stainless steel will be susceptible to localised attack. Further information on the different corrosion mechanisms a steel structure may be subject to, and the performance of stainless steel grades in different environments, is given in SCI publication *Design Manual for Structural Stainless Steel* (DMSSS)[31].

When a structure is exposed to a corrosive environment, the connections tend to be the part of the structure that is most affected by corrosion. This is due to their particular features that make them more prone to crevice corrosion, and galvanic corrosion, if dissimilar metals are connected. For this reason, bolting components should have a corrosion resistance equivalent to, or greater than, the corrosion resistance of the elements being joined. The bolting components do not need to belong to the same alloy family as the material being connected, i.e., austenitic stainless steel bolts may be used to connect duplex stainless steel elements providing their corrosion resistance is equivalent or greater.

Stainless steel reinforcing bars are also used to improve the durability of the concrete decks of bridges. In this case, any corrosion of the reinforcing bars is mostly due to carbonation or the presence of chlorides. The chlorides may be already present in the concrete or may enter the concrete because of exposure to seawater or de-icing salts. Stainless steel reinforcing bars may be used in conjunction with carbon steel reinforcing bars, for example, in parts of the concrete deck slab that are most exposed to a corrosive environment. It is well accepted that the any galvanic corrosion arising from contact between the embedded stainless steel and carbon steel reinforcement is negligible.

Grade selection procedures for structural components/bolts and reinforcing bars embedded in concrete are described in the following sections.

4.1 Grade selection procedure for structural components and bolts

Annex A of EN 1993-1-4[6] includes a simple grade selection procedure that can be used to select a duplex stainless steel grade for the steelwork of the bridge with adequate corrosion resistance for the service environment. The service environment of a bridge structure is mostly conditioned by its exposure to chloride salts from seawater or de-icing salts (i.e. the location and use of the bridge), whether there is a cleaning regime or whether the structural components are exposed to washing by rainwater. The grade selection procedure in Annex A of EN 1993-1-4 is only suitable for structures located within Europe, and for components that are not permanently or frequently immersed in seawater. It does not take into account the grade or product availability, or any surface finish requirement. The procedure assumes that the requirements of EN 1090-2[32] are followed in relation to welding procedures and post weld cleaning. It also assumes the avoidance (or removal and cleaning) of contamination of the stainless steel surfaces after thermal or mechanical cutting - failure to do so may reduce the corrosion resistance of welded parts.

In the Second Generation of EN 1993-1-4[7], the table giving the grades in each Corrosion Resistance Class is extended to cover bolt grades also.

A grade selection procedure for stainless steel reinforcing bars embedded in concrete is also included in the Second Generation of EN 1992-1-1[22], which relates the environmental condition and the minimum concrete cover protecting the reinforcing bars to the stainless steel grade selected.

In the grade selection procedure for structural components and bolts, first, the service environment is characterised by a Corrosion Resistance Factor (CRF), which is determined using Equation 4.1.

$$\text{CRF} = F_1 + F_2 + F_3 \tag{4.1}$$

The coefficients F_1, F_2 and F_3 are determined from Table 4.1 (which gives the risk of exposures relevant to bridges). The value of F_1 for applications on the coast depends on the particular location in Europe and is derived from experience with existing structures, corrosion test data and chloride distribution data. The wide range of environments within Europe means that in some cases the calculated CRF will be conservative.

The National Annex may specify whether a less severe CRF can be chosen when validated local operating experience or test data support such a choice. The UK National Annex permits the use of a less severe CRF when local operating experience of at least five years' duration demonstrates the suitability of a grade in the adjacent lower CRC. However, the maximum permitted improvement to the CRF is +5. The performance data should be obtained from a location less than 5 km from the proposed site and, for coastal locations, less than 1 km inland from the proposed site. Evaluation of the performance should consider the material grade, quality of surface finish, orientation of the components and exposure to airborne contaminants (particularly chlorides) to ensure these are comparable with the proposed design.

Different parts of the same structure may have different exposure conditions, indeed one part may be fully exposed and another part fully sheltered. Each exposure case should be assessed separately.

Once the CRF has been determined, the required Corrosion Resistance Class (CRC) is obtained from Table 4.2.

Grades are grouped in one of the five CRCs based on their corrosion resistance, as shown in Table 4.3 for the duplex stainless steel grades covered by EN 1993-1-4 and the austenitic and duplex stainless steel bolt grades. Lean duplexes are in CRCs II or III which may be suitable for bridges located at a distance more than 250 m from the sea. For bridges located closer to the sea, the standard duplex grade 1.4462 in CRC IV will most likely be suitable unless the structural component is not exposed to washing by rain and no cleaning regime is implemented. When this is the case, the super duplexes in CRC V, which have higher alloying additions of chromium, nickel and molybdenum, will provide the required corrosion resistance. Specification of the material by CRC and design strength, e.g. CRC II and f_y= 450 N/mm^2, is sufficient to allow the supplier to determine the actual grade from the CRC.

	F_1 Risk of exposure to chlorides from salt water or de-icing salts	
1	Internally controlled environment	
0	Low risk of exposure	M > 10 km or S > 0.1 km
-3	Medium risk of exposure	1 km < M ≤ 10 km or 0.01 km < S ≤ 0.1 km
-7	High risk of exposure	0.25 km < M ≤ 1 km or S ≤ 0.01 km
-10	Very high risk of exposure	M ≤ 0.25 km [a] North Sea coast of Germany and all Baltic coastal areas
-15	Very high risk of exposure	M ≤ 0.25 km Atlantic coastline of Portugal, Spain and France. English Channel and North Sea Coastline of UK, France, Belgium, Netherlands and Southern Sweden. All other coastal areas of UK, Norway, Denmark and Ireland. Mediterranean coast
	NOTE: M is distance from the sea and S is distance from roads with de-icing salts. [a] The distance M < 0.25 km assumes the structure is not sheltered by ground topography. If the topography provides partial shelter to the structure, experience shows a grade from one lower class may be used. Examples of sheltering include structures built over inlets or estuaries with limited wave heights and permanent physical barriers within the 0.25 km zone.	
	F_2 Risk of exposure to sulphur dioxide	
0	Low risk of exposure	<10 µg/m³ average gas concentration
-5	Medium risk of exposure	10 - 90 µg/m³ average gas concentration
	NOTE: For European coastal environments the sulphur dioxide concentration is usually low. For inland environments the sulphur dioxide concentration is either low or medium. Sulphur dioxide concentration may be evaluated according to the method in ISO 9225.	
	F_3 Cleaning regime or exposure to washing by rain	
0	Fully exposed to washing by rain	
-2	Specified cleaning regime	
-7	No washing by rain or no specified cleaning	
	If $F_1 + F_2 \geq 0$, then $F_3 = 0$.	
	NOTE: If the component is to be regularly inspected for any signs of corrosion and cleaned, this should be made clear to the user in written form. The inspection, cleaning method and frequency should be specified. The more frequently cleaning is carried out, the greater the benefit. The frequency should not be less than every 3 months. Where cleaning is specified, it should apply to all parts of the structure, and not just those easily accessible and visible.	

Table 4.1 Determination of corrosion resistance factor CRF (CRF = $F_1 + F_2 + F_3$) for bridges (extracted from Table A.1 of EN 1993-1-4)

Corrosion Resistance Factor (CRF)	Corrosion Resistance Class (CRC)
CRF = 1	I
0 ≥ CRF > -7	II
-7 ≥ CRF > -15	III
-15 ≥ CRF ≥ -20	IV
CRF < -20	V

Table 4.2 Determination of corrosion resistance class CRC (Table A.2 of EN 1993-1-4)

Corrosion resistance class CRC [a,b,c]				
I	II	III	IV	V
Duplex grades for structural components				
	1.4482	1.4162	1.4462	1.4410
		1.4662		1.4501
		1.4362		1.4507
		1.4062		
Austenitic and duplex stainless steel bolt grades				
	A2, A3 or D2	A4, A5 or D4	A8 or D6	A8 or D8

[a] A grade from a higher class may be used in place of the class indicated by the CRF.

[b] The Corrosion Resistance Classes should only be used for structural applications and in accordance with this grade selection procedure.

[c] The corrosion resistance of the bolts should be equivalent to, or greater than, the corrosion resistance of the parent metal.

Table 4.3 Grades in each corrosion resistance class CRC (extracted from Table A.3 of EN 1993-1-4

4.2 Grade selection procedure for reinforcing bars embedded in concrete

According to the procedure in the Second Generation of EN 1992-1-1[22], a suitable stainless steel grade for the reinforcing bars can be determined using Table 4.4 based on the Exposure Class and the Exposure Resistance Class ERC of the concrete component (e.g. the concrete deck slab in the bridge), and the concrete cover for the stainless steel reinforcement.

The concrete covers in Table 4.4 are intended for a design service life of 50 years. For a design service life of 100 years, which may be more applicable to bridges constructed using stainless steel, the Second Generation of EN 1992-1-1 recommends that the concrete cover should be increased by 10 mm. Longer service lives are certainly possible for stainless steel, but no guidance is given.

The definition of the Exposure Classes included in Table 4.4 is given in EN 1992-1-1. They are used to categorize the service environment based on the risk of corrosion of the embedded bars by carbonation (XC classes), or chlorides from seawater (XS classes) or other sources such as de-icing salts (XD classes). Likewise, the ERC is used to classify concrete with respect to its resistance against carbonation (class XRC) or chlorides (class XRDS) ingress. At the time of writing this Design Guide, it is understood that definitions for the different ERCs will be given in a CEN Technical specification associated with EN 206[33].

The stainless steel grades are grouped into four different Stainless Steel Resistance Classes (SSRC) based on their Pitting Resistance Equivalent (PRE), which is determined using Equation 4.2

$$\mathrm{PRE} = \mathrm{Cr} + 3.3\mathrm{Mo} + n \times \mathrm{N} \qquad (4.2)$$

where Cr, Mo and N are the amount of chromium, molybdenum and nitrogen as a % of mass, and n is taken as 16 for duplex and 30 for austenitic.

The relationship between the SSRC and the PRE is determined from Table 4.5, which also lists some of the stainless steel grades that may be used for reinforcing bars, of which the lean duplex 1.4362 is the most widely used.

Exposure Class	Exposure Resistance Class ERC	Stainless Steel Resistance Class			
		SSRC1	SSRC2	SSRC3	SSRC4
XC1, XC2	≤ XRC7	0	0	0	0
XC3	≤ XRC4	0	0	0	0
	≤ XRC7	15	0	0	0
XC4	≤ XRC4	15	0	0	0
	≤ XRC7	20	0	0	0
XD1, XS1	≤ XRDS0,5	10	0	0	0
	≤ XRDS1,5	20	10	0	0
	≤ XRDS3	25	15	10	0
	≤ XRDS6	35	25	15	0
	≤ XRDS10	45	35	25	15
XD2, XD3, XS2, XS3	≤ XRDS0,5	15	10	10	0
	≤ XRDS1,5	25	20	15	0
	≤ XRDS3	35	30	20	10
	≤ XRDS6	50	40	30	20
	≤ XRDS10	65	50	40	30

NOTE 1 The tabulated cover values apply for a design service life of 50 years unless the National Annex excludes some classes or gives other values.

NOTE 2 For a design service life of 100 years $c_{\mathrm{min,dur}}$ should be increased by 10 mm for ERC classes unless the National Annex excludes some classes or gives other values.

NOTE 3 In case of combined action of carbonation and chloride induced corrosion, $c_{\mathrm{min,dur}}$ should be increased by 20 mm or a higher stainless steel resistance class should be chosen unless the National Annex gives other values.

NOTE 4 As an alternative to the class system in this table, a performance-oriented service life design may be applied if the input parameters from technical product specifications are available.

Table 4.4 Minimum concrete cover $c_{\mathrm{min,dur}}$ to stainless steel reinforcement (as given in Table Q.3 of the Second Generation of EN 1992-1-1)

Table 4.5 Classification of corrosion resistance of austenitic and duplex stainless steel reinforcing bars dependent on the Pitting Resistance Equivalent PRE (Extracted from Table Q.2 of the Second Generation of EN 1992-1-1)

Stainless Steel Resistance Class	Pitting Resistance Equivalent PRE	Informative examples in EN 10088-1[34]	
		Duplex	**Austenitic**
SSRC2	17 to 22	1.4482	1.4301 1.4307
SSRC3	23 to 30	1.4362	1.4401 1.4404 1.4571
SSRC4	≥ 31	1.4462	1.4529

STRUCTURAL ANALYSIS

The structural analysis of duplex stainless steel composite bridges should be in accordance with Clause 5 of EN 1994-2[3] when the bridge is in the composite stage and Clause 5 of EN 1993-2[12] when the bridge is in the construction stage before the steel and concrete act compositely, except as modified in this Chapter.

EN 1993-1-4[6] requires that the structural analysis of stainless steel structures be limited to an elastic analysis, for which the rules for carbon steel structures are applicable. Therefore, the structural analysis of duplex stainless steel composite bridges should only be based on an elastic analysis. Due to the lower modulus of elasticity of duplex stainless steel, the modular ratio for the composite cross-section will be 5% lower than when the beam is made of carbon steel.

Even though the Second Generation of EN 1993-1-4[7] gives provisions for conducting (material) nonlinear global analysis (referred to as 'plastic zone' analysis), these provisions are not included in this Design Guide because nonlinear analysis is rarely used for bridge structures.

5.1 Structural stability

The provisions in EN 1994-2 and EN 1993-2 related to the use of an elastic analysis are applicable to duplex stainless steel composite bridges.

EN 1994-2, Clause 5.2.1 indicates when second-order effects should be considered in an elastic analysis, while EN 1994-2, Clause 5.2.2 refers to EN 1993-2 for the rules on how these effects should be accounted for. The rules in EN 1993-2, Clause 5.2.2 are based on those in EN 1993-1-1[13] for steel structures. Since the amplification factor in EN 1993-2, Clause 5.2.2 is used to account for the second order effects that originate from a second-order elastic analysis (i.e. it only accounts for second order effects resulting from geometric nonlinearity), this expression is applicable to duplex stainless steel composite bridges.

The Second Generation of EN 1994-2[35] and EN 1993-2[36] both refer to the criteria in the Second Generation of EN 1993-1-1 regarding when second order effects may be omitted in an elastic global analysis. In order to apply these design rules to duplex stainless steel composite bridges, the modifications in the Second Generation of EN 1993-1-4 also have to be considered. Thus, the criteria for when second order effects may be omitted in the elastic global analysis of a duplex stainless steel composite bridge during the construction or fully composite stage are as follows:

If $\alpha_{\mathrm{cr,ns}} = \dfrac{F_{\mathrm{cr,ns}}}{F_{\mathrm{d}}} \geq k_0$	Second order effects due to in-plane or out-of-plane member (non-sway) buckling may be neglected.	(5.1)
If $\alpha_{\mathrm{cr,sw}} = \dfrac{F_{\mathrm{cr,sw}}}{F_{\mathrm{d}}} \geq 10$	Second order effects due to in-plane sway buckling may be neglected and first order analysis may be used for the determination of the in plane sway bending moments.	(5.2)

where:

$\alpha_{\mathrm{cr,ns}}$ is the factor by which the design value of the loading would have to be increased to cause elastic instability in the in-plane or out-of-plane member (non-sway) buckling mode;

$\alpha_{\mathrm{cr,sw}}$ is the factor by which the design value of the loading would have to be increased to cause elastic instability in a global, in-plane (sway) mode;

$F_{\mathrm{cr,ns}}$ is the minimum elastic critical flexural buckling load for either the in-plane or out-of-plane member non-sway buckling mode (torsional buckling, torsional-flexural and lateral torsional buckling are not considered);

$F_{\mathrm{cr,sw}}$ is the minimum elastic critical in-plane flexural buckling load for a global sway buckling mode;

F_{d} is the design value of the loading on the structure.

The Second Generation of EN 1993-1-4 recommends that the value of k_0 in Equation 5.1 is taken as $1/\bar{\lambda}_0{}^2$, where $\bar{\lambda}_0$ is the smallest limiting relative slenderness for the members in the system, according to Table 6.1. In addition to this, during the construction stage and before the concrete deck slab has hardened, second order effects due to lateral torsional buckling of the girders have to be considered in the global analysis (or member verification) unless the limiting slenderness for lateral torsional buckling (see Section 6.2.2) is not exceeded.

5.2 Imperfections

Imperfections are used to account for second-order effects in the structural analysis or when checking the resistance of the structural members. They generally account for geometric imperfections and residual stresses. However, for stainless steel, they also account for the effect of the nonlinear material response.

EN 1994-2 refers to EN 1993-2 for the calculation of the imperfections of transverse frames, while EN 1993-2 refers to Clause 5.3 of EN 1993-1-1 for all the design rules regarding the calculation of imperfections in bridges. These rules are also applicable to duplex stainless steel bridges. However, for the equivalent member imperfections, the magnitude of the imperfection factor should be based on the flexural buckling curves in EN 1993-1-4, by correlating the buckling curve (i.e. a_0, a, b, c, d) with the imperfection factor α used to define the buckling curves in Table 5.3 of EN 1993-1-4.

The Second Generation of EN 1994-2 and EN 1993-2 also refer to the design rules given in the Second Generation of EN 1993-1-1 for the calculation of imperfections in bridges. The rules in EN 1993-1-1 for calculating the equivalent bow imperfections of members are revised, and the Second Generation of EN 1993-1-4 also adopts these rules with slight modifications, as follows:

- For flexural buckling, the equivalent bow imperfection, e_0, is given by:

$$e_0 = \frac{\alpha}{\varepsilon}\beta L \tag{5.3}$$

- For lateral-torsional buckling, the equivalent bow imperfection (in the direction of the minor axis), $e_{0,\mathrm{LT}}$, is given by:

$$e_{0,\mathrm{LT}} = \beta_{\mathrm{LT}}\frac{L}{\varepsilon} \tag{5.4}$$

where:

L is the member length;

α is the imperfection factor, which depends on the type of member and axis of buckling, as given in Table 6.1;

ε is the material parameter given by Equation 3.1;

β is the reference relative bow imperfection, which depends on the type of cross-section verification and is given in Table 5.1;

β_{LT} is the reference relative bow imperfection for lateral-torsional buckling, which depends on the type of cross-section verification and is given in Table 5.2.

Table 5.1 Reference relative bow imperfection β

Buckling about axis	Elastic cross-section verification	Plastic cross-section verification
y-y	1/110	1/75
z-z	1/200	1/68

Table 5.2 Reference relative bow imperfection β_{LT} for lateral-torsional buckling

Cross-section	Condition	Elastic cross-section verification	Plastic cross-section verification
Welded	$h / b \leq 2.0$	1/200	1/150
	$h / b > 2.0$	1/150	1/100

Similar to the modelling approach in EN 1993-1-1, the determination of imperfections may be based on elastic critical buckling modes, in which case the amplitude of the (combined global and local) imperfection should be determined using Equation 5.5. This approach can be used as an alternative to the use of equivalent member imperfections in the analysis, and requires the imperfection shape of the structure to be obtained from an eigen buckling analysis of the entire structure.

$$e_{0,\mathrm{m}} = \alpha_{\eta,\mathrm{m}}(\bar{\lambda}_{\mathrm{m}} - \bar{\lambda}_{0,\mathrm{m}}) \frac{M_{\mathrm{Rk,m}}}{N_{\mathrm{Rk.m}}} \quad (5.5)$$

where:

m is an index denoting the critical cross-section of the bridge structure or of the verified member. Index m indicates that the value or property belongs to the critical cross-section;

$\bar{\lambda}_{\mathrm{m}} = \sqrt{\frac{N_{\mathrm{Rk,m}}}{N_{\mathrm{cr,m}}}}$ is the relative slenderness of the member determined at the critical cross-section m;

$N_{\mathrm{cr,m}} = \alpha_{\mathrm{cr}} N_{\mathrm{Ed,m}}$ is the value of critical axial force in the cross-section m and the critical axial force for the equivalent member;

α_{cr} is the minimum force amplifier for the axial force configuration N_{Ed} in members to reach the elastic critical buckling load of the structure;

$\alpha_{\eta,\mathrm{m}}$ is the imperfection factor α for the relevant buckling curve of the member containing the critical cross-section m, which can be taken from Table 6.1 for different types of duplex stainless steel members;

$\bar{\lambda}_{0,\mathrm{m}}$ is the limiting relative slenderness $\bar{\lambda}_0$ for the relevant buckling curve of the member containing the critical cross-section m, which can be taken from Table 6.1 for different types of duplex stainless steel members;

$M_{\mathrm{Rk,m}}$ is the characteristic value of the moment resistance of the critical cross-section m, e.g. $M_{\mathrm{el,Rk,m}}$ or $M_{\mathrm{pl,Rk,m}}$ as relevant;

$N_{\mathrm{Rk,m}}$ is the characteristic value of resistance to axial force of the critical cross-section m.

Equation 5.5 is included in the Second Generation of EN 1993-1-4. This expression is very similar to the one in the Second Generation of EN 1993-1-1, except that it includes the limiting relative slenderness $\bar{\lambda}_{0,\mathrm{m}}$ as opposed to the 0.2 coefficient used in the expression for carbon steel.

5.3 Classification of cross sections

The rules in Clause 5.5.2 of EN 1994-2 for the classification of composite cross-sections are also applicable when the beam is made of duplex stainless steel. However, the limiting width-to-thickness ratios should be in accordance with those in EN 1993-1-4.

In the Second Generation of EN 1993-1-4, different Class 3 limits are given for sections fabricated with conventional welding, as shown in Table 5.3. This is to account for the larger residual stresses that are associated with this type of welding, compared to hot-rolling or laser welding, and their more detrimental impact on the local buckling resistance of the cross-section. For sections other than those fabricated with conventional welding, the Class 3 limits for internal elements in pure bending and combined bending and compression are also adjusted to account for the change in the effective width expression (see Section 6.1.1), which has the same presentation as the corresponding equation used for carbon steel internal elements. The change in the effective width expression naturally has an impact on the limiting width-to-thickness ratio for which local buckling does not affect the cross-sectional resistance (i.e. the Class 3 limit). The change in the effective width expression for internal elements in sections, other than conventionally welded sections, does not affect elements that are subject to pure compression and therefore the corresponding width-to-thickness limit is not changed.

Internal element				
Class	**Section type**	**Part subject to compression**	**Part subject to bending**	**Part subject to bending and compression**
1	All	$c\,/\,t \le 33\varepsilon$	$c\,/\,t \le 72\varepsilon$	when $\alpha_c > 0.5$: $c/t \le \dfrac{396\varepsilon}{13\,\alpha_c - 1}$ when $\alpha_c \le 0.5$: $c/t \le \dfrac{36\varepsilon}{\alpha_c}$
2	All	$c\,/\,t \le 35\varepsilon$	$c\,/\,t \le 76\varepsilon$	when $\alpha_c > 0.5$: $c/t \le \dfrac{420\varepsilon}{13\,\alpha_c - 1}$ when $\alpha_c \le 0.5$: $c/t \le \dfrac{38\varepsilon}{\alpha_c}$
3	Welded	$c\,/\,t \le 35.4\varepsilon$	$c\,/\,t \le 87\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{35.4\varepsilon}{0.71 + 0.296\psi + 0.006\psi^2}$ when $\psi \le -1$: $c/t \le 43.5\varepsilon(1-\psi)$
	Other	$c\,/\,t \le 37\varepsilon$	$c\,/\,t \le 99\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{37\varepsilon}{0.678 + 0.318\psi + 0.012\psi^2}$ when $\psi \le -1$: $c/t \le 49\varepsilon(1-\psi)$

Table 5.3 Maximum width-to-thickness ratios for compression parts of duplex stainless steel sections included in the Second Generation of EN 1993-1-4

Table 5.3 cont... Maximum width-to-thickness ratios for compression parts of duplex stainless steel sections included in the Second Generation of EN 1993-1-4

Class	Section type	Part subject to compression	Outstand element: Part subject to bending and compression (compression in supported end)	(compression in unsupported end)
1	All	$c\,/\,t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha_c}$	$c/t \leq \frac{9\varepsilon}{\alpha_c\sqrt{\alpha_c}}$
2	All	$c\,/\,t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha_c}$	$c/t \leq \frac{10\varepsilon}{\alpha_c\sqrt{\alpha_c}}$
3	Welded	$c\,/\,t \leq 11.5\varepsilon$	$c/t \leq 17\varepsilon\sqrt{k_\sigma}$	
	Other	$c\,/\,t \leq 14\varepsilon$	$c/t \leq 21.0\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

Clause 5.5.1(4) of EN 1994-2 requires the classification of the steel section to be based on the design values of the strengths of the different materials making up the composite section. For carbon steel composite beams, whether the classification is based on the design or nominal value of the yield strength of the steel section makes no difference to the classification because the partial factor γ_{M0} for carbon steel sections is equal to 1.0, and therefore, the design value of the yield strength is the same as the nominal yield strength. For stainless steel, however, the partial factor γ_{M0} is equal to 1.10, as shown in Table 2.1. Both EN 1993-1-1 and EN 1993-1-4 require that classification is based on the nominal yield strength. This discrepancy within the design approach is because, as was explained in Section 2.1, while in Eurocode 3 (i.e. EN 1993-1-1, EN 1993-2, EN 1993-1-4, etc.) the partial factors are generally applied to the resistance, in Eurocode 4 these are generally applied to the material strength. Eurocode 4 takes this design approach because materials with different partial factors contribute to the resistance of the composite section.

To be consistent with the classification methodology in EN 1993-1-4, it is recommended that for duplex stainless steel composite beams, the design values of the strengths of the different materials of the composite section are only used to calculate the stress distribution within the cross-section (and therefore, the position of the neutral axis), while the class limits are conservatively calculated based on the nominal yield strength of the duplex stainless steel section. For the construction stage, when the duplex stainless steel section is working on its own, the classification should be based on the nominal yield strength.

CONSTRUCTION STAGE

During the construction stage, when the girders and the concrete slab are still not working compositely, the design of the steelwork should be in accordance with EN 1993-2[12], except as modified in this Chapter.

EN 1993-2 refers extensively to EN 1993-1-1[13] and EN 1993-1-5[37]. However, the designer should be aware that EN 1993-1-4[6] modifies some of the referenced provisions in these standards. The provisions modified by EN 1993-1-4 are discussed in the following sections.

The verification of the steelwork in a duplex stainless steel bridge should be carried out using the partial factors in EN 1993-1-4.

6.1 Resistance of cross-sections

6.1.1 Effective width of compression elements

The cross-sectional resistance of duplex stainless steel sections classified as Class 4 should be based on the effective width equations in EN 1993-1-4.

Duplex stainless steel I-section girders are most likely to be fabricated using a conventional welding process, such as submerged arc welding (SAW). However, the use of laser welding techniques is now becoming more common: the reduced heat introduced during fabrication of the structural section results in lower residual stresses, smaller plate distortions and hence a superior buckling performance than with conventional welding.

The Second Generation of EN 1993-1-4[7] provides different effective width expressions depending on the cross-section type. New, and more onerous, effective width expressions are given for compression elements in conventionally welded sections, while the effective width curves in EN 1993-1-4 are retained for any other section type, including laser welded sections. The presentation of the effective width expression for internal elements is also slightly modified to align with the presentation of the corresponding effective width expression in EN 1993 1-5. These stainless steel effective width expressions are reproduced by Equations 6.1 to 6.8, and the curves are compared in Figure 6.1.

The definition of the plate slenderness $\bar{\lambda}_p$ is also revised in the Second Generation of EN 1993-1-4, by replacing the coefficient 28.4 with 27.7, as shown by Equation 6.9. This change in the definition of the plate slenderness is a consequence of the change in the definition of the parameter ε (see Equation 3.1), and accounts for the fact that for stainless steel the modulus of elasticity is equal to 200 000 N/mm^2 (as opposed to 210 000 N/mm^2 for carbon steel).

For conventionally welded sections:

- Internal compression elements

$$\rho = 1 \quad \text{for } \bar{\lambda}_p \leq 0.328 + \sqrt{0.100 - 0.003\psi} \tag{6.1}$$

$$\rho = \frac{0.655\,\bar{\lambda}_p - 0.003(3+\psi)}{\bar{\lambda}_p^2} \leq 1.0 \quad \text{for } \bar{\lambda}_p > 0.328 + \sqrt{0.100 - 0.003\psi} \tag{6.2}$$

- Outstand compression elements

$$\rho = 1 \quad \text{for } \bar{\lambda}_p \leq 0.639 \tag{6.3}$$

$$\rho = \frac{0.655\,\bar{\lambda}_p - 0.01}{\bar{\lambda}_p^2} \leq 1.0 \quad \text{for } \bar{\lambda}_p > 0.639 \tag{6.4}$$

For other sections (including laser welded sections):

- Internal compression elements

$$\rho = 1 \quad \text{for } \bar{\lambda}_p \leq 0.386 + \sqrt{0.089 - 0.02\psi} \tag{6.5}$$

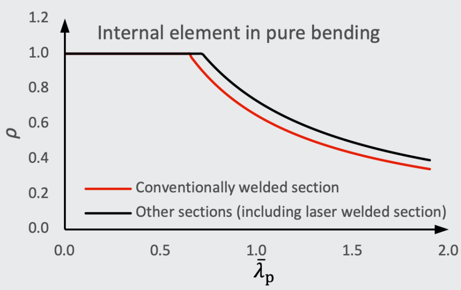
$$\rho = \frac{0.772\,\bar{\lambda}_p - 0.02(3+\psi)}{\bar{\lambda}_p^2} \leq 1.0 \quad \text{for } \bar{\lambda}_p > 0.386 + \sqrt{0.089 - 0.02\psi} \tag{6.6}$$

- Outstand compression elements

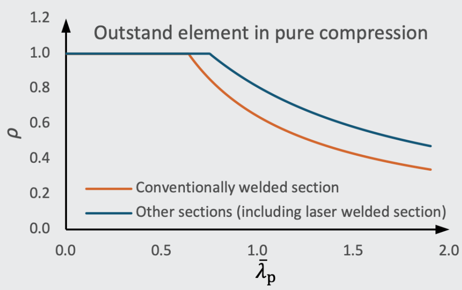
$$\rho = 1 \quad \text{for } \bar{\lambda}_p \leq 0.748 \tag{6.7}$$

$$\rho = \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} \leq 1.0 \quad \text{for } \bar{\lambda}_p > 0.748 \tag{6.8}$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr,p}}} = \frac{\bar{b}/t}{27.7\varepsilon\sqrt{k_\sigma}} \tag{6.9}$$



a) internal element in pure bending



b) outstand element in pure compression

Figure 6.1 Comparison between effective width for laser welded and conventionally welded compression elements

6.1.2 Shear buckling resistance

The shear buckling resistance of duplex stainless steel webs should be calculated in accordance with EN 1993-1-4. Note that EN 1993-1-4 refers to EN 1993-1-5 for most of the calculations needed to compute the shear buckling resistance. However, EN 1993-1-4 gives more stringent slenderness limits beyond which shear buckling will control the shear resistance of the web. It also gives different values for the web shear buckling reduction factor χ_w, which are more onerous at intermediate slenderness to account for the more rounded stress strain behaviour of duplex stainless steel. The equation in EN 1993-1-4 for calculating the parameter c, which is used to account for the contribution of the flanges to the shear buckling resistance of the section is also slightly different from the corresponding equation in EN 1993-1-5.

6.2 Buckling resistance of members

The provisions in EN 1993-2 for calculating member buckling resistance are generally applicable to duplex stainless steel except where modified by EN 1993-1-4, as highlighted in this Design Guide.

EN 1993-2 refers extensively to EN 1993-1-1 for the design of steel members susceptible to buckling instabilities.

6.2.1 Resistance to flexural, torsional or flexural-torsional buckling

The buckling resistance of duplex stainless steel members in compression can be determined using the buckling equations in EN 1993-1-1 for flexural, torsional or torsional-flexural buckling, but using the buckling coefficients in EN 1993-1-4.

In the Second Generation of EN 1993-1-4, the buckling coefficients for duplex stainless steel I-sections, channels, angles and tees are slightly modified to better represent the behaviour shown by more recent studies. The buckling coefficients of duplex stainless steel compressive members that are most likely to be used in multi-girder or ladder deck bridges, for example, as part of the bracing system, are shown in Table 6.1.

Table 6.1 Flexural, torsional and torsional-flexural buckling coefficients included in the Second Generation of EN 1993-1-4 for duplex stainless steel members typically used in bridges

Type of member	Axis of buckling	α	$\bar{\lambda}_0$
Hot rolled and welded I-sections	Major	0.49	0.3
	Minor	0.60	0.3
Cold-formed rectangular and circular hollow sections	Any	0.49	0.3
Channels angles and tee sections	Any	0.76	0.2

6.2.2 Resistance to lateral-torsional buckling

The provisions in Clause 6.3.2.3 of EN 1993-1-1 for calculating the lateral-torsional buckling resistance of rolled sections or equivalent welded sections are not applicable to duplex stainless steel beams.

Also, according to EN 1993-1-4, lateral torsional buckling of duplex stainless steel beams does not need to be considered for $\bar{\lambda}_{\mathrm{LT}} \leq 0.4$, or, if the cross-section is not intended to be loaded to its full cross-sectional bending resistance, when the ratio $M_{\mathrm{Ed}} / M_{\mathrm{cr}} \leq 0.16$.

In the Second Generation of EN 1993-1-1 and EN 1993-1-4, a new lateral-torsional buckling resistance equation is included that better represents the impact of second order effects acting on the beam, although at the expense of slightly more complex calculations. The new lateral-torsional buckling curve is given by Equations 6.10 and 6.11, with the expression for calculating the imperfection factor of duplex stainless steel I-section beams given by Equation 6.12.

$$\chi_{\mathrm{LT}} = \frac{f_{\mathrm{M}}}{\Phi_{\mathrm{LT}} + \sqrt{\Phi_{\mathrm{LT}}^{2} - f_{\mathrm{M}}\bar{\lambda}_{\mathrm{LT}}^{2}}} \quad \text{but } \chi_{\mathrm{LT}} \leq 1.0 \tag{6.10}$$

$$\Phi_{\mathrm{LT}} = 0.5\left[1 + f_{\mathrm{M}}\left(\left(\frac{\bar{\lambda}_{\mathrm{LT}}}{\bar{\lambda}_{\mathrm{z}}}\right)^{2} \alpha_{\mathrm{LT}}\left(\bar{\lambda}_{\mathrm{z}} - 0.2\right) + \bar{\lambda}_{\mathrm{LT}}^{2}\right)\right] \tag{6.11}$$

$$\alpha_{\mathrm{LT}} = 0.23\sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{el,z}}}} \quad \text{but } \alpha_{\mathrm{LT}} \leq 0.76 \tag{6.12}$$

6.2.3 Resistance to combined bending and axial compression

The stability of duplex stainless steel members subject to axial compression and bending should be verified using the interaction equations in Clause 5.5 of EN 1993-1-4. These interaction equations are slightly different from those in EN 1993-1-1.

The simplified interaction equation in Clause 6.3.3 of EN 1993-2 should not be used for duplex stainless steel I-girders because it can lead to unconservative results when compared to the corresponding interaction equation in EN 1993-1-4, especially for members with high nondimensional slenderness $\bar{\lambda}_{\mathrm{y}}$.

In the Second Generation of EN 1993-1-4, the verification of members subject to axial compression and bending is carried out using the interaction equations in EN 1993-1-1, with interaction factors specifically derived for different types of stainless steel sections, as shown in Table 6.2 and Table 6.3.

Table 6.2
Interaction factors k_{yy} and k_{yz} for duplex stainless steel members included in the Second Generation of EN 1993-1-4

$\boldsymbol{k_{yy}}$	$\boldsymbol{k_{yz}}$
Doubly symmetric I-sections	
For $\bar{\lambda}_y < 1.3$: $k_{yy} = C_{my}\left[1 + 2.00\left(\bar{\lambda}_y - 0.30\right)n_y\right]$ For $\bar{\lambda}_y \geq 1.3$: $k_{yy} = C_{my}\left[1 + 2.00 n_y\right]$	$k_{yz} = k_{zz}$ (for k_{zz} see Table 6.3)
Rectangular hollow sections	
For $\bar{\lambda}_y < 1.4$: $k_{yy} = C_{my}\left[1 + 1.50\left(\bar{\lambda}_y - 0.40\right)n_y\right]$ For $\bar{\lambda}_y \geq 1.4$: $k_{yy} = C_{my}\left[1 + 1.50 n_y\right]$	$k_{yz} = k_{zz}$ (for k_{zz} see Table 6.3)
Circular hollow sections	
For $\bar{\lambda}_y < 1.3$: $k_{yy} = C_{my}\left[1 + 2.00\left(\bar{\lambda}_y - 0.38\right)n_y\right]$ For $\bar{\lambda}_y \geq 1.3$ $k_{yy} = C_{my}\left[1 + 1.84 n_y\right]$	$k_{yz} = k_{yy}$

Table 6.3
Interaction factors k_{zy} and k_{zz} for duplex stainless steel members included in the Second Generation of EN 1993-1-4

$\boldsymbol{k_{zy}}$	$\boldsymbol{k_{zz}}$
Doubly symmetric I-sections	
For $\bar{\lambda}_z < 0.8$: $k_{zy} = 1 - \dfrac{0.2\bar{\lambda}_z n_z}{C_{mLT} - 0.2}$ But $k_{zy} \leq 0.6 + \bar{\lambda}_z$ for $\bar{\lambda}_z < 0.8$ For $\bar{\lambda}_z \geq 0.8$: $k_{zy} = 1 - \dfrac{0.16 n_z}{C_{mLT} - 0.2}$	For $\bar{\lambda}_z < 1.2$: $k_{zz} = C_{mz}\left[1 + 2.70\left(\bar{\lambda}_z - 0.50\right)n_z\right]$ For $\bar{\lambda}_z \geq 1.2$: $k_{zz} = C_{mz}\left[1 + 1.89 n_z\right]$
Rectangular hollow sections	
$k_{zy} = k_{yy}$ (for k_{yy} see Table 6.2)	For $\bar{\lambda}_z < 1.4$: $k_{zz} = C_{mz}\left[1 + 1.50\left(\bar{\lambda}_z - 0.40\right)n_z\right]$ For $\bar{\lambda}_z \geq 1.4$: $k_{zz} = C_{mz}\left[1 + 1.50 n_z\right]$
Circular hollow sections	
$k_{zy} = k_{zz} = k_{yy}$ (for k_{yy} see Table 6.2)	
Channels	
$k_{zy} = 1.0$	For $\bar{\lambda}_z < 1.3$: $k_{zy} = C_{mz}\left[1 + 2.50\left(\bar{\lambda}_z - 0.40\right)n_z\right]$ For $\bar{\lambda}_z \geq 1.3$: $k_{zy} = C_{mz}\left[1 + 2.25 n_z\right]$

The Second Generation of EN 1993-2 keeps the simplified interaction equation in EN 1993-2 by reference to a CEN Technical Specification (CEN/TS EN 1993-1-101[38]). Although this simplified equation should not be applied to duplex stainless steel I-girders, an equivalent to this equation can be derived from the interaction equations for duplex stainless steel I-girders, as given by Equation 6.13, which can be used when the girder is restrained against lateral-torsional buckling.

$$\frac{N_{\mathrm{Ed}}}{\frac{\chi_{\mathrm{y}} N_{\mathrm{Rk}}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{yy}} \frac{\left(M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}\right)}{\frac{M_{\mathrm{y,Rk}}}{\gamma_{\mathrm{M1}}}} \leq 1.0 \tag{6.13}$$

where:

is given in Table 6.2 and all the other parameters are as defined in EN 1993-1-1.

ULTIMATE LIMIT STATE DESIGN OF COMPOSITE BEAMS

The design rules in Clause 6 of EN 1994-2[3] for downstand I-section composite beams with a concrete slab are applicable except as modified in this Chapter.

7.1 Cross-section resistance

The provisions in EN 1994-2 for calculating the cross-section resistance of carbon steel composite beams are applicable to duplex stainless steel composite beams except as modified by Sections 7.1.1 and 7.1.2, and provided the partial factors used for the duplex stainless steel section are as given in EN 1993-1-4[6].

7.1.1 Bending resistance

7.1.1.1 Plastic bending resistance

The plastic bending resistance of duplex stainless steel composite beams can be determined based on the same assumptions as those in EN 1994-2 (but using a different β reduction factor), and using the same plastic stress block distribution as for carbon steel composite beams. However, the designer should be aware that due to the more pronounced nonlinear stress-strain behaviour of duplex stainless steel, the actual structural response of a duplex stainless steel composite beam is less accurately represented by the idealized plastic stress block distribution.

Figure 7.1a shows a composite beam subject to a sagging moment in which the position of the plastic neutral axis relative to the total depth of the composite cross-section (x_{pl}/h) is smaller than 0.15. For this scenario, the gradual (and significant) strain hardening exhibited by duplex stainless steel from the onset of yielding leads to a greater bending resistance than that predicted by an idealized plastic stress block distribution, or that may be expected from a composite beam in which the steel section is made of a carbon steel grade with the same yield strength as the stainless steel. Conversely, if the ratio x_{pl}/h is large, the bending resistance may be limited by crushing of the concrete at the top of the slab in a similar way as in a carbon steel composite beam. When this is the case, the plastic bending resistance of the duplex stainless steel composite section should be reduced using the β reduction factor given by Equation 7.1, which was derived in the same way as those for carbon steel (i.e. by comparing the resistance of composite beams using the strain limited method against the plastic resistance assuming a plastic stress block distribution). This reduction factor predicts a more pronounced reduction of the plastic bending resistance (when the ratio x_{pl}/h increases) than the one used for carbon steel beams with similar yield strength (i.e. S460). This more pronounced reduction is due to the more nonlinear behaviour of duplex stainless steel prior to the attainment of the yield strength, which means that the portion of the duplex stainless steel section for which the stresses are below the yield strength is larger than in a carbon steel section. When x_{pl}/h is greater than 0.15, the strain hardening developed by the most strained fibres of the duplex stainless steel section is not sufficient to compensate for the lack of yielding in the regions closer to the neutral axis, as shown in Figure 7.1b. The β reduction factor for duplex stainless steel is compared in Figure 7.2 against the reduction factor in the Second Generation of EN 1994-1-1 for composite beams in which the steel section is made of different carbon steel grades.

$$M_{\mathrm{Rd}} = \beta M_{\mathrm{pl,Rd}} = \left(1.12 - 0.8\, x_{\mathrm{pl}}/h\right) M_{\mathrm{pl,Rd}} \leq M_{\mathrm{pl,Rd}} \tag{7.1}$$

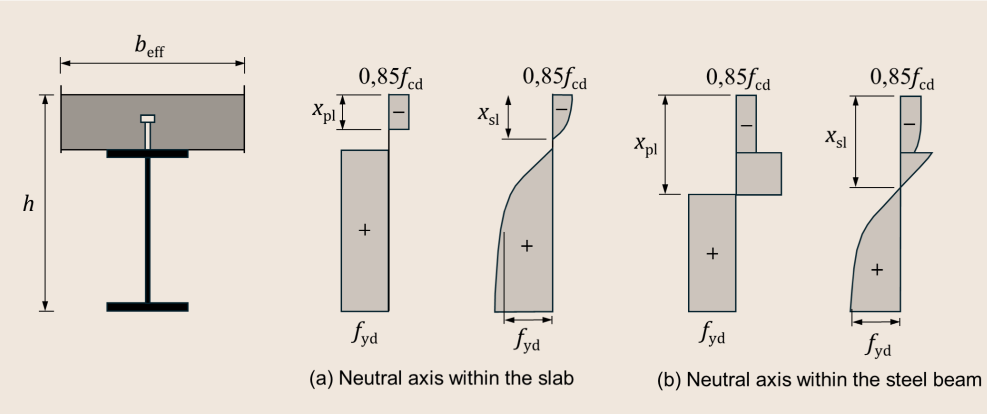


Figure 7.1 Idealized and real stress distribution in duplex stainless steel composite beams for different locations of the plastic neutral axis

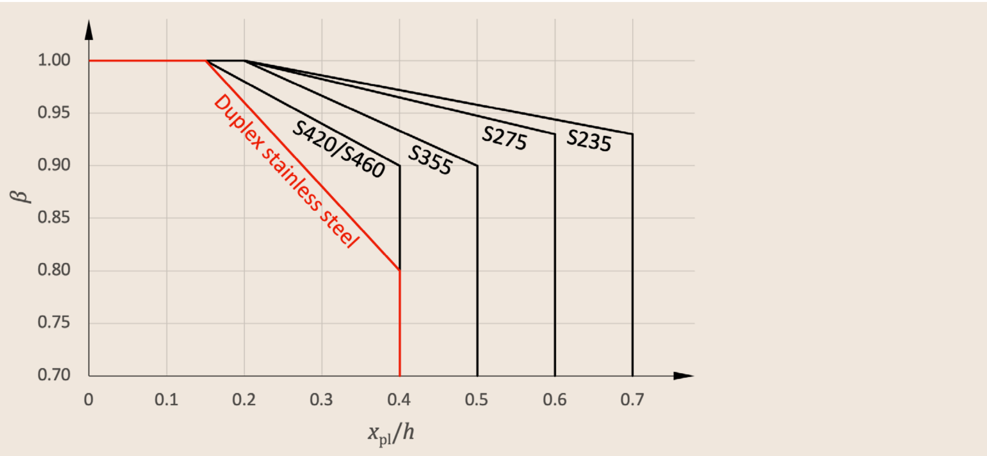


Figure 7.2 Reduction factor β for $M_{pl,Rd}$

7.1.1.2 Non-linear bending resistance

The non-linear bending resistance of duplex stainless steel composite beams can be determined using the same assumptions and material stress-strain relationships given in EN 1994-2 for the concrete and the steel reinforcement. Note that the material model for stainless steel reinforcement in the Second Generation of EN 1992-1-1 is the same as the one for carbon steel reinforcement and is given in Section 3.3.1. For a duplex stainless steel beam, however, the stress-strain behaviour should be represented by the Ramberg-Osgood material model (Section 3.1.7).

7.1.1.3 Elastic bending resistance

In most steel concrete composite bridges, the composite beams are designed based on their elastic bending resistance. The rules in EN 1994-2 are applicable to duplex stainless steel composite beams. However, for duplex stainless steel sections classified as Class 4, the effective section should be determined using the effective width equations in EN 1993-1-4 (see Section 6.1.1).

The lower modulus of elasticity of duplex stainless steel will result in a modular ratio for the composite cross-section that is 5% lower than when the beam is made of carbon steel, and consequently a 5% larger width for the transformed concrete slab.

7.1.2 Resistance to vertical shear

The resistance to vertical shear and to combined bending and vertical shear should be determined in accordance with EN 1994-2. However, the shear buckling resistance of the duplex stainless steel web and the parameter c, which is used to calculate the shear contribution from the flanges of the steel section, should be calculated in accordance with EN 1993-1-4, as explained in Section 6.1.2.

7.2 Resistance to lateral-torsional buckling

The provisions in Clause 6.4 of EN 1994-2 are generally applicable to duplex stainless steel composite beams and frames. However, in those cases in which EN 1994-2 refers to EN 1993-1-1 or EN 1993-2, due consideration should be given to the modifications and limitations made by EN 1993-1-4, as described in Section 6.2.2.

An imperfection factor of 0.76 should be used for the lateral-torsional buckling curve with all the methods covered in Clause 6.4 of EN 1994-2. This is the value of the imperfection factor in EN 1993-1-4 for stainless steel I-girders. For the simplified method in Clause 6.4.3.2 of EN 1994-2, the value of $\bar{\lambda}_{c,0}$ = 0.2 should be used, as recommended in EN 1993-2. For all the other methods, the lateral-torsional buckling curve as given in EN 1993-1-4 should be used.

7.3 Shear connection

Austenitic stainless steel headed studs welded to duplex stainless steel beams can be designed in accordance with the provisions in Clause 6.6 of EN 1994-2. However, the ultimate tensile strength of the stud material does not need to be limited to 500 N/mm^2. The detailing of shear connectors in duplex stainless steel composite beams should satisfy the requirements in EN 1994-2.

If duplex stainless steel headed studs are to be used, their structural performance (load-slip behaviour) should be demonstrated by push-out tests in accordance with Annex B of EN 1994-1-1.

Equations 7.2 and 7.3 give the resistance of headed stud shear connectors in the Second Generation of EN 1994-1-1[39]. These equations are very similar to those in EN 1994, with the equation for calculating the resistance due to stud failure (Equation 7.2) being identical. For the equation accounting for crushing of the concrete around the base of the stud (Equation 7.3), the reduction factor α, which accounts for lower resistance exhibited by short studs, is removed, and instead, it is required that all studs should have $h_{\mathrm{sc}}/d \geq 3.9$, where h_{sc} is the overall nominal height of the stud after welding and d is the diameter of its shank. This more stringent requirement on the stud geometry should have no implication for bridges where studs typically have h_{sc}/d of at least 5. Also, the equation includes the reduction factor K_{cc} which accounts for concrete relaxation under sustained loading. However, the recommended value for this factor is 1.0, which is also to be adopted by the UK National Annex, therefore having no impact on the resistance of the shear connector. All other parameters used in Equations 7.2 and 7.3 are as defined in EN 1994-2.

$$P_{\mathrm{Rd}} = \frac{0.8 f_{\mathrm{u}} \pi d^2/4}{\gamma_{\mathrm{V}}} \tag{7.2}$$

$$P_{\mathrm{Rd}} = \frac{0.29 K_{\mathrm{cc}} d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}}}{\gamma_{\mathrm{V}}} \tag{7.3}$$

FATIGUE

Fatigue can be a significant consideration in the design of composite bridges, especially in road or railway bridges in which the bridge components are exposed to a large number of repetitive loading cycles resulting from the constant road or rail traffic moving over the bridge.

Tests have shown that no difference is needed in the fatigue verification of duplex stainless steel composite bridges compared to that of carbon steel composite bridges. The specific provisions needed to carry out the fatigue verification of duplex stainless steel composite bridges are given in EN 1994-2[3]. This standard also gives provisions for carrying out the fatigue verification of automatically welded headed studs, such as those covered in this Design Guide, and refers to other Eurocode parts for the verification of the other structural components. For duplex stainless steel structural sections, the fatigue verification should be carried out in accordance with EN 1993-2[12], which refers to EN 1993-1-9[40] for the calculation of the fatigue resistance associated with different steelwork details. For the concrete deck slab and the reinforcement, the fatigue verification should be in accordance with EN 1992-2[41] and EN 1992-1-1[21], respectively.

In the Second Generation of EN 1993-1-9[42], most of the existing detail categories remain unchanged. However, additional categories are included, and modifications are made to the description and supplementary requirements associated with some of the detail categories in EN 1993-1-9. Table 8.1 lists the detail categories included in the Second Generation of EN 1993-1-9 which are most likely to require a fatigue verification when designing the type of duplex stainless steel composite bridges covered in this Design Guide.

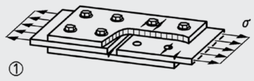
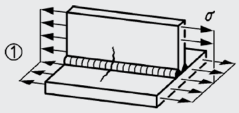
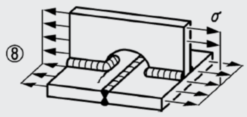
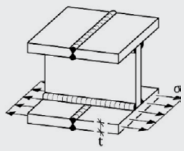
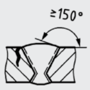
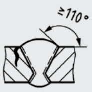
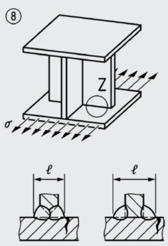
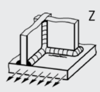
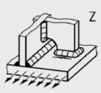
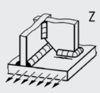
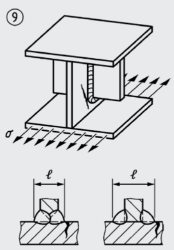
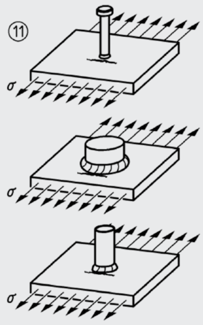
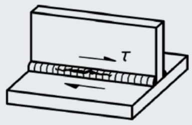
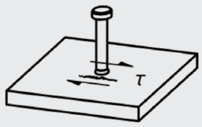
Detail category [a]	Constructional detail	Description	Supplementary Requirements
112 ($m_1 = 5$)		Double covered symmetrical joint subject to normal stress with preloaded high strength bolts	$\Delta\sigma$ should be calculated for all members containing a potential crack initiation using the gross cross-section.
125 ($m_1 = 3$)		Automatic or fully mechanised butt welds, welded from both sides, without stop-starts	None.
112 ($m_1 = 3$)		As forementioned, but with stop-starts	
71 ($m_1 = 3$)		Butt or fillet or intermittent welds with a semi-circular cope hole of a height ≤ 60 mm	$\Delta\sigma$ should be calculated using normal stress in the parent metal assuming the weld is continuous. Longitudinal weld should be all around inside the cope hole.
112 ($m_1 = 3$)		 Flange and web splices in plate girders, welded from both sides, ground flush	No application to full cross-section joint. Splices should be welded before assembly of girder.
90 ($m_1 = 3$)		 As aforementioned, but as welded with weld with flank angle ≥150°	
80 ($m_1 = 3$)	(See Note for size effect at the bottom of the table)	 As aforementioned, but as welded with weld with flank angle ≥110°	
If l ≤ 50 mm 80 If 50 < l ≤ 80 mm 71 ($m_1 = 3$)		 Flanges and webs of built-up sections subject to normal stress with fitted transverse attachment at their surfaces, welded all round, as-welded	$\Delta\sigma$ should be calculated using normal stress in parent metal neglecting the attachment. Ends of welds should be ground to remove undercut if exists.
		 As aforementioned, but with cut holes, welded all around	
		 As aforementioned, but not welded all around	
If l ≤ 50 mm 80 If 50 < l ≤ 80 mm 71 ($m_1 = 3$)		Flanges subject to normal stress with welded shear studs at their surfaces	$\Delta\sigma$ should be calculated using normal stress in parent metal neglecting the small attachment.

Table 8.1 Detail categories included in the Second Generation of EN 1993-1-9 that may be relevant to duplex stainless steel composite bridges

Table 8.1 cont... Detail categories included in the Second Generation of EN 1993-1-9 that may be relevant to duplex stainless steel composite bridges

Detail category [a]	Constructional detail	Description	Supplementary Requirements	
80 (m_1 = 5)		Flanges subject to normal stress with welded shear studs at their surfaces	$\Delta\sigma$ should be calculated using normal stress in parent metal neglecting the small attachment.	
80 (m_1 = 5)		Joints transmitting shear stress, with continuous fillet welds and partial penetration butt welds	Stress concentrations due to macrogeometric effects should be accounted for when calculating $\Delta\tau$.	$\Delta\tau$ should be calculated using shear stress in the weld.
100 (m_1 = 5)		As aforementioned, but with full penetration butt welds		$\Delta\tau$ should be calculated using shear stress in the parent metal.
90 (m_1 = 8), see EN 1994-2		NOTE Effective full penetration butt welds according to EN 1993-1-8 are considered as partial penetration butt welds for fatigue.	$\Delta\tau$ should be calculated using cross section of the stud shear connector.	
Note: Size effect for t > 25 mm: $k_s = (25/t)^{0.2}$, where t is the thinner plate thickness in mm for which the stress range is calculated.				

The provisions for calculating the fatigue resistance (S-N curve) for the steel reinforcement are also revised in the Second Generation of EN 1992-1-1[22] to account for the effect of the bar diameter, as shown in Table 8.2 and Figure 8.1. The revised fatigue curves are also applicable to stainless steel reinforcement meeting the requirements of Section 3.3. In addition to this, it is possible to assume adequate fatigue resistance of the reinforcement in tension if the stress range under the fatigue load combination according to Equation 8.1 with the frequent cyclic load ($\Delta\sigma_{sd}$) meets the following requirements:

- $\Delta\sigma_{sd} \leq 90$ N/mm² for unwelded reinforcing bars with $\phi \leq 12$ mm;
- $\Delta\sigma_{sd} \leq 73$ N/mm² for unwelded reinforcing bars with $\phi > 12$ mm;
- $\Delta\sigma_{sd} \leq 40$ N/mm² for butt and tack welded reinforcing bars with $\phi \leq 12$ mm;
- $\Delta\sigma_{sd} \leq 30$ N/mm² for butt and tack welded reinforcing bars with $\phi > 12$ mm;
- $\Delta\sigma_{sd} \leq 19$ N/mm² for couplers.

$$\sum_i G_{\mathrm{k,i}} + \sum_j \psi_{2,\mathrm{j}} Q_{\mathrm{k,j}} + F_{\mathrm{fat,d}} \qquad (8.1)$$

In Equation 8.1, $G_{k,j}$ represents the characteristic value of the permanent actions, and $Q_{k,j}$ represents the non-cyclic, non-permanent actions, including temperature. $F_{fat,d}$ is the design value of the fatigue action as defined in EN 1991-2 for traffic loading: for road bridges and railway bridges, F_{fat} should be taken as the frequent load of Load Model 1 and Load Model 71, respectively.

Table 8.2 Design parameters for S-N curves for carbon steel and stainless steel reinforcement given in the Second Generation of EN 1992-1-1

Type of reinforcing steel	**Diameter**	**$\Delta\sigma_{Rsk}$ 5% quantile (Test σ_{max} 0.6f_{yk})**			
		$\Delta\sigma_{Rsk}$ (N/mm²)	**N^***	**Stress exponent**	
				k_{f1}	**k_{f2}**
Bars [a]	$\phi \leq 12$ mm	160	2×10^6	5	9
	12 mm < $\phi \leq 16$ mm	140			
	$\phi > 16$ mm	130			
Tack welded bars [b] and welded fabrics	$\phi \leq 12$ mm	100		3	5
	$\phi > 12$ mm	80			
Couplers [c]	-	35	10^7	3	5

[a] Values for bent parts of bars should be obtained using a reduction factor $\zeta_f = 0.35 + 0.026\ \phi_{mand}/\phi$. The reduction factor ζ_f may be omitted for shear reinforcement with 90° stirrups, in which the stirrups have a diameter $\phi \leq 16$ mm and depth $h \geq 600$ mm.

[b] Values for $\Delta\sigma_{Rsk}$ of tack welded bars apply for a distance of 5ϕ at each side of the weld.

[c] Values for couplers apply unless more accurate S-N curves are available and confirmed by testing.

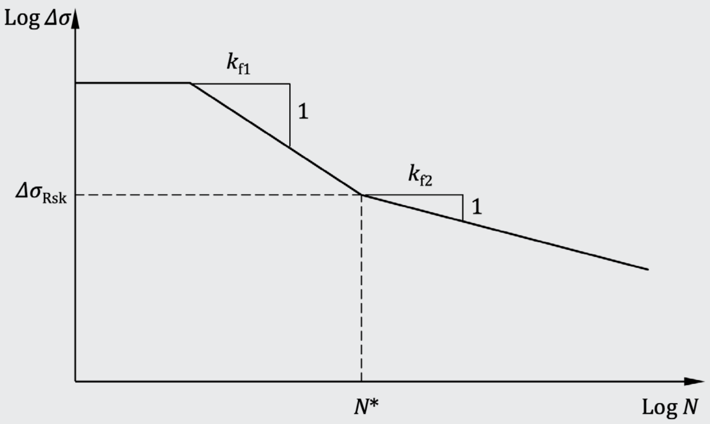


Figure 8.1 S-N curve for carbon steel and stainless steel reinforcement

SERVICEABILITY LIMIT STATES

The serviceability of duplex stainless steel composite bridges should be assessed in accordance with Clause 7 of EN 1994-2, except as modified in this Chapter.

9.1 Deflections

Deflections in duplex stainless steel composite bridges both at the composite stage and the construction stage can be determined in a similar way as for equivalent carbon steel bridges, i.e., by simple hand calculation methods based on elastic theory, or software to perform an elastic analysis of the bridge. However, due to the gradual loss in stiffness exhibited by duplex stainless steel from the onset of loading, deflections of the girders should be expected to be slightly larger than in geometrically equivalent carbon steel girders.

To account for the gradual loss of stiffness, the initial modulus of elasticity of duplex stainless steel should be replaced by a reduced modulus, which depends on the level of stresses within the member. EN 1993-1-4 provides an expression for calculating the reduced modulus, based on the secant modulus calculated for the stresses in the top and bottom flange of the structural section. The initial modulus and the secant modulus are represented in Figure 9.1. EN 1993-1-4 recommends that as a simplification, the reduced modulus should be calculated for the critical cross-section (i.e. the most highly loaded). This method gives an accurate estimation for the deflection when the stresses in the member are below 65% of the yield strength[31], for which the greatest reduction in the elastic modulus would be 4%.

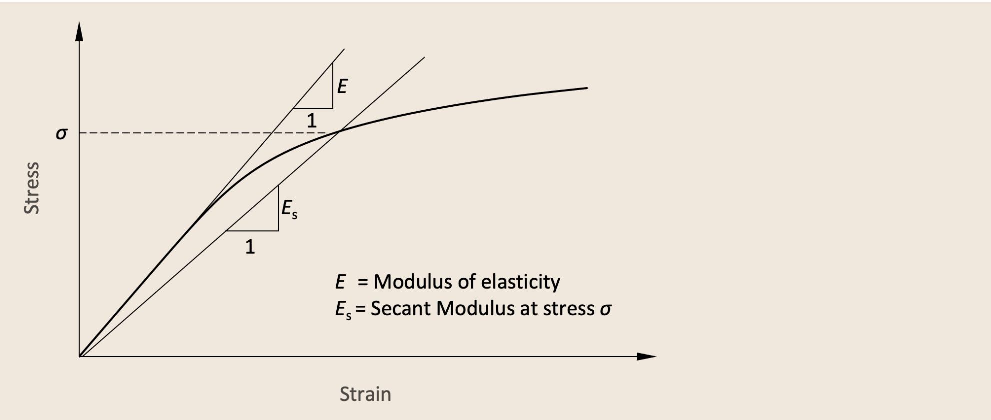


Figure 9.1 Modulus of elasticity and secant modulus

For stresses larger than 65% of the yield strength, the method becomes very conservative because the material starts to lose stiffness more rapidly. Hence, inaccuracies due to the simplifications associated with this method become more accentuated. These simplifications include the assumption that the secant modulus of the entire cross-section is given by the secant moduli of the flanges, which are subjected to the largest stresses within the cross-section, or ignoring the variation of the elastic modulus along the member by using a single reduced modulus. A schematic representation of the variation of the secant modulus of a simply supported duplex stainless steel beam working non-compositely and subject to a uniformly distributed load is shown in Figure 9.2. A better and less conservative estimation of the deflection, while using the reduced modulus in EN 1993-1-4, may be achieved by dividing the girder into several segments along its length and calculating the secant modulus for each segment. The deflection can then be determined considering the variation of the secant modulus over the length of the girder. This approach may be used with simple formulae for the isolated member under idealized support and loading conditions or it may be used as input parameters when carrying out an elastic analysis of the entire bridge. The latter option has the benefit of being able to account for more realistic types of support and loading conditions as well as restraints between different structural components. If an elastic analysis of the entire bridge is used to estimate deflections, a single value for the secant modulus may also be conservatively used. If the stresses in the duplex stainless steel member are not known at the time of performing the analysis, as an initial estimation, these may be modelled with a reduced modulus corresponding to a maximum stress equal to 65% of the yield strength, provided it is later verified that the stresses do not exceed this limit.

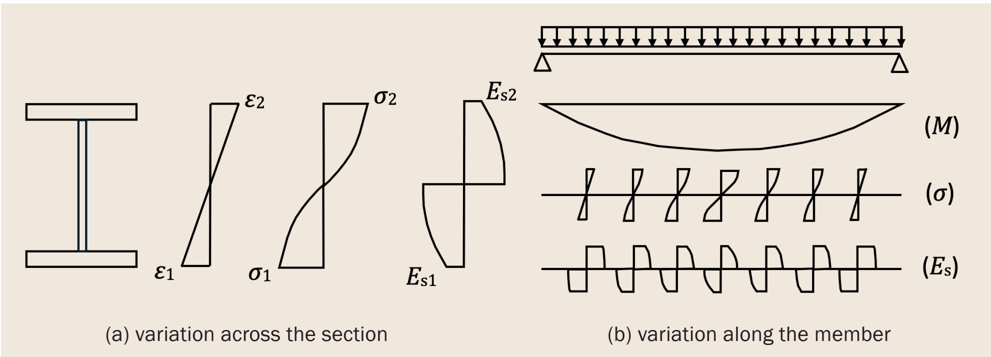


Figure 9.2 Variation of the secant modulus of a duplex stainless steel simply supported beam subject to a uniformly distributed load

The deflection of doubly-symmetric I-girders in their non-composite stage may also be estimated using the analytical equations in the commentary to the DMSSS. These equations were derived by approximating the moment-curvature relationship using an expression similar to the Ramberg-Osgood stress-strain relationship and carrying out a direct integration procedure of this curvature law for specific structural sections (i.e. I-sections and box sections) under simple loading cases, including a simply supported beam subject to a uniformly distributed load or a concentrated load at mid-span. The disadvantage of this method is its limited scope, with the need for derivation of different expressions for other loading conditions or structural shapes.

Deflections may also be determined through a nonlinear analysis using the material model in Section 3.1.7 and the initial modulus of elasticity for the duplex stainless steel structural members. Although this type of analysis will provide the most accurate estimation of deflection this type of analysis is rarely used within the bridge engineering community.

It is worth noting that if local buckling occurs at the serviceability limit state, the effective moment of inertia of the cross-section should be calculated using the initial modulus of elasticity. Since the effective moment of inertia may be considered to be dependent on the level of stress in the cross-section, as given in Annex E of EN 1993-1-5[37], it will also vary along the member length, increasing as the compressive stresses diminish, and potentially reaching the gross moment of inertia if the compressive stresses are sufficiently low. Although it is conservative to calculate the effective moment of inertia for the most highly stressed cross-section, the strategy of dividing the member into several segments and calculating the effective moment of inertia for each segment is also recommended in order to avoid an unduly conservative estimation of the deflection.

9.2 Stresses

EN 1994-2[3] refers to the method in EN 1993-2[12] for the verification of the steel section against web breathing. Although the provisions in EN 1993-2 are generally applicable to duplex stainless steel I-section girders, the calculation of the critical stresses should be based on σ_{E} =181 000 $(t/b_{\mathrm{p}})^2$ as opposed to σ_{E} = 190 000$(t/b_{\mathrm{p}})^2$ which is the expression used for carbon steel webs (where σ_{E} is given in N/mm^2). The reason for this difference is the lower nominal value used for the modulus of elasticity of stainless steel. As a result of this, the web of a duplex stainless steel I-girder will be slightly more susceptible to web breathing than an equivalent carbon steel web. However, most bridge girders will satisfy the web slenderness b_{p}/t limits in Clause 7.4(2) of EN 1993-2 for which web breathing may be neglected.

The lower modulus of elasticity of duplex stainless steel will also have a slight effect on the modular ratio which is used to account for the effect of creep and shrinkage in the concrete when calculating stresses, as the modular ratio for short-term loading will be around 5% lower than that calculated for an equivalent carbon steel composite member.

CONNECTIONS

For the types of duplex stainless steel composite bridges covered in this Design Guide, the most common structural connections are at the splices between sections of the main girders and the connections between the bracing system or cross girders to the main girders. These types of connections can be welded or bolted, with the choice mostly driven by whether the connection is made on site.

As with carbon steel bridges, it is more cost-effective to use bolted connections on site rather than require site welding, possibly even more so for duplex stainless steel bridges due to the need for more tightly controlled welding parameters (see Section 11.6).

This section discusses the design of stainless steel bolted and welded connections. Guidance on the design of the shear connection between the steel section and the concrete deck slab to achieve composite behaviour is given in Section 7.3.

10.1 Bolted connections

For bridges, most bolted connections should be made with preloaded bolts in Category B or C connections, as recommended in EN 1993-2[12], in order to satisfy fatigue and serviceability requirements. These types of connections transfer the shear forces between the plates through friction between the faying surfaces at the serviceability and/or ultimate limit state.

The design and detailing of stainless steel bolted connections and the resistance of the connected parts should be in accordance with EN 1993-1-8[8] and EN 1993-1-1[13], but with the modifications in EN 1993-1-4[6], as highlighted below.

EN 1993-1-4 requires that the corrosion resistance of the bolting assembly should be equivalent to, or better than, the corrosion resistance of the parent material.

EN 1993-1-8 requires that only bolting assemblies of classes 8.8 and 10.9 in accordance with EN 14399[43] should be used as preloaded bolts. Different types of bolting assemblies are covered in EN 14399, including System HR or HV with or without tension indicators, and System HRC, none of which are currently available in stainless steel. In fact, EN 1993-1-4 says that stainless steel bolting assemblies should not be used in preloaded applications unless their acceptability in a particular application can be demonstrated by testing. If stainless steel bolts are used for preloaded applications, the following tests need to be specified to ensure that the required level of preload and slip resistance can be achieved:

- Slip testing in accordance with Annex G of EN 1090-2;
- Suitability test in accordance with EN 14399-2;
- Bolt tightening qualification procedure for the derivation of tightening parameters.

No design or execution rules for stainless steel preloaded bolted connections are given in EN 1993-1-4 or EN 1090-2.

The requirements for stainless steel bolting assemblies in non-preloaded applications are given in Section 3.5.

The design of preloaded stainless steel bolting assemblies is covered in the Second Generation of EN 1993-1-4[7]. Only austenitic and duplex stainless steel bolting assemblies that meet the requirements of Section 3.5 and are in property class 80 or 100 should be used in preloaded applications.

10.1.1 Slip resistance

A European research project[44] demonstrated that austenitic and duplex stainless steel bolting assemblies in property class 80 and 100 in accordance with EN ISO 3506[24],[25] can be satisfactorily preloaded providing the correct bolt grade, tightening method and lubricant are used. In fact, the study showed that the loss of preload exhibited by a stainless steel bolted assembly is comparable to that in a carbon steel bolted assembly, and slip factors measured on grit blasted stainless steel surfaces were consistently at least equivalent to Class B (0.4).

Based on this, and other studies, design rules have been developed for calculating the slip resistance of austenitic and duplex stainless steel bolting assemblies which are included in the Second Generation of EN 1993-1-4 and are reproduced by Equations 10.1 and 10.2.

To use these equations, the suitability of the stainless steel bolting assembly to achieve the intended level of preload should be demonstrated, and the tightening parameters have to be derived by a bolt tightening qualification procedure, which specifies the structural bolting assembly, tightening method, and tightening parameters including appropriate lubrication, preload losses and inspection requirements. This bolting qualification procedure testing may be specified by the National Annex. At the time of preparing this Design Guide, a CEN Technical Specification (CEN/TS 1090-203[45]) is being written which provides guidance on how to derive a bolt tightening qualification procedure for stainless steel preloaded bolting assemblies.

At ultimate limit state $$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,S} \tag{10.1}$$

At serviceability limit state $$F_{s,Rd,ser} = \frac{k_s n \mu}{\gamma_{M3,ser}} F_{p,S} \tag{10.2}$$

where:

k_s is given in EN 1993-1-8, Table 3.6;

n is the number of friction planes;

μ is the slip factor obtained either from specific tests for the friction surface based on test specimens representative of the surfaces used in the structure in accordance with EN 1090-2, or, when relevant, as given in Table 10.1;

$F_{p,S}$ is the nominal preload level of the bolting assembly given by: $= 0.7 f_{yb} A_s$

f_{yb} is the nominal yield strength of the bolt, as given in Table 3.5;

A_s is the tensile stress area of the bolt.

Table 10.1 Slip factors μ for friction surfaces

Surface condition [a]		Class	Slip factor μ
Surface finish	**Rz [b] [μm]**		
Surfaces blasted with clean stainless steel grit media [c]	≥ 55	SSA	0.50
	≥ 45	SSB	0.40
Surfaces blasted with clean stainless steel shot media [c]	≥ 35	SSC	0.20
As rolled surfaces	≥ 25	SSD	0.15

NOTE: The potential loss of preloading force from its initial value is considered in these slip factor values.

[a] The classification of any other surface treatment should be based on test specimens representative of the surfaces used in the structure, following the procedure set out in EN 1090-2[32], Annex G.

[b] Rz is the surface roughness according to EN ISO 21920-2[46].

[c] Care should be taken during grit and shot blasting processes to ensure there is no detrimental effect on the corrosion resistance

It should be noted that although the Second Generation of EN 1993-1-4 specifies a nominal preload level $F_{p,S} = 0.7 f_{yb} A_s$, it is expected that in future revisions of the standard a preload level of up to $0.7 f_{ub} A_s$, where f_{ub} is the ultimate tensile strength of the bolt, will be allowed (which is aligned with the maximum preload allowed in CEN/TS 1090-203). However, designers should be aware that preload levels greater than $0.7 f_{yb} A_s$ may not always be achievable with bolts in sizes equal to or greater than M20. For these bolts, a lower preload level, as given by the bolt tightening qualification procedure testing, should be used in design. A preload level of $0.7 f_{yb} A_s$ is achievable by most bolt sizes

10.1.2 Bearing resistance

The bearing resistance at the bolt hole should be checked for ultimate limit state loads in preloaded bolted connections in Category B and in non-preloaded bolted connections in Category A. For stainless steel bolted connections, the bearing resistance at the bolt hole should be determined using the resistance equation in EN 1993-1-8, but with the ultimate tensile strength of the connected plates reduced, as prescribed in EN 1993-1-4.

The use of bearing-type bolted connections with oversized or slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force, is permitted in EN 1993-1-8. These rules are also applicable to stainless steel with no modification.

In the Second Generation of EN 1993-1-4, the design rules for calculating the bearing resistance at the bolt hole of stainless steel bolted connections are modified. The resistance equation in EN 1993-1-8 (see Equation 10.3 below) is retained, but it is based on the ultimate tensile strength of the plates, as opposed to the reduced tensile strength. Different expressions are also given for calculating the bearing coefficients α_{b} and k_1, which depend on whether the thickness of the connecting plates is greater or less than 4 mm and whether deformations at the serviceability limit state is a design consideration or not. Table 10.2 gives the expressions for calculating α_{b} and k_1 when the thickness of the connecting plates is greater than 4 mm, which is likely to be the case in bolted connections used in bridges. For preloaded connections (Category B), since slip is prevented at the serviceability limit state through friction between the faying surfaces, the bearing resistance at the ultimate limit state only needs to be calculated based on the strength criterion.

The new provisions for calculating the bearing resistance are applicable to non-preloaded and preloaded stainless steel bolting assemblies in accordance with Section 3.5.

The Second Generation of EN 1993-1-8 and EN 1993-1-4 do not allow bolted connections with oversized holes that are designed to slip. This is due to concern that the large clearance at the bolt hole could result in a non-simultaneous transfer of forces between the bolts as some bolts bear against the edge of the bolt hole while others still require a significant amount of slip before transferring force. Stainless steel bolted connections with oversized holes therefore need to be designed as preloaded Category C bolted connections, in which slip is prevented at both the serviceability and ultimate limit state.

$$F_{\mathrm{b,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u}} d t}{\gamma_{\mathrm{M2}}} \tag{10.3}$$

where:

d is the bolt diameter;

d_0 is the hole diameter;

e_1 is the end distance from the centre of a bolt hole to the adjacent end of any part, measured in the direction of load transfer;

e_2 is the edge distance from the centre of a bolt hole to the adjacent edge of any part, measured at right angles to the direction of load transfer;

p_1 is the spacing between centres of bolts in a line in the direction of load transfer;

p_2 is the spacing measured perpendicular to the load transfer direction between adjacent lines of bolts;

t is the thickness of the connected plate;

f_u is the ultimate tensile strength of the connected plate.

	Bearing coefficient α_b	**Bearing coefficient k_1**
Strength criterion	For end bolts: $\alpha_b = \min\left\{2.5, \dfrac{5e_1}{6d_0}\right\}$	For end bolts: $k_1 = \begin{cases} 1.0 & \text{if } \min\left\{\dfrac{e_2}{d_0}, \dfrac{p_2}{2d_0}\right\} > 1.5 \\ 0.8 & \text{if } \min\left\{\dfrac{e_2}{d_0}, \dfrac{p_2}{2d_0}\right\} \leq 1.5 \end{cases}$
	For inner bolts: $\alpha_b = \min\left\{2.5, \dfrac{5p_1}{12d_0}\right\}$	For inner bolts: $k_1 = \begin{cases} 1.0 & \text{if } \left(\dfrac{p_2}{2d_0}\right) > 1.5 \\ 0.8 & \text{if } \left(\dfrac{p_2}{2d_0}\right) \leq 1.5 \end{cases}$
Deformation criterion	For end bolts: $\alpha_b = \min\left\{2.5, \dfrac{5e_1}{4d_0}\right\}$	For edge and inner bolts: $k_1 = 0.5$
	For inner bolts: $\alpha_b = \min\left\{2.5, \dfrac{5p_1}{8d_0}\right\}$	

Table 10.2 α_b and k_1 coefficients for plates with t > 4 mm

10.1.3 Shear resistance

The shear resistance of the bolt should be checked for ultimate limit state loads in preloaded bolted connections in Category B and in non-preloaded bolted connections in Category A.

EN 1993-1-4 adopts the same equation as EN 1993-1-8 for calculating the shear resistance of austenitic and duplex stainless steel bolts and prescribes the following values for the α coefficient:

- $\alpha = 0.6$, if the shear plane passes through the unthreaded portion of the bolt;
- $\alpha = 0.5$, if the shear plane passes through the threaded portion of the bolt.

Although the provisions in EN 1993-1-4 are intended for non-preloaded bolted connections, they are also applicable to preloaded connections.

In the Second Generation of EN 1993-1-4, the shear resistance equation is retained, as shown by Equation 10.4, but less conservative values are specified for the α coefficient. The new α coefficients depend on the alloy family of the bolt and the property class, but are independent of whether the shear plane is located in the threaded or unthreaded portion of the bolt, as shown in Table 10.3, as this is already captured in the shear resistance equation by using the cross-sectional area of the bolt in the shear plane.

The provisions for calculating the shear resistance are applicable to non-preloaded and preloaded stainless steel bolting assemblies in accordance with Section 3.5.

$$F_{\mathrm{v,Rd}} = \frac{\alpha f_{\mathrm{ub}} A}{\gamma_{\mathrm{M2}}} \qquad (10.4)$$

where:

A is the gross cross-section area of the bolt if the shear plane passes through the untreaded portion of the bolt, or the tensile stress area of the bolt if the shear plane passes through the threaded portion of the bolt;

f_{ub} is the ultimate tensile strength of the bolt, as given in Table 3.5.

Table 10.3
Values of α

Property Class	Austenitic	Duplex
50	0.8	-
70	0.7	0.8
80	0.7	0.7
100	0.6	0.6

10.1.4 Tension resistance

Tensile bolted connections are not common in bridges. However, when used, they should be designed as preloaded Category E connections, which may be appropriate for end plate connections attaching deck cantilevers or U-frames.

EN 1993-1-4 does not give special rules for stainless steel bolted connections subject to tensile force, which means that the rules in EN 1993-1-8 should be applied.

In the Second Generation of EN 1993-1-4, the tension resistance of stainless steel bolted connections is given by Equation 10.5, in which the k_2 is taken as 1.0, unless the bolt is countersunk, in which case k_2 = 0.63. As a result of this, the tensile resistance of stainless steel bolts other than countersunk is 10% larger than that of an equivalent carbon steel bolt.

The new provisions for calculating the tension resistance are applicable to non-preloaded and preloaded stainless steel bolting assemblies in accordance with Section 3.5.

$$F_{\mathrm{t,Rd}} = \frac{k_2 f_{\mathrm{ub}} A_s}{\gamma_{\mathrm{M2}}} \quad (10.5)$$

where:

A_{s} is the tensile stress area of the bolt;

f_{ub} is the ultimate tensile strength of the bolt, as given in Table 3.5.

10.1.5 Resistance to combined shear and tension

The interaction between shear and tension in stainless steel bolted connections is accounted for using the same interaction equation in EN 1993-1-8 for carbon steel bolted connections.

In the Second Generation of EN 1993-1-4 the interaction between shear and tension is accounted for by using the interaction equations given by Equations 10.6 and 10.7, depending on the location of the shear plane. The reason for providing two interaction equations is because the tensile resistance of the bolt is calculated assuming tensile rupture always occurs within the threaded portion of the bolt. Therefore, when the shear plane is within the unthreaded portion of the bolt, the tensile resistance of the bolt is corrected by the factor 1.25, which represents the ratio between the gross cross-section area and the tensile stress area of the bolt.

If the threads are not excluded from the shear plane:

$$\left(\frac{F_{\mathrm{v,Ed}}}{F_{\mathrm{v,Rd}}}\right)^{1.7} + \left(\frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}}\right)^{1.7} \leq 1.0 \quad \text{but} \quad \frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}} \leq 1.0 \quad (10.6)$$

If the threads are excluded from the shear plane:

$$\left(\frac{F_{\mathrm{v,Ed}}}{F_{\mathrm{v,Rd}}}\right)^{1.7} + \left(\frac{F_{\mathrm{t,Ed}}}{1.25F_{\mathrm{t,Rd}}}\right)^{1.7} \leq 1.0 \quad \text{but} \quad \frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}} \leq 1.0 \quad (10.7)$$

10.1.6 Resistance of connected elements

The resistance of stainless steel connected elements should be checked considering the possibility of block tearing in accordance with EN 1993-1-8, as well as yielding of the gross-section or fracture of the net section due to tension in accordance with EN 1993-1-4. The provisions for calculating the resistance to net section fracture given in EN 1993-1-4 differ from those give in EN 1993-1-1[13] in the definition of the k_r coefficient – in EN 1993-1-1, k_r is taken as 0.9; in EN 1993-1-4, the expression for calculating k_r was taken from EN 1993-1-3[47].

In the Second Generation of EN 1993-1-1[14], the tensile resistance of the net cross-section (A_{net}) is given by Equation 10.8 with the values of the k coefficient given in Table 10.4. These provisions are also applicable to stainless steel members. For bridges, due to fatigue considerations, the value of k will most likely have to be taken as 0.9.

Table 10.4 Values of k

Type of holes and loading	k
For sections with smooth holes (i.e. holes without notches), for example holes fabricated by drilling or water jet cutting	1.0
For sections with rough holes (i.e. holes with notches), for example holes fabricated by punching or flame cutting	0.9
For structures subjected to fatigue	0.9

10.2 Welded connections

When welding duplex stainless steel, it is essential that suitable welding procedures and compatible consumables are used to ensure the weld can achieve the desired strength and that the corrosion resistance of the weld and surrounding material is maintained.

Several arc welding processes are commonly used in traditional steel bridges, and can also be used when welding duplex stainless steel (see Section 11.6). Complete and partial penetration butt welds and fillet welds are the most common type of welds used in bridges, for which design rules are given in EN 1993-1-8. EN 1993-1-4 only includes a few modifications to these design rules.

The most noticeable modification is that when calculating the design resistance of fillet welds made of stainless steel, in accordance with the ‘directional’ or ‘simplified’ method in EN 1993-1-8, the value of the correlation factor β_w should be taken as 1.0, unless a lower value is justified by tests.

No specific rules for the design of welded connections between stainless steel and carbon steel are given in EN 1993-1-4.

The Second Generation of EN 1993-1-4 gives a range of values for the correlation factor β_w depending on the stainless steel alloy family being joined. However, for welds between duplex stainless steel, the value of β_w = 1.0 is retained. For welded connections between different stainless steel alloy families and between stainless steel and carbon steel, β_w = 1.0 is also be specified.

FABRICATION AND ERECTION

The fabrication and erection of duplex stainless steel composite bridges should be carried out in accordance with EN 1090-2[32]. While most requirements are identical to those for carbon steel structures, there are a few instances in which specific requirements are given for stainless steel structures. This section describes those that are relevant to the type of duplex stainless steel composite bridges that are covered in this Design Guide.

Additional information on fabrication of duplex stainless steel is given in the IMOA publication *Practical Guidelines for the Fabrication of Duplex Stainless Steel*[48] .

11.1 Surface condition

The surface appearance of duplex stainless steel components is an important design criterion which should be clearly specified based on aesthetic and functional requirements. EN 10088-4[9] specifies surface finishes for stainless steel plate products. The designation 1D describes a mill finish on hot rolled material which is free of scale, and has undergone heat treatment and pickling to restore good corrosion resistance. It would be an appropriate finish for bridge girders with no specific aesthetic requirements.

Surface blemishes are not only unsightly, but they are usually unacceptable and prove time consuming and expensive to remove. Whereas surface blemishes will normally be hidden by paint in bridges made of carbon steel, they will be more obvious in duplex stainless steel bridges. Therefore, it is important that the good surface appearance of duplex stainless steel components is preserved throughout fabrication and erection of the bridge.

11.2 Handling and storage

The additional handling and storage requirements in Table 8 of EN 1090-2 are intended to avoid damaging the surface finish and contamination from other metal particles and substances. They are equally applicable to all stainless steel grades, including duplex stainless steels.

In particular, iron contamination is the most likely type of metal contamination, as it can result when carbon steel particles get in contact with the stainless steel surface, potentially leading to rust staining. These iron particles may be transferred onto the stainless steel surface from carbon steel tools and fixtures that are commonly used in most fabrication shops.

Surface damage and iron contamination are of particular importance for stainless steel because unlike carbon steel, they are not coated, and therefore, any damage on the surface is more noticeable. In should be recognized, however, that issues related to iron contamination are likely to affect the surface appearance only, and not the structural performance of the stainless steel plate.

Section 11.8.1 gives information on how to remove surface contamination from duplex stainless steel material.

11.3 Cutting

EN 1090-2 makes no distinction between the cutting of carbon steel and stainless steel. However, it is important to know that stainless steels, including duplex stainless steel, cannot be cut using oxyacetylene (also known as flame cutting) unless a powder fluxing technique is used, and even so, this is strongly discouraged. This is because oxyacetylene cutting can only be used for cutting metals with oxides that have a low melting point, as is the case for carbon steel. Duplex stainless steel has a chromium oxide layer with a high melting point that prevents the oxyacetylene cutting process from being effective.

Other cutting techniques, commonly used to cut carbon steel, including shearing, sawing, abrasive cutting, water jet cutting, and thermal cutting by plasma or laser can all be used for cutting duplex stainless steel. Further information on the use of these cutting techniques for stainless steel is given in DMSSS[31].

11.4 Shaping

11.4.1 Hot forming

Hot forming should be avoided in the fabrication of duplex stainless steel components wherever possible for the same reasons flame straightening should be avoided (see 11.4.2).

11.4.2 Flame straightening

Flame straightening of duplex stainless steel should be avoided wherever possible and only used as the last resort as it may lead to changes in the microstructure which may impair the toughness and/or corrosion resistance of the material.

EN 1090-2 gives additional requirements that should be followed when flame straightening duplex stainless steel material. It is important that the temperature is carefully controlled. EN 1090-2 requires that the maximum temperature should not exceed 500°C - 600°C, and the exposure time from the start of preheating until the end of the cooling-off time should not exceed 8 minutes. This requirement is particularly critical for the super duplex stainless steel grades (i.e. 1.4410, 1.4501 and 1.4507). For the standard and lean duplex stainless steel grades (i.e. 1.4662, 1.4482, 1.4462, 1.4362, 1.4162 and 1.4062), it may be possible to increase the maximum exposure times. However, this should always be carried out under the expert advice from the manufacturer, giving due consideration to the toughness (and corrosion resistance) required by the specific application.

11.5 Connection between dissimilar metals

EN 1090-2 requires that consideration should be given to the risk and implication of galvanic corrosion resulting from dissimilar metals being in contact.

When two dissimilar metals are in direct electrical contact with each other (as may be the case in bolted or welded connections between carbon steel and duplex stainless steel) and are bridged by an electrolyte solution, the less noble metal (i.e. carbon steel) will corrode at higher rates than would be expected for the service environment. There are several factors that will affect the rate at which the less noble metal will corrode, including the conductivity of the electrolyte solution, the type of metals being connected (i.e. their relative galvanic potentials), and the relative surface area of the metals in contact. Because of this, galvanic corrosion will be more severe in joints that are exposed to seawater than to freshwater, which can be expected to have lower conductivity. Although rainwater in clean rural environments can be expected to have low conductivity, fuel combustion products may dissolve in rainwater in more polluted environments, or sea-salts in marine environments, resulting in water of moderately high conductivity. In a similar way, moisture from the air can dissolve contaminants, such as de-icing salts, creating the conditions for galvanic corrosion. Galvanic corrosion is likely to be more severe under immersed conditions than in atmospheric conditions. The severity depends on the time the galvanic contact remains wet, and increases with the presence of crevices in the joint. Information on factors affecting the risk of galvanic corrosion between dissimilar metals can be found in DMSSS[31].

The main strategies to prevent the galvanic corrosion of the metal in contact with stainless steel, assuming that the metal in contact with stainless steel cannot be replaced with stainless steel, consist of:

a. insulating the dissimilar metals from each other (i.e. breaking the metallic path), or
b. preventing the formation of a continuous bridge of electrolyte solution between the two metals (i.e. breaking the electrolytic path).

The method of isolation should be appropriate for the type of exposure and should not permit moisture infiltration into the joint, particularly in immersed or otherwise regularly wet applications.

11.6 Welding

Duplex stainless steel can be welded using any of the arc welding processes commonly used in the fabrication of bridges, as given in Table 8.1. EN 1090-2 refers to EN 1011-1[49] and EN 1011-3[50] for the recommendations that should be followed when welding duplex stainless steel using an arc welding procedure. Useful information on how to weld duplex stainless steel can also be found in the Outokumpu Welding Handbook[51] and IMOA publication[48].

Table 11.1 Welding processes commonly used in the fabrication of bridges

Welding process	Comments
Manual metal arc welding (MMA)	Mostly on site.
Gas-shielded metal arc welding (MIG/MAG or GMAW)	Mainly in the fabrication shop, although its use is growing on site
Self-shielded tubular cored arc welding	Fabrication shop
Submerged arc welding (SAW)	Fabrication shop

Welding duplex stainless steel requires tighter control of the input energy and cooling rate than required for welding carbon steel in order to maintain a balanced ferrite-austenite microstructure and avoid unwanted phases that may form when the material is exposed to high temperatures for a long period of time. However, provided a qualified welding procedure is followed (just as required for carbon steels), site welding of duplex stainless steel should not be significantly more difficult than site welding of carbon steel.

In general terms, lean duplexes are less susceptible to the formation of unwanted phases and so the controls on the acceptable input energy and cooling rate can be slightly relaxed.

If duplex stainless steel welds need to be repaired, this should be done using filler material.

11.6.1 Preparation and execution of welding

The preparation and execution rules in EN 1090-2 can be applied when welding duplex stainless steel. However, the following aspects deserve special consideration as they may be different from welding carbon steel.

Joint preparation

Joint preparation is a very important step when welding duplex stainless steel because it controls the dilution of the material and helps the filler material reach all parts of the weld. Duplex stainless steel may show lower penetration and fluidity than carbon steel. To compensate for the lower penetration it is recommended that, as a general rule, the bevel angle (α) used in V-joints and double V-joints made of carbon steel is increased by 5-10° and the land (c) reduced (see Figure 11.1). Details on joint configurations for duplex stainless steel can be found in the Outokumpu Welding Handbook[52] and the IMOA publication[48].

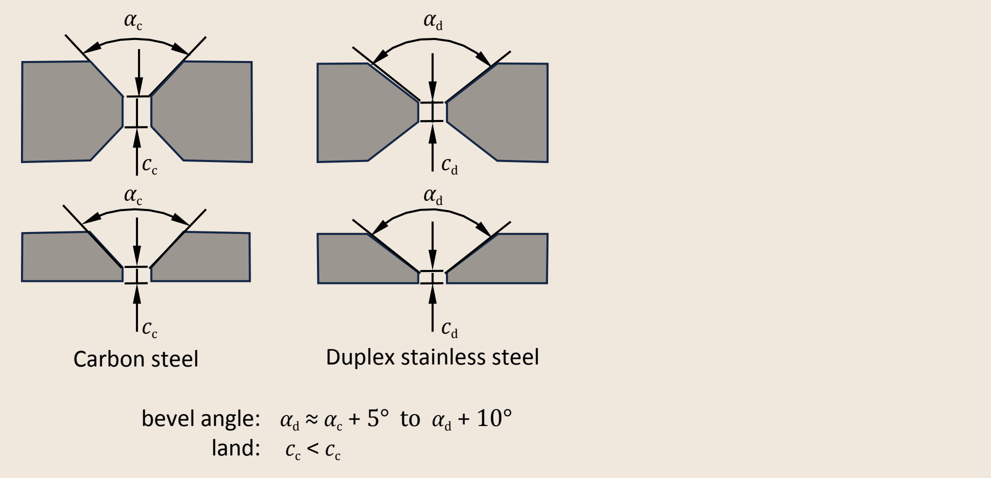


Figure 11.1 Bevel angle and land in V-joint and double V-joints

Preheating

Preheating of duplex stainless steel is generally not recommended unless it is necessary to eliminate moisture. When this is the case, the heat should not exceed 100 °C, and heating should be carried out only after the weld preparation has been cleaned.

Stud welding

While austenitic stainless steel studs can be stud welded to duplex stainless steel beams, the applicability of the stud welding process to duplex stainless steel studs is less well understood, and therefore it is recommended that its structural performance should be demonstrated by testing to confirm adequate toughness and corrosion resistance.

Stud welding carbon steel studs to a duplex stainless steel beam is not recommended because this is likely to result in a mixture of martensite, austenite and ferrite in the weld, with a high density of pores and voids due to the low nitrogen solubility in the weldment. This results in an unreliable weld which may demonstrate brittle behaviour.

11.6.2 Welded connections between dissimilar metals

When welding duplex stainless steel to carbon steel, an over-alloyed filler metal should be used to compensate for dilution with the carbon steel. The filler metal should have sufficient alloying content to produce an austenitic or duplex weld. Therefore, the filler can be of austenitic type (23 12 L or 23 12 2 L) or duplex type (22 9 3 N L). For most situations, it is recommended to use an austenitic filler metal to ensure a weld with sufficient toughness. When joining different duplex grades, the filler metal corresponding to the higher alloyed grade should generally be used. Table 11.2 lists filler metals that are commonly used to weld duplex stainless steel to dissimilar metals.

When welding duplex stainless steel to carbon steel it is important to have as little mixing as possible. To achieve this, the heat input should be reduced, and the arc should be aimed towards the duplex stainless steel part. It is also important that any coating on the carbon steel is removed prior to welding over a region of 20-50 mm from the weld. After welding, the protective coating on the carbon steel should be continued over the weld onto the duplex stainless steel for a minimum of 75 mm in order to avoid the risk of galvanic corrosion (see Section 11.5). Further information on how to weld duplex stainless steel to other metals can be found in the Outokumpu Welding Handbook[52] and the IMOA publication[48].

Table 11.1 Examples of suitable welding consumables for duplex stainless steel grades welded to dissimilar metals

	Designation of nominal composition of welding consumable in accordance with EN ISO 3581, EN ISO 14343 or EN ISO 17633		
	Lean duplex: 1.4482, 1.4162, 1.4362, 1.4062, 1.4662	**Standard duplex: 1.4462**	**Super duplex: 1.4410, 1.4501, 1.4507**
Lean duplex: 1.4482, 1.4162, 1.4362, 1.4062, 1.4662	23 7 N L 22 9 3 N L	22 9 3 N L	22 9 3 N L
Standard duplex: 1.4462	22 9 3 N L	22 9 3 N L	25 9 4 N L
Super duplex: 1.4410, 1.4501, 1.4507	22 9 3 N L	25 9 4 N L	25 9 4 N L
Carbon steel	23 12 L 23 12 2 L	22 9 3 N L 23 12 L 23 12 2 L	22 9 3 N L 23 12 L 23 12 2 L

11.7 Bolting assemblies

As Clause 5.6.3 of EN 1090-2 indicates, bolting assemblies made of carbon steel (including galvanized steel) should not be used to join duplex stainless steel plates. This restriction applies irrespective of whether the bolted connection is preloaded or non-preloaded, and is intended to avoid the risk of galvanic corrosion. Further information on galvanic corrosion is given in Section 11.5 and the DMSSS[31].

Stainless steel bolting assemblies should conform to the requirements in Section 3.5.

11.7.1 Bolted connections between dissimilar metals

The risk of galvanic corrosion in bolted connections between duplex stainless steel and carbon steel in non-immersed environments can be reduced by coating the less noble metal with a waterproof coating. This coating can be an eposy coating or metal primer and paint system, and should meet all the requirements imposed on the coating used in slip-resistant connections made of carbon steel.

Since the application of a coating on the faying surface may reduce the slip factor, and as water is unlikely to penetrate into the faying surfaces, the faying surfaces may be left uncoated following the guidance in Series 1906, paragraphs 6 and 10[52]. The outer surfaces and edges of the joint should, however, be coated to exclude water from the joint as shown in Figure 11.2. Note that since carbon steel is often coated anyway, using this type of solution to protect connections between carbon steel and duplex stainless steel should not add additional complications to the assembly of the connection. However, the slip factor of the faying surfaces should be established by tests.

As an additional means of protection, an insulating bushing may be installed to separate the stainless steel bolt from the less noble metal. This option may be considered because it is not always easy to ensure that the inside of the bolt hole is coated. However, it should be noted that the inside of a bolt hole is somewhat protected from ingress of an electrolyte solution by the bolting components, and even if galvanic corrosion takes place, this is likely to be limited to the edge of the bolt hole.

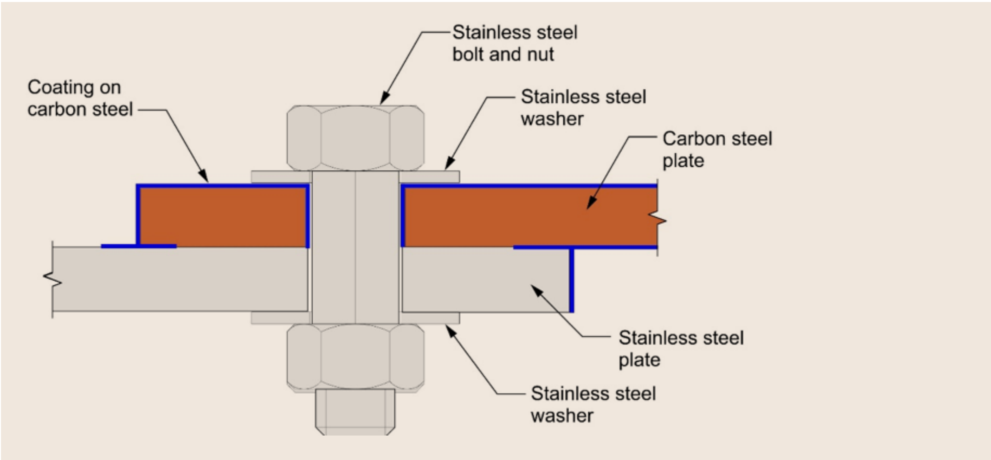


Figure 11.2 Galvanic separation for bolted connections using waterproof coatings in a non-immersed service environment

Note that there may be other types of solutions that are successful at preventing galvanic corrosion in bolted joints, such as the one presented in the DMSSS[31]. However, not all these solutions will be suitable for preloaded bolted connections. For example, the inclusion of compressible materials between the faying surfaces may exacerbate the loss of pretension in the bolt and will affect the slip resistance of the joint.

For structures exposed to immersed environments, it is recommended that any bolted connection between dissimilar metals is located away from the immersed zone.

11.7.2 Preparation of contact surfaces in slip resistant connections

Although EN 1090-2 does not give specific rules for stainless steel preloaded bolted connections, design rules for this type of connection are given in the Second Generation of EN 1993-1-4[7], as discussed in Section 10.1.

Unlike carbon steel, duplex stainless steel is rarely coated, and therefore, the slip-resistance that is achieved in a duplex stainless steel preloaded bolted connection is significantly more dependent on the roughness of the faying surfaces. The Second Generation of EN 1993-1-4 gives slip factors for grit blasted, shot blasted and as rolled faying surfaces, which depend on the roughness parameter Rz given in EN ISO 21920-2[47]. It is not necessary to inspect the surface roughness of every faying surface. However, a blast process should be qualified to produce the required surface roughness and faying surfaces should require intermittent inspection. When blasting the surfaces, clean stainless steel media should be used to avoid contamination.

For any other surface condition, such as those between duplex stainless steel and carbon steel, the slip factor should be determined from tests carried out in accordance with Annex G of EN 1090-2.

The same precautions given in EN 1090-2, Clause 8.4 should be taken prior to the assembly of duplex stainless steel preloaded bolted connections.

11.7.3 Tightening of preloaded bolting assemblies

Stainless steel preloaded bolting assemblies may be tightened using the torque method or the combined method in EN 1090-2, based on the tightening parameters determined from a bolt tightening qualification procedure.

The CEN Technical Specification CEN/TS 1090-203[46] includes a bolt tightening qualification procedure which specifies requirements that can be used for the execution of austenitic and duplex stainless steel preloaded bolting assemblies that are in accordance with Section 3.5.

The bolt tightening qualification procedure in CEN/TS 1090-203 includes a suitability test (now referred to as a fitness-for-purpose test) to ensure that the required preload can be reliably obtained by the chosen tightening method with a sufficient margin of safety against overtightening.

11.7.4 Galling and seizure

As EN 1090-2 indicates, galling may result from local adhesion and rupture of surfaces under load and in relative motion during fastening. In some cases, this results in weld bonding and seizure.

Stainless steel bolting assemblies are more susceptible to galling than carbon steel bolting assemblies, and galling may occur in both non-preloaded and preloaded bolted connections.

EN 1090-2 gives a series of measures that can be taken to minimize the risk of galling. In particular, adequate lubrication of stainless steel bolts and washer faces during installation of the bolting assembly can significantly reduce the risk of galling during tightening. A bolt tightening qualification procedure, such as the one included in CEN/TS 1090-203, should be used to select the right lubricant which minimizes the risk of galling.

11.8 Surface treatment

The following sections focus on the surface treatments covered in EN 1090-2 that require special consideration in the fabrication of duplex stainless steel bridges.

11.8.1 Cleaning

Cleaning the duplex stainless steel surface is an important part of the fabrication process to ensure the material is able to achieve its intended corrosion resistance.

Certain types of defects, impurities and particular contaminants should be removed as they might have a detrimental effect on the corrosion resistance of the duplex stainless steel material.

Weld defects such as weld spatter, slag inclusions, arc strikes, and undercuts, which could negatively impact the local corrosion resistance of the welded connection and its mechanical properties, are commonly removed by grinding, although in some cases repair welding may also be necessary.

Even after a properly executed weld, there will always be a heat tint and oxide scale that produces a thicker oxide layer with inferior protective properties compared with those of the original passive layer. This chromium depleted layer needs to be removed for the weld to achieve adequate corrosion resistance. Purely mechanical methods do not give optimal cleaning results, which may negatively affect the corrosion resistance. It is therefore recommended that after the application of a mechanical cleaning method, such as brushing, a chemical method, such as the application of a pickling paste, is carried out.

Crayon marks, paint, dirt, grease and oil could lead to crevice corrosion in aggressive environments, and therefore should be removed by applying non-chlorinated solvents.

There are different methods for the removal of iron contamination depending on the severity of the resulting rust staining. In all cases, the contaminated surface should be degreased prior to acid treatment, and thoroughly rinsed after the treatment is completed. To avoid staining, it is important that surfaces do not dry between successive steps of acid cleaning, passivation and rinsing. The removal of iron contamination is only necessary for aesthetic reasons.

For mild staining, iron contamination may be removed by applying a saturated oxalic acid solution with a soft cloth or cotton wool and allowed to stand for a few minutes, without rubbing or abrading. This should etch out the iron particles, without leaving scratches or significantly altering the surface texture of the stainless steel. For moderate rust staining, phosphoric acid cleaners can be effective if sufficient time and care is taken, with minimal risk of etching the surface. Alternatively, dilute nitric acid should remove small amounts of embedded iron and will help repassivate the cleaned surface. For more severe rust staining, nitric / hydrofluoric acid pickling preparations should remove more embedded iron than nitric acid alone. Surface etching is likely to occur, and therefore complete restoration to the original finish and surface texture may not be possible.

Further information on the different cleaning methods that can be applied to duplex stainless steel surfaces is given in ASTM A380[53].

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DUPLEX STAINLESS STEEL COMPOSITE BRIDGES

Stainless steels are inherently corrosion resistant. In the presence of oxygen, a tightly adherent protective layer of chromium oxide spontaneously forms on their surface, which means they can perform satisfactorily in a wide range of environments without protective coatings. This intrinsic characteristic of stainless steel is particularly important for bridges, which often need a long service life with minimum maintenance in aggressive environments.

Design lives of 120 years can be anticipated for duplex stainless steel composite bridges with very low maintenance requirements for the stainless steel girders, even in coastal locations or where de-icing salts are used.

This Design Guide gives provisions which extend and modify the application of the Eurocode design rules for carbon steel to cover I-girder composite bridges in which the steelwork is made of duplex stainless steel and the concrete deck slab is reinforced using austenitic, duplex or carbon steel reinforcing bars. The provisions given in this Design Guide are focused on the multi-girder and ladder deck forms of construction, but the principles can be extended to other forms of bridge construction.

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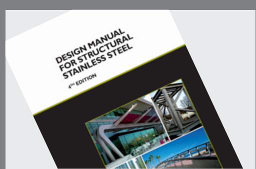

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