Reinforced Concrete Design with Stainless Steel

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Chapter

Reinforced Concrete Design with Stainless Steel

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Abstract

In the design of reinforced concrete structures, the bond property is crucial. This is important for achieving the composite action between the two materials constituents, allowing loads to be efficiently transmitted. The higher strain hardening and ductility capacity of stainless steel over mild steel are one of its major benefits. International design codes, such as Eurocode 2, do not provide a separate design model for concrete structures with stainless reinforcing bars. The background paper to Eurocode 2 highlighted that there is no technical reason of why the Eurocode 2 design model cannot be used in conjunction with other types of reinforcement, provided allowance is made for their properties and behaviour. While this notion is valid when using a mild steel reinforcing bar, it produces erroneous results when a stainless reinforcing bar with a lap splice is used in a reinforced concrete section. Even though there has been a large number of studies on the behaviour of structure with stainless steel in recent years, most of it has been on plain stainless-steel members rather than reinforced concrete or stainless-steel reinforced concrete with lap splice. As a result, the purpose of this chapter is to evaluate and compare the behaviour of stainless and mild steel reinforced concrete with and without lap splices.

Keywords: mild steel, stainless steel, lap, splice length, reinforced concrete, cement-based materials

1. Introduction

The foundations of human civilisations have long been made of cement-based materials. These materials were changed in order to maintain their function in our lives as human activities advanced. Cement's main function is to serve as a hydraulic binder, strengthening the bond between fragmented particles so that they may be used in a variety of applications. The resulted material will differ from the initial materials in terms of its mechanical and physical properties. The exothermic hydration reactions that begin when the water and the binder are mixed are responsible for these changed properties.

In recent years, there has been an increased interest in and use of stainless-steel reinforcing bars in concrete buildings, due to its distinctive properties such as high ductility, long life cycle, excellent corrosion resistance and significant development of strain hardening. This chapter provides a comprehensive overview of prior studies on the design and behaviour of stainless steel as a structural material. The first section provides an overview of stainless steel in general, covering material properties, classifications, chemical composition, and grade categories. The second section delves into the details of using stainless steel as a reinforcing bar in concrete structures, including the obstacles and requirements. It also covers the bond performance of stainless steel in reinforced concrete. In addition, the current applications for stainless steel reinforcement are discussed in this section. The third section focuses on the design requirements of stainless-steel reinforcement outlining the distinctions between mild steel and stainless steel, particularly in terms of constitutive relationship, and discussing current codes of practice for concrete members reinforced with stainless steel.

1.1 Structural applications of stainless steel

In structural engineering, stainless steel is commonly employed for load-bearing applications mainly because of its superior corrosion resistance. It has good formability and recyclability, excellent mechanical characteristics, a long-life cycle, and requires very little maintenance [1]. When compared to mild steel, stainless steel has superior strain hardening capacity and ductility, making it ideal for use as a ductile section that warns of impending collapse. Stainless steel was first used in building in the 1920s for façade and roofing purposes [2, 3]. Stainless steels have recently gained popularity in load-bearing applications that require strength, ductility, durability and stiffness, as well as high resistance.

Stainless steels are manufactured in various forms including tube, plate, sheet, bar, fasteners and fixings, rolled and cold-formed structural sections. Because they are the most readily available and relatively simple to make, cold-formed sections manufactured from steel plates are the most widely utilised materials for structural components [4, 5].

1.1.1 Composition

Stainless steels are a class of corrosion-resistant alloying steels with a maximum carbon content of 1.2% and a minimum chromium concentration of 10.5% [6, 7]. Stainless steel's distinctive properties are determined by the constituent elements of stainless-steel alloy, therefore selecting the right grade for each purpose is essential. In all stainless-steel alloys, chromium is one of the most essential elements, because it offers corrosion resistance by forming a thin chromium oxide film on the material's surface in the presence of oxygen, leading to a passive protective layer [8, 9]. Other alloying elements that have a role in determining the characteristics of stainless steel are also essential. For instance, nitrogen significantly enhances the mechanical properties of the material, molybdenum improves the resistance against uniform and localised corrosion, and nickel improves the formability and ductility of the material [10]. Among the other alloying elements that are commonly present are: Sulphur, carbon, phosphorus, copper and silicon. The European Standard [6] provides comprehensive information on the chemical composition of various stainless-steel grades. The chemical composition of some commonly used stainless steel reinforcement grades is shown in **Table 1**.

1.1.2 Classification

Stainless steels are classified using a number of international categorisation systems. The American Iron and Steel Institute (AISI) specification and the European standards are the most extensively used. More details on these classification systems may be found in the sub-sections below.

1.1.2.1 The American iron and steel institute system

Stainless steels are classified by the AISI into different categories. Ferritic and austenitic stainless steels, for instance, are classed as 400 series alloys (e.g., 403, 409) and 300 series alloys (e.g., 316, 304). The fundamental flaw of this system is that it does not provide specifics on the chemical composition of each grade. **Table 1** lists some of the stainless-steel reinforcement grades available, along with their equivalent

1.1.2.2 European standard

The chemical composition of stainless steel is classified by the European standard [11, 12]. An individual number is assigned to correspond to the nominal alloy composition and then a generic number is assigned to each grade to identify it as part of a group. The numeral in grade 1.4436, for instance, represents:

- 1 represents the steel
- 44 represents the stainless-steel group
- 36 represents the individual material ID

American (AISI)	**European (EN 10088-1)**		**Chemical composition (%)**								
Grade	**Name**	**Grade**	**C Max**	**Si Max**	**Mn Max**	**P Max**	**S Max**	**Cr Max**	**Ni Max**	**Mo Max**	**N Max**
2205	X5CrNi 18-10	1.4301	0.3	1.0	2.0	0.35	0.015	21.0/ 23.0	4.5/ 6.5	2.5/ 3.5	0.10/ 0.22
2304	X5CrNiMo 17-12-2	1.4401	0.3	1.0	2.0	0.35	0.015	22.0/ 24.0	3.5/5.5	0.1/ 0.6	0.05/ 0.20
LDX 2191	X2CrNiMoN 17-13-3	1.4429	0.3	0.4	5.0	—	—	21.5	1.5	0.3	max 0.22
316LN	X3CrNiMoN 22-2-0	1.4162	0.3	1.0	2.0	0.045	0.015	16.5/ 18.5	11.0/ 14.0	2.5/ 3.0	0.12/ 0.22
316	X3CrNiMoN 23-4	1.4362	0.7	1.0	2.0	0.045	0.3	16.5/ 18.5	10.0/ 13.0	2.0/ 2.5	Max 0.11
304	X2CrNiMoN 22-5-3	1.4462	0.7	1.0	2.0	0.045	0.3	17.5/ 19.5	8.0/ 10.5	—	Max 0.11

Table 1.
The chemical composition of various stainless-steel grades [10].

To provide more information about a grade's chemical composition, the grade number is also given the corresponding grade name. For instance, grade 1.4436 is designated as X3CrNiMo 17-13-3, which means:

- X represents a high alloy steel
- 3 represents percentage of carbon content
- CrNiMo is the chemical symbol of the main alloying elements.
- 17-13-3 refers to the nominal percentage of the main alloying elements

1.1.3 Categories of stainless steel

There are five primary types of stainless steel that are categorised based on metallurgical structure, namely, precipitation hardened, duplex, austenitic, martensitic and ferritic grade. In structural applications, such as stainless-steel reinforcement, duplex and austenitic grades are most common. An overview of each category's primary benefits and drawbacks are as follows:

1.1.3.1 Precipitation hardened stainless steel

Precipitation hardened stainless steel offers superior corrosion resistance over martensitic or ferritic stainless steel and is identical to austenitic grades, which include 8% nickel and 18% chromium [13, 14]. Depending on the heat treatment conditions, they have good ductility, toughness and strength. These grades are primarily utilised in aerospace and oil and gas industries, and construction industries for applications like tie-bolts [15].

1.1.3.2 Duplex stainless steel

Due to the mixed microstructure of austenitic and ferritic stainless steels, duplex stainless steels are also referred to as austenitic-ferritic stainless steels. Duplex stainless steels typically include 4–5% nickel and 22–23% chromium [3]. These grades show great ductility and high strength properties as well as excellent corrosion resistance. Duplex stainless steel should be utilised where the materials are exposed to a contaminated or aggressive environment or high strength is required. As a result, they are extensively utilised as shafts, tension bars, valves and pin connections in offshore structures and chemical industries [13, 16].

1.1.3.3 Austenitic stainless steel

Due to their high corrosion resistance and excellent to mechanical properties, austenitic stainless steels are the most frequently utilised grades in structural applications. They generally include at least 8–11% nickel and 17–18% chromium [4, 17]. The austenitic grades provide excellent formability and weldability as well as a wide range of service temperatures [17, 18]. Austenitic stainless steels have been employed in industrial piping, housewares, architectural facades, containers and load-bearing structural members.

1.1.3.4 Martensitic stainless steel

The carbon content of martensitic stainless steel is greater than that of other grades. Martensitic stainless steels have a microstructure identical to carbon steels and ferritic stainless steels. These grades have corrosion resistance that is similar to ferritic grades. In comparison with ferritic, austenitic and duplex grades, they have a lower ductility [18, 19]. Moreover, heat treatment is also required for martensitic stainless steels before and after welding. Despite their lower cost in comparison to other stainless-steel grades, their weldability requirements and poor corrosion resistance limit their applicability to knife blades and valves. They are not utilised in load-bearing applications.

1.1.3.5 Ferritic stainless steel

The chromium concentration in ferritic stainless steel is normally between 11% and 17% [20]. They have an atomic structure that is similar to carbon steel and contain less nickel than austenitic stainless steel. As a result, they have usually limited toughness and ductility and poorer weldability, formability and corrosion resistance than austenitic stainless steels. Because of the limited nickel content, ferritic stainless steels are less costly and have less price volatility. Ferritic stainless steels are better suited to interior applications like handrails and shop fittings, as well as other domestic items like boilers and washing machine parts because of their less corrosion resistance compared to other grades [15].

1.1.4 Material properties

Stainless steels provide excellent corrosion resistance as well as weldability, toughness and strength. Stainless steel material properties differ based on a number of factors, including the direction of rolling and level of cold-working, material thickness and chemical composition. When compared to carbon steels, duplex and austenitic stainless steels offer significant strain hardening properties and higher strength. These grades are known for their high ductility, which may exceed 40%. Stainless steels that are martensitic or ferritic have lower strain hardening and strength. Precipitation-hardened stainless steels, on the other hand, have exceptionally high strength, often exceeding 1500 N/mm^2, although they have limited ductility, depending on the heat treatment condition [20]. The elasticity modulus of various stainless-steel categories is identical to that of carbon steel in general. A value of 200,000 N/mm^2 may be used to define the elasticity modulus for all stainless-steel grades, based on the European standard [6]. **Table 2** presents information on the mechanical properties of some typical grades of stainless steel.

1.1.5 Recycle

A large amount of waste material is generated by the construction industry. The use of more environmentally friendly materials is required in the construction industry to minimise waste. Stainless steels, in this regard, are long-lasting materials with a high residual value of fundamental elements such as molybdenum, chromium and nickel [20]. Around 80% of new stainless steel manufactured in Europe is created from recycled waste stainless steel, according to research [21]. This gives stainless steel more environmental and economic benefits.

Stainless steel type	Grade	Minimum 0.2% proof strength (N/mm^2)	Ultimate tensile strength (N/mm^2)	Modulus of elasticity, E (kN/mm^2)	Minimum elongation after fracture (%)
Ferritic	1.400 (410S)	220	400–600	200	19
	1.4512 (409)	210	380–560	200	21
Duplex	1.4362 (SAF2304)	400	600–850	200	20
	1.4462 (2205)	400	640–840	200	20
Austenitic	1.4301 (304)	210	520–720	200	45
	1.4307 (302L)	200	500–650	200	45
	1.4401 (316)	220	520–670	200	40
	1.4404 (316L)	220	520–670	200	40

Table 2.
Mechanical properties of stainless steels grades [6].

1.1.6 Cost

Stainless steels are inevitably more costly than carbon steels in structural applications [4, 11]. This prevents stainless steel from becoming more widely used. Nevertheless, the initial material cost, on the other hand, does not reflect the overall cost of the construction during its lifetime. Other aspects such as inspection and maintenance costs as well as the immediate cost associated with fire and corrosion protection must be taken into account in order to make an informed decision. When all of these aspects are considered simultaneously, stainless steel is a superior option to carbon steel, particularly for buildings that are subjected to extreme environments.

1.2 Stainless steel reinforcing bar in concrete structures

One of the most commonly used structural solutions in building construction is reinforced concrete. It is popular because it is an efficient, cost-effective and versatile solution with plenty of performance criteria and design guidelines. Owing to their great ductility, significant strain hardening, excellent durability, exceptional corrosion resistance and long-life cycle, stainless steel has recently been used in reinforced concrete structures. Because of the effective usage of readily available constituent materials, reinforced concrete constructions are extensively employed for a variety of applications such as bridges, multi-storey buildings and tunnels.

The stainless steel's constitutive behaviour differs significantly from that of carbon steel as it shows a rounded behaviour from the start, with high ductility and significant stain hardening and without a clearly defined yield point. As shown in **Figure 1**, carbon steel has a more linear relationship in the elastic stage with a moderate degree

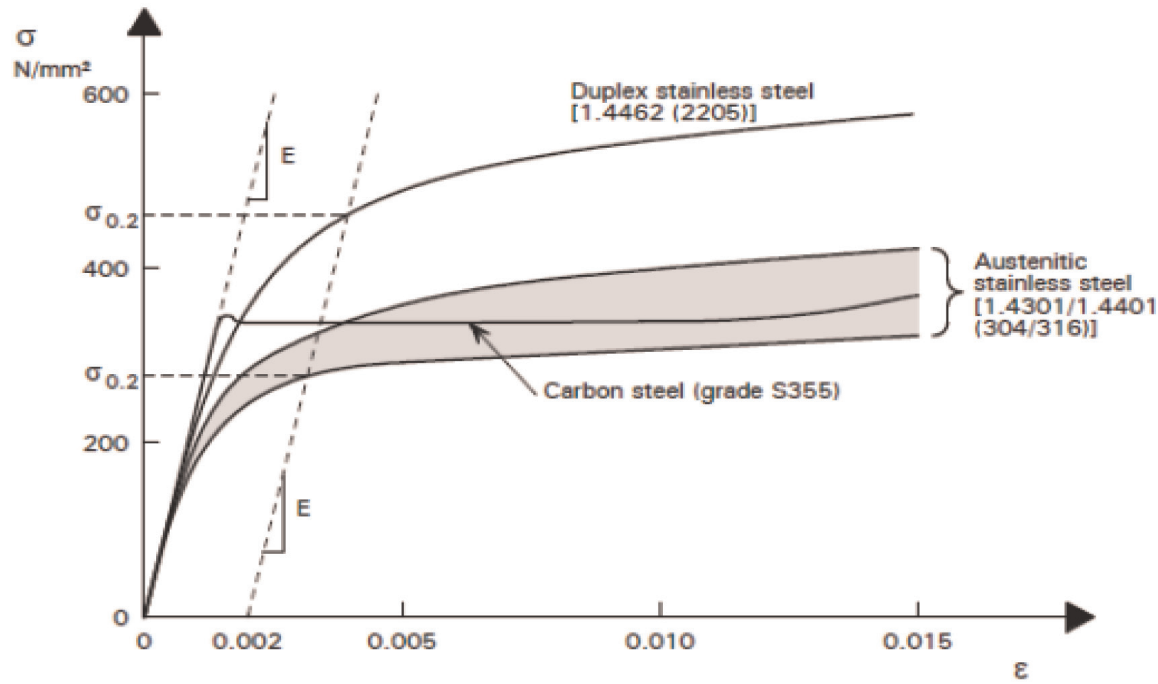


Figure 1.
Stress–strain curves for stainless steel and carbon steel [15].

of strain hardening and a well-defined yield point. When there is no observable yield point, 0.2% proof stress is typically used in the design.

Stainless steels are often represented using the modified-Osgood stainless steel material model which is an improvement of the original version presented in [8].

1.2.1 Life cycle cost

Stainless steel reinforcing bars have a higher initial cost than ordinary carbon steel reinforcing bars, ranging from 3 to 8 times more depending on the grade [13, 22]. Due to the high initial cost, stainless steel reinforcement is sometimes restricted to the outer layer, which is more vulnerable to chloride-ingress. Despite this, stainless steel reinforcement has been proven to save total maintenance costs by up 50% throughout the life of a structure, particularly marine and bridge structures [5, 23]. According to a study conducted by the Arup Research and Development team and oversaw by the UK Highways Agency, stainless steel reinforcement may drastically enhance the lifetime of buildings while simultaneously lowering maintenance costs [24]. Incorporating stainless steel in concrete structures reduces the amount of rehabilitation and maintenance work required during their lifetime. These qualities are critical for infrastructure and highways to minimise rerouting and road closures as well as carbon emissions and delays that come with them. Furthermore, employing corrosion-resistant reinforcing bars like stainless steel saves a lot of money by allowing some durability criteria such as the need for reinforcement coating, design crack width and depth of concrete cover to be relaxed. Incorporating these adjustments into the design of reinforced concrete buildings might save a lot of money, especially on big projects. The Oland Bridge in Sweden, which employed both carbon and stainless-steel reinforcing bar is presented in **Figure 2** as an instance of real-life cycle costs. The data in the figure demonstrated that the cost of a bridge with stainless steel stays unchanged during its lifetime, suggesting no extra expenses, but the cost of a carbon

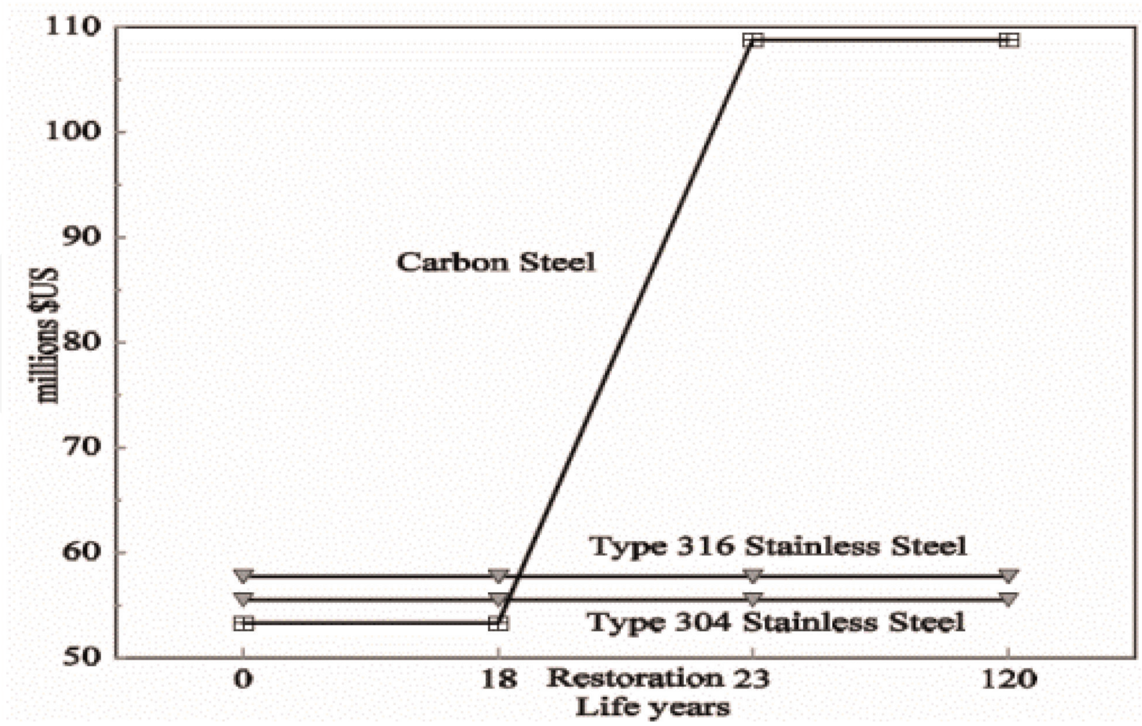


Figure 2.
Life cycle cost analysis for Sweden's Oland bridge [25].

steel-reinforced concrete solution increases drastically after around 20 years. Another research on the Schaffhausen bridge in Switzerland found that stainless steel grade has a 14% lower life cycle cost than carbon steel rebar [25]. This is compelling proof of stainless-steel reinforcement's long-term cost-effectiveness in infrastructure projects.

1.2.2 Durability

Due to the high maintenance costs related to carbonation and steel reinforcement corrosion and of the concrete, there is a growing desire to increase the life cycle cost and durability of reinforced concrete buildings. This is especially true for buildings in harsh environments like those found in industrial or coastal and marine settings. Corrosion is hard to avoid in buildings with carbon steel and exposed to a harsh environment. Changing some design parameters, such as controlling the alkalinity of the concrete mix or the thickness of the concrete cover, is a common approach to increasing the durability of reinforced concrete structures [26]. However, in harsh environments, these precautions may not be sufficient to stop the intolerable level of corrosion from forming. In this respect, the utilisation of stainless-steel reinforcement in exposed structures such as tunnels, bridges and retaining walls can be an effective way to combat corrosion and deterioration. This may even mean that the structure will not require rehabilitation works and expensive inspection in the future. Existing concrete structures can also be rehabilitated and restored using stainless steel reinforcement [16, 27].

1.2.3 Mechanical behaviour

When compared to traditional carbon steel, stainless steel reinforcement has a superior mechanical behaviour. In recent years, there has been a small number of experiments on the mechanical behaviour of stainless-steel reinforcement. When compared to carbon steel reinforcement, austenitic stainless steel reinforcement grades 1.4429 and 1.4311 give superior hardness and strength properties [28]. Reference [29] studied the mechanical and ductility properties of duplex and austenitic stainless steel reinforcement grades 1.4482, 1.4301 and 1.4362 with reference to carbon steel grade B500SD. It was discovered that stainless steels have three times the ductility of carbon steel. However, compared to carbon steels, these stainless steels had a 15% lower elasticity modulus. This is attributed to the fact that stainless steels show nonlinear behaviour from the beginning, making the modulus of elasticity difficult to measure. **Table 3** shows an overview of some of the mechanical parameters of several grades of stainless-steel reinforcement. The stainless steels have great ductility, significant strain hardening and excellent tensile strength. In order to minimise unexpected collapse, these characteristics are very essential in design.

1.2.4 Commonly used stainless steel reinforcement grade

In the open market, stainless steel reinforcement is available in a variety of grades, including duplex grades 1.4362, 1.4162 and 1.4462, as well as austenitic grades 1.4301, 1.4307 and 1.4311. Grade 1.4307 is a standard low-carbon austenitic stainless steel and the most commonly found grade used in construction, whereas grade 1.4311 is a low-carbon austenitic stainless with improved strength and low-temperature toughness due to its higher nitrogen and nickel content. Both of these grades are appropriate for structural applications that require minimal magnetic strength. Due to the relatively high nickel content compared to the austenitic grades, grade 1.4362 is duplex stainless steel that offers excellent corrosion resistance. The lean duplex grades are a new form of duplex stainless steel that has been produced in recent years and has a comparatively low nickel content. Due to the low nickel content, grade 1.4162 offers exceptional corrosion resistance while also having nearly twice the characteristic strength of austenitic steel for almost the same cost.

1.2.5 Selection and classification of stainless-steel reinforcement

The excellent corrosion resistance of stainless-steel reinforcement is undoubtedly one of the most important benefits. As result, categorising stainless steels based on

Product form	Grade	Bar diameter (mm)	Yield strength $\sigma_{0.2}$ (N/mm^2)	Tensile strength (N/mm^2)	Modulus E (kN/mm^2)	Elongation ε_u
Ribbed bars	1.4311	12	480	764	202.6	48.3
	1.4311	16	528	717	199.9	47.9
	1.4162	12	682	874	199.1	32.4
	1.4162	16	646	844	195.2	32.9
	1.4362	16	608	834	171.4	35.1
Plain round bars	1.4307	12	562	796	210.2	39.9
	1.4307	16	537	751	211.1	42.4
	1.4162	12	805	964	308.7	18.8
	1.4162	16	760	860	197.5	22.0

Table 3.
Mechanical properties of stainless-steel reinforcement [23].

their corrosion resistance may make choosing the appropriate grade easier. The Pitting Resistance Equivalent Number (PREN) is the most commonly used categorisation technique for measuring the relative corrosion resistance of a metal. The content of molybdenum, chromium and nickel in an alloy determines the corrosion resistance of the metal, as indicated in Eqs. (1) and (2) [10].

$$\text{PREN} = \%\text{Cr} + 3.3\ (\%\text{Mo}) + 30\ (\%\text{N}) \qquad \text{for duplex stainless steels} \tag{1}$$

$$\text{PREN} = \%\text{Cr} + 3.3\ (\%\text{Mo}) + 16\ (\%\text{N}) \qquad \text{for austenitic stainless steels} \tag{2}$$

When it comes to choosing the appropriate grade of stainless steel for a specific application, classifying stainless steel by its PREN number is useful. However, the PREN ignores the beneficial effects that come from the concrete cover and also does not take into account the chloride threshold of each grade on the passivity of stainless steel [10].

Table 4 shows a categorisation example for stainless steel reinforcement. In this example, the reinforcement is divided into four categories based on their PREN, the surrounding environment and the lifetime of the structure. Class 0 is proposed for structures with a design service life of 10–30 years that are subjected to relative humidity and moderate temperature and are located in the marine environment. For the same conditions, class 1 is recommended with a design service life of 50–100 years. Class 2 is appropriate for structures with a moderate design service life and high chloride penetration, as well as moderate to high relative humidity and temperature. Finally, class 3 is suggested for marine environment structures that required a long design service life in relative humidity and high temperature.

Table 5 gives suggestions for choosing the suitable grade of stainless-steel reinforcement according to the exposure conditions as recommended by the design manual for bridges and roads [31] for infrastructure and highways.

1.2.6 Use of stainless-steel reinforcement

It is widely acknowledged that carbon steel reinforcement in reinforced concrete structures may not be as durable as previously anticipated in all conditions [12, 18]. Corrosion of carbon steel reinforcement in harsh environments like coastal and marine regions can lead to inconvenient rehabilitation, challenging and very

Corrosion resistance class	Steel type	Stainless steel grade	PREN
Class 0	Carbon steel	N/A	N/A
Class 1	Austenitic stainless steel (without molybdenum)	1.4542	17
		1.4301	19
Class 2	Austenitic stainless steel (with molybdenum)	1.4571	25
		1.4436	26
		1.4429	26
		1.4401	25
Class 3	Duplex	1.4462	36

Table 4.
Stainless steel reinforcement categorisation based on their corrosion resistance [10].

Exposures condition	Stainless steel grade
Specific structural requirements for the use of higher strength reinforcement and suitable for all exposures	1.4429 1.4462
Stainless steel reinforcement embedded in concrete with normal exposures to chlorides in soffits, diaphragm walls, edge beams, substructures and joints	1.4301
Direct exposures to chlorides and chloride-bearing waters for example dowel bars, holding down bolts and other components protruding from the concrete.	1.4429
As above but where additional relaxation of design for durability is required for specific reasons on a given structure or component that is where waterproofing integrity cannot be guaranteed over the whole life of the structures.	1.4436

Table 5.
Stainless steel grade selection [30].

expensive work. Stainless steel reinforcement is an effective and long-lasting option in this situation. The Progresso Pier in Mexico depicted in **Figure 3**, was built in the 1940s employing grade 1.4301 austenitic stainless steel and is one of the earliest examples of the usage of stainless-steel reinforcement. The bridge has been in service for more than 70 years without requiring any significant maintenance or major repair work.

Other projects that have utilised stainless steel reinforcement include the Sheikh Zayed Bridge in Abu Dhabi and Stonecutters Bridge in Hong Kong, as depicted in **Figures 4** and **5**, respectively. These two bridges are made with duplex stainless-steel grade 1.4462. The stainless-steel reinforcing bars are well situated and only utilised for the reinforcement outer layer in both bridges in the supposed splash zone. The Broadmeadow Bridge in Ireland and the Highnam bridge expansion project in the UK both employed grade 1.4436 stainless steel. The Queensferry Crossing in Scotland, which opened in 2017, is one of the

Figure 3.
The Progresso Pier in Mexico [32].

Figure 4.
Stonecutters bridge [13].

high-profile and recent applications of stainless-steel reinforcing bars. Stainless steel reinforcement has been employed for restoration and renovation as well as new construction. The pillars and stone arches of the Knucklas rail bridge, for instance, were rehabilitated using austenitic grade 1.4301 stainless steel reinforcement [25].

1.2.7 Fire behaviour

One of the most crucial attributes for creating fire-resistant buildings is the material's capacity to maintain strength and stiffness at high temperatures. Because of the chemical composition, stainless steel has exceptional strength and stiffness retention at extreme temperatures [2]. There have been a lot of studies into stainless steel fire performance [27, 34–36] but very little research studies into the stainless-steel performance at high temperatures [23].

Figures 6 and **7** show a comparison of stiffness and strength retention factors between carbon steel and stainless steel at 0.2% proof stress. In comparison to carbon steel, stainless steel has a distinct advantage in terms of stiffness and strength at high temperatures. In the event of a fire, these distinguishing characteristics are immensely useful and give the structure the resistance needed for a longer duration of time.

1.2.8 Corrosion behaviour

Corrosion of the reinforcement, particularly for members subjected to a harsh environment, is now recognised as one of the most significant issues faced by reinforced concrete structures [37]. Corrosion is a major issue that causes weakness in the bond strength between the surrounding concrete and the reinforcement, as well as the reduction in the nominal reinforcement area which affects the integrity and safety of concrete structures. Corrosion takes place due to carbonation and chloride penetration of concrete. While the former is caused by carbon dioxide in the surrounding

Figure 5.
Sheik Zayed bridge [33].

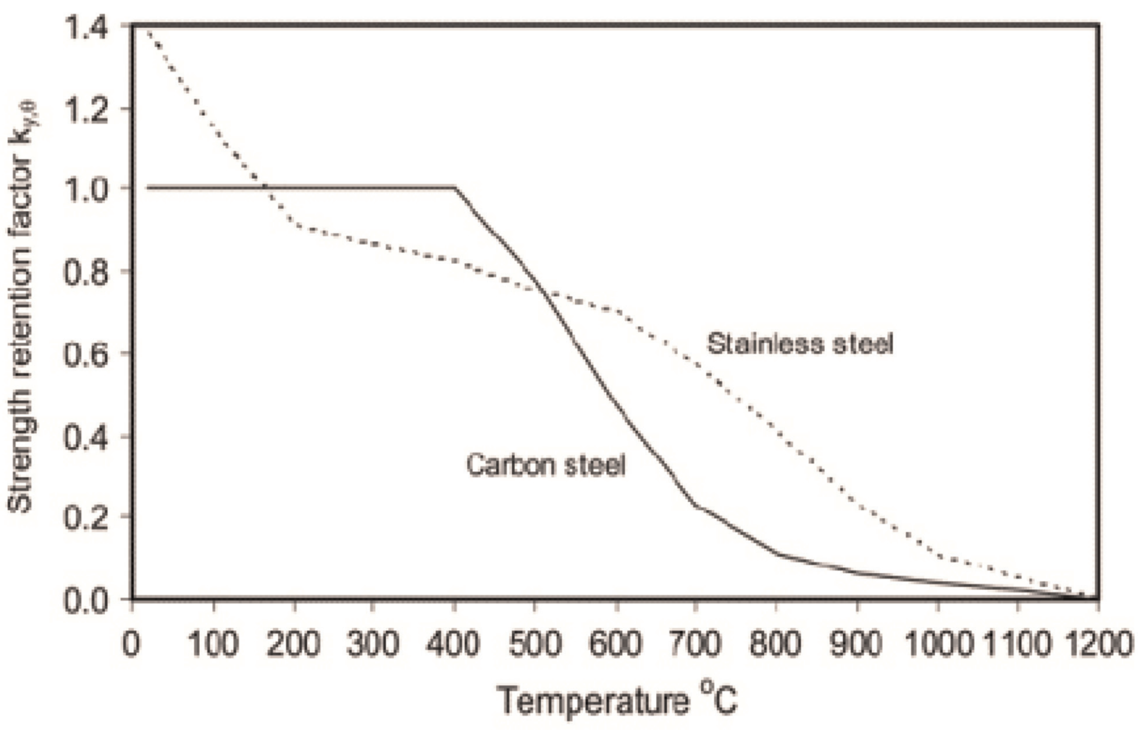


Figure 6.
Comparison of carbon steel and stainless-steel strength retention factor [2].

air attacking the calcium in the concrete. While Ingress of chloride from the marine environment or de-icing salts in frosty weather causes the latter. The corrosion protection of the reinforcement in a typical reinforced concrete design is mostly dependent on the durability of the steel passivation layer and the concrete cover. This passivation layer on typical carbon steels can quickly break down, allowing corrosion to form, particularly in a hash or contaminated environment.

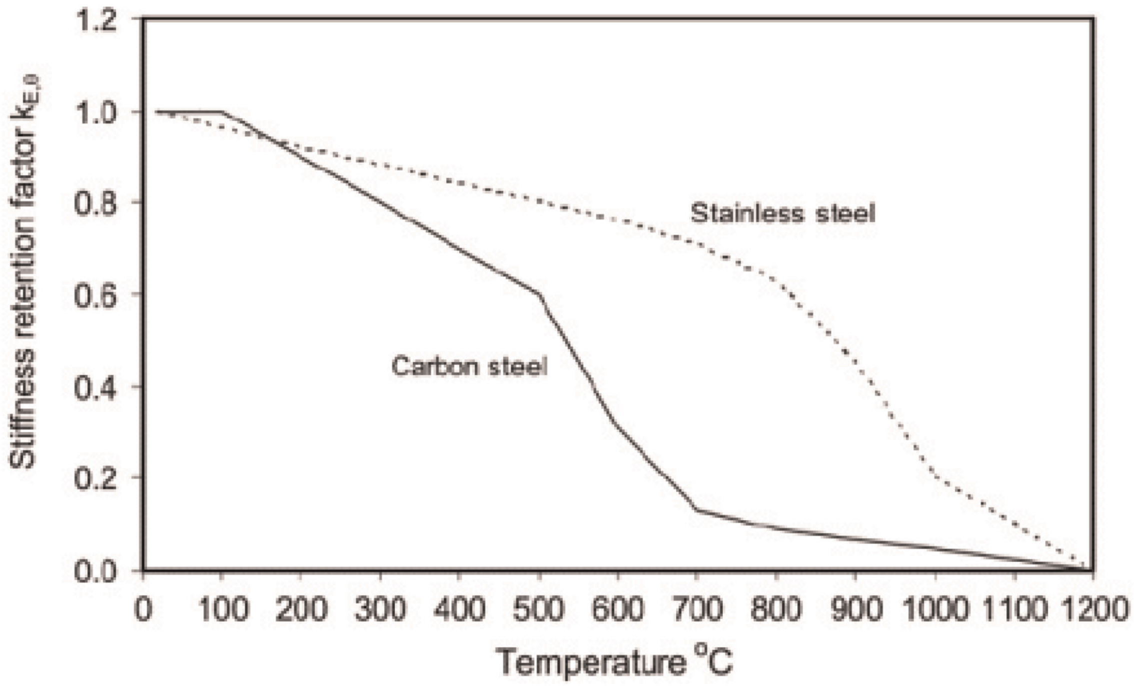


Figure 7.
Comparison of carbon steel and stainless-steel stiffness retention factor [2].

The usual methods of decreasing the potential corrosion risk are to control the concrete alkalinity, use reinforcement coating materials or cement inhibitors or increase the depth of concrete cover [20]. These precautions, however, may not be sufficient to avoid corrosion to undesirable levels. In this situation, stainless steel reinforcement is an excellent alternative for dealing with inherent corrosion issues. Due to its high chromium content (i.e., a minimum of 10.5%), stainless steel offers excellent corrosion resistance even in a harsh environment. In the presence of oxygen, chromium produces a thin self-regenerating chromium oxide coating on the material's surface forming a strong passive protective layer [8, 19, 35].

Austenitic stainless-steel reinforcement is 10 times more corrosion resistant than carbon steel reinforcement [38, 39]. When compared to austenitic reinforcement, duplex reinforcement has equivalent or even greater corrosion resistance [32, 39, 40]. The corrosion performance of several grades of stainless is compared to that of carbon steel in **Figure 8**. The *x*-axis depicts the influence of concrete's PH value, while the y-axis depicts the effect of chloride concentration. Even at very low chloride contents, it is clear that carbon steel has poor corrosion resistance. The PH value of carbon steel is also very sensitive as corrosion occurred. Stainless steel reinforcement, on the other hand, has great corrosion resistance even at low PH values and high chloride content.

1.2.9 Bond behaviour

In the design of reinforced concrete structures, bond is an essential property. It is necessary to ensure that the composite action between the two materials constituent is attained, allowing loads to be transferred efficiently. Insufficient concrete-steel bond can cause excessive rotation or deflection, ineffective anchorage of the reinforcing bar, as well as excessive slippage of the reinforcement leading to serious cracking of the concrete. The many interconnected parameters that determine the development of a bond make it a complicated phenomenon. The surface geometry of the reinforcing bar and the quality of

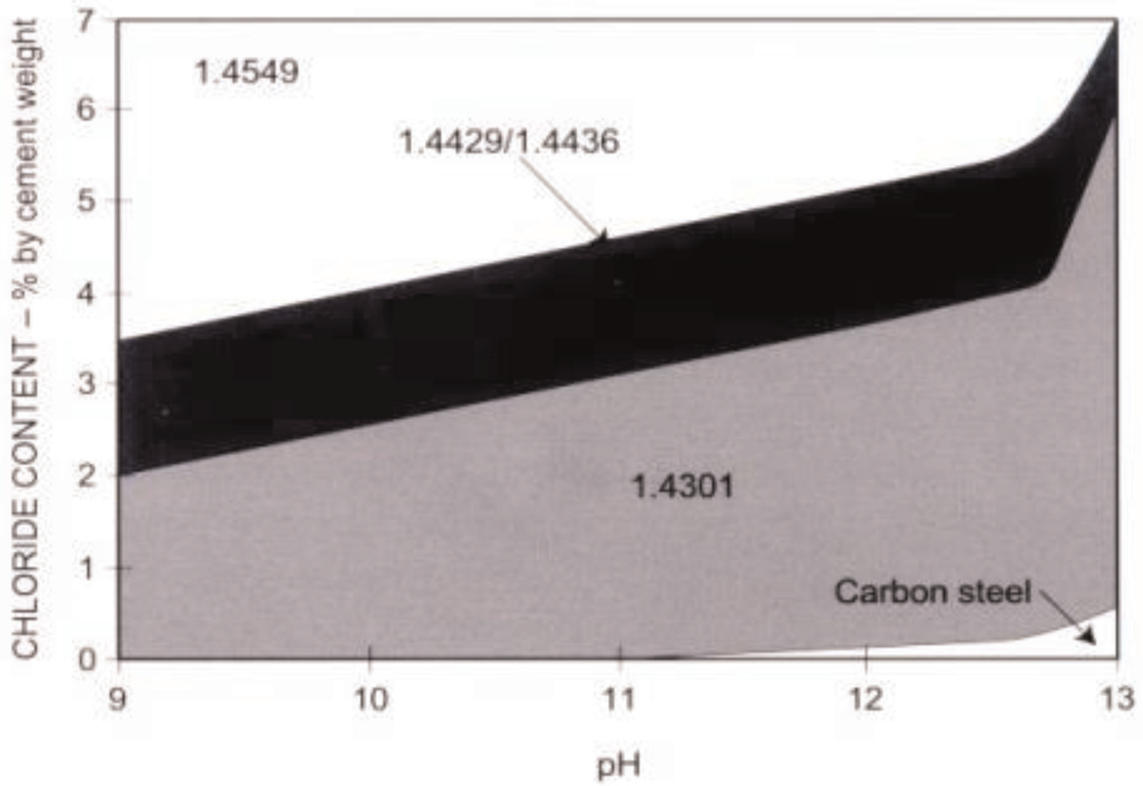


Figure 8.
Corrosion performance of various stainless steel compared to carbon steel [41].

the concrete are the most important parameters. Other key parameters are bar size, the cover distance, the direction of casting with respect to the orientation of the bars, clear space between adjacent bars and the number of reinforcement layers.

There has been little investigation into the bond performance of stainless-steel reinforcement in corrosive environments [20, 42]. And even a few research on the bond behaviour of stainless-steel reinforcement in normal conditions, with some research indicating that the bond developed by some duplex and austenitic stainless-steel bars is relatively low in comparison to similar carbon steel reinforcing bars [34, 43]. As a result, more study into the bonding properties of stainless-steel reinforcing bars in concrete is necessary.

Due to many international design guidelines, such as [44, 45] do not have specific bond design rules for stainless steel-reinforced concrete structures, designers generally use the same design guidelines established for conventional carbon steel reinforcing bars when designing reinforced concrete structures with stainless steel reinforcement. Because stainless steel has been reported to have a reduced bond strength than carbon steel, this is not always a safe approach unless particular test data is presented.

1.3 Current design models for laps

Design models for required lap length are used in design codes for structural members to account for stress formed in bond regions. With each new code reissue, the design models are updated regularly [45], which has been in use since 2004, and includes a lap design model that was originally published in the Ref. [44]. For the next generation of Eurocode 2, the project team for Eurocode 2 has developed a new proposal [46]. The lap design model was derived from [47], which is the background document for ref. [44]. The Eurocode 2 project team provides preliminary calibration factors for change from average values provided in [47] to design values that are still to be verified [46] for lap design models.

Models with and without bond strength definitions are differentiated in code provisions for laps. Earlier design codes specified lap lengths for different concrete classes based on bond strength (for example, [32, 44]). The ACI and Fib Bulletin models, on the other hand, were developed from statistical analysis of experimental data that took into consideration the maximum bar strength in laps without determining bond strength. The bond strength for Model Code 2010 was determined from the [47] design model. Bond strength is an optional parameter that simplifies the design, but it is not required for lap design.

Experimental and partially computational studies with finite element analysis are used to develop models for estimating developable stresses in laps. The processes for finding calibration factors, the influencing parameters, and the composition of equations differ. The major impacting parameters for all the models are compressive strength and lap length, however, their components differ. Some models introduce a summand for shear link contribution; while others consider the shear link contribution by a coefficient. A cover-to-bar diameter ratio is used in all models to account for the influence of concrete cover. The models take into account the influence of shear links differently. Some models comprise the transverse bar spacings_{st} , while others consider the number of shear link bars n_{st}. The correlation is provided by expression (3).

$$\sum A_{st} = n_{st}A_{st} = \left(\frac{l_0}{s_{st}} + 1\right)A_{st} \approx \frac{A_{st}l_0}{s_{st}} \tag{3}$$

The horizontal cracking plane is crossed by the number of shear link leg in side splitting. As a result, if side splitting is assumed, the shear link leg number n_l is included in the lap design models. The shear link cross-section is only evaluated once in face splitting design models because it only provides tensile resistance in one face crack. The number of shear link leg is not considered in this case (see **Figure 9**).

The design expression for laps based on Eurocode, National Annex, and ACI are provided in this section. The origin of the design expressions in the proposal for the next generation of Eurocode 2 from ref. [47] and in ref. [44] are discussed.

1.3.1 Fib bulletin 72

Reference [47] describes the background of bond strength established in ref. [44]. The semi-empirical expression for estimating the average bar stress in tension lap joints was obtained from 800 tests performed in Asia, Europe and the United States.

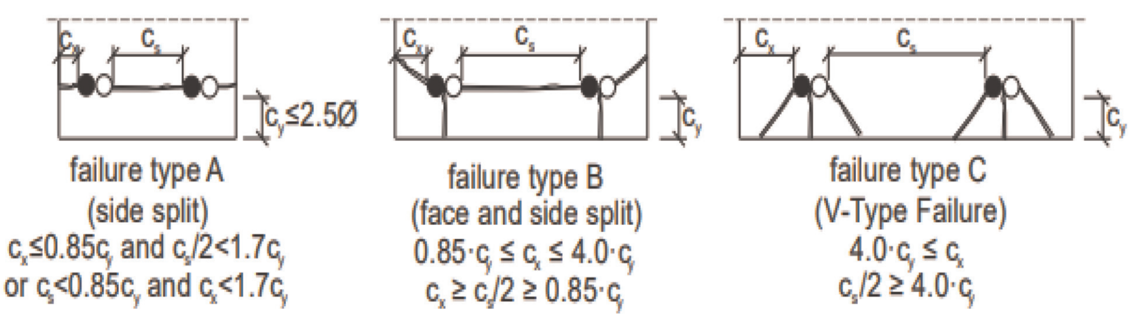


Figure 9.
Splitting failure modes. (Reproduced from [48] based on [49]).

The expression for average lap stress representing the authenticated affecting factors is provided as follows:

$$f_{stm} = 54\left(f_{cm}/25\right)^{0.25}.(25/\varnothing)^{0.2}.(l_b/\varnothing)^{0.55}.\left[(c_{min}/\varnothing)^{0.25}.(c_{max}/c_{min})^{0.1}.\ k_m k_{tr}\right] \le f_y \tag{4}$$

Where;
f_{cm} is the measured concrete cylinder compressive strength
c_{min} is the minimum cover concrete
k_m is the coefficient of efficiency of shear link
c_{max} is the maximum cover concrete
l_b is the lap or anchorage
k_{tr} is the density of shear link

$$k_{tr} = n_l n_{st} A_{st}/(n_b \varnothing l_b) \le 0.05$$

n_b is the number of lapped bars at a section
n_{st} is the number of stirrups in the lap length
f_y is the yield stress
n_l is the number of stirrups legs that crosses the potential splitting failure plane

The bond strength does not increase with a shear link ratio k_{tr} above 0.05. The parameter k_m compensates for the efficacy of the shear link depending on possible failure planes and their position. The shear link is particularly effective when the lap or an anchored bar has a lesser spacing to the next shear link leg across a splitting crack. When the horizontal spacing between bars is greater than 5∅ or 125 mm, the efficiency is decreased by 50%. There is zero effect on bond strength if the shear link does not cross the splitting carack.

Since test results outside of these boundaries hardly exist, Eq. (4) is restricted to the following boundary conditions.

- Good bond condition

$$\frac{l_0}{\varnothing} \ge 10$$

$$\frac{15}{\varnothing} \le 5$$

$$\frac{25}{\varnothing} \le 2$$

$$\frac{c_{max}}{c_{min}} \le 5$$

$$0.5 \le c_{min}/\varnothing \le 3.5$$

The stress developed by bond increases when the transverse pressure is present to

$$f_{stm,tr} = f_{stm} + 6(l_b/\varnothing)p_{tr} < 1.75 f_{st,0} + 0.8(l_b/\varnothing)p_{tr} < 8.0(l_b/\varnothing)f_{cm}^{0.5} \tag{5}$$

Where:

$f_{st,0}$ average stress formed by bond for the base conditions of confinement with

$$f_{st,0} = 54(f_{cm}/25)^{0.25}(l_b/\varnothing)^{0.55}(25/\varnothing)^{0.2} \tag{6}$$

p_{tr} average compression stress perpendicular to the potential splitting failure surface

f_{stm} is the mean estimated stress developed in the bar

1.3.2 Model code 2010

The International Federation for Structural Concrete (known as the Federation Internationale du Beton or Fib) offers advice for the design of prestressed and reinforced concrete in the ref. [44]. In order to establish the required lap length, Model Code 2010 like Eurocode 2, necessitates the calculation of the design bond strength (f_{bd}).

The design bond strength f_{bd} provided in Model Code 2010 was determined by rewriting ACI express for transverse reinforcement index Eq. (34) with a lead coefficient of 41 to enable the formation of the reinforcement design strength $f_{yd} = 435$ MPa. The basic bond strength $f_{bk,0}$ was determined by rearrangement of Eq. (4) with a coefficient of 41. The shear link stress f_{stk} was set to $\frac{500}{1.5} = 435$ MPa and

$$l_{b,0}/\varnothing = \left(f_{yk}/\gamma_c.41\right)^{1.82}.\left(f_{cm}/25\right)^{-0.45}.(25/\varnothing)^{-0.36} = 73.5.\left(f_{cm}/25\right)^{-0.45}.(25/\varnothing)^{-0.36} \tag{7}$$

$$f_{bk,0} = f_{yd}.\varnothing/4.l_{b,0} = 1.5.\left(f_{cm}/25\right)^{0.45}.(25/\varnothing)^{0.36} \tag{8}$$

The coefficient, as well as the indices, were approximated to more practical values, and the confining reinforcement and cover values equivalent to the least detailing requirements were established. The coefficient 1.5 was modified to $\eta_1 = 1.6$ for the calculation of the basic bond strength without stirrups. The coefficient η_1 was modified to 1.75 and the values of basic bond strength were increased by 10% to account for the increase in bond strength if minimal confining reinforcement is provided [44]. The design bond strength is calculated using Eq. (9).

$$f_{bd} = (\alpha_2 + \alpha_3) f_{bd,0} - 2P_{tr}/\gamma_{cb} < 2f_{bd,0} - 0.4P_{tr}/\gamma_{cb} < 1.5\sqrt{f_{ck}}/\gamma_{cb} \tag{9}$$

Where:

$f_{bd,0}$ is the basic bond strength, which is derived using Eq. (10) and is a function of the characteristic compressive strength (f_c)

$$f_{bd,0} = \eta_1\eta_2\eta_3\eta_4\left(\frac{f_c}{25}\right)^{0.5} \tag{10}$$

η_1 is a coefficient taken 1.75 for ribbed bars (including stainless and galvanised reinforcement)

η_2 represents the casting position of the bar during concreting: $\eta_2 = 1.0$ for good bond condition.

γ_{cb} is the partial safety factor for bond $\gamma_{cb} = 1.5$

The influence of passive confinement from transverse reinforcement and concrete cover is represented as α_2. and α_3. Where; $\alpha_2 = \left(\frac{C_{\min}}{\varnothing}\right)^{0.5} . \left(\frac{C_{\max}}{C_{\min}}\right)^{0.15}$ and $\alpha_3 = k_{\mathrm{d}}.(k_{\mathrm{tr}} - \alpha_{\mathrm{t}}/50) \geq 0.0, k_{\mathrm{tr}} \leq 0.05$

Where:

$C_{\min}$ is the minimum cover concrete: $c_{\min} = \min\left\{c_{\mathrm{x}}; c_{\mathrm{y}}; \frac{c_{\mathrm{s}}}{2}\right\}$

$C_{\max}$ is the maximum cover concrete: $c_{\max} = \min\left\{c_{\mathrm{x}}; c_{\mathrm{y}}; \frac{c_{\mathrm{s}}}{2}\right\}$

α_{t} is the coefficient for the bar diameter;

$\alpha_{\mathrm{t}} = 1.0$ for $\varnothing = 25$ mm

$\alpha_{\mathrm{t}} = 0.5$ for $\varnothing = 50$ mm

$k_{\mathrm{tr}} = n_{\mathrm{t}}.\frac{A_{\mathrm{st}}}{(n_{\mathrm{b}} \varnothing s_{\mathrm{t}})}$ is the density of transverse reinforcement, relative to the lapped bars;

n_{l} is the number of legs of confining reinforcement crossing a potential splitting failure surface at a section;

A_{st} is the cross-sectional area of one leg of a confining bar (mm^2);

s_{t} is the longitudinal spacing of confining reinforcement (mm);

$\varnothing$ is the bar diameter;

n_{b} is the number of pairs of lapped bars in the potential splitting failure section;

n_3 represent the bar diameter: $n_3 = 1.0$ mm for $\varnothing \leq 25$

n_4 represents the characteristics strength of steel reinforcement that is being lapped (see **Table 6**).

k_d is an effective factor dependent on the reinforcement details and accounts for the stress developed and the nonlinear behaviour between the lap length in the bar (see **Figure 10**).

In case the concrete class is grade C60 or below and the anchored bar's diameter is less than 20 mm, the stirrup added for other reasons can be deemed to be adequate to meet the least criteria for confined reinforcement without additional explanation [44]. Therefore, the minimum stirrup has to be located with:

n_4	Characteristic strength of steel reinforcement f_{yk} (MPa)
1.2	400
1.0	500
0.85	600
0.75	700
0.68	800

Table 6.
Coefficient n_4 [50].

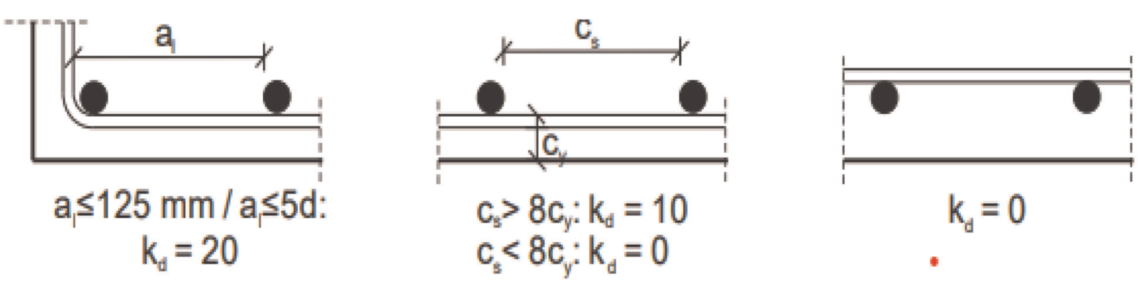


Figure 10.
Coefficient k_{d} for efficiency of stirrups (Source: [44]).

$$\sum A_{st} = n_g n_t A_{st} \geq \frac{\alpha_t A_{s,cal}}{A_{s,prov}}.n_b A_s \tag{11}$$

Where;
$A_{s,cal}$ calculated area of reinforcement
n_t is the number of stirrups crossing a potential splitting failure surface at a section
$A_{s,prov}$ area of reinforcement provided
n_g number of items of confining reinforcement within the bond length
The design anchorage length l_b can be calculated from Eq. (12):

$$l_b = \frac{\varnothing \sigma_{sd}}{4 f_{bd}} \geq l_{b,\,min} \tag{12}$$

$l_{b,\,min}$ is the minimum accepted design value for lap length, calculated as:

$$l_{b,\,min} > max\left\{\frac{0.3\varnothing f_{yd}}{4 f_{bd}}; 10\varnothing,\, 100\ mm\right\} \tag{13}$$

Where σ_{sd} is the stress in the bar that will be anchored by bond across the length of the lap, and is calculated as:

$$\sigma_{sd} = \alpha_1.f_{yd} \tag{14}$$

$$\alpha_1 = \frac{A_{s,cal}}{A_{s,ef}}$$

Where $A_{s,ef}$ and $A_{s,cal}$ are the actual area of reinforcement and the required area of reinforcement determined in design, f_{yd} is the design yield strength of the reinforcement.

The design lap length is calculated as follows:

$$l_0 = \alpha_4 \frac{\varnothing f_{yd}}{4 f_{bd}} \geq l_{0,\,min} \tag{15}$$

$l_{0,\,min}$ is the minimum accepted design value for lap length, calculated as:

$$l_{0,\,min} > max\left\{\frac{0.7\varnothing f_{yd}}{4 f_{bd}}; 15\varnothing,\, 200\ \text{mm}\right\} \tag{16}$$

The coefficient α_4 = 0.7 may be used if no more than 34% of the bars are lapped at the section or the reinforcement stress estimated at the limit state does not surpass 50% of the reinforcement's characteristic strength, otherwise α_4 = 1.0 may be used.

According to Model Code 2010, the stress developed in a lap may be taken as:

$$\sigma_{sd} = l_0/\varnothing.4/\alpha_4.\left[(\alpha_2 + \alpha_3).1.75.(25/\varnothing)^{0.3}.(f_{ck}/25)^{0.5} - 2P_{tr}\right]/\gamma_{cb} \tag{17}$$

Ref [44], specifies two distinct design bond stress-slip relationships in addition to the lap length and design bond strength. The designer determines the suitable relationship to use according to the mode of failure, which is confinement or bond failure. **Table 7** depicts the general bond stress-slip model.

For a well-confined concrete pull-out failure is expected when the clear spacing between bars is greater than 10∅ and the concrete cover is greater than 5∅. Another bond-slip model provided in ref. [44] is for splitting failure, the bond strength is obtained using the following expression:

$$\tau_{bu,split} = 6.5\left(f_{cm}/25\right)^{0.25}(25/\varnothing)^{0.2}\left[(c_{min}/\varnothing)^{0.33}(c_{max}/c_{min})^{0.1} + k_m k_{tr}\right] \tag{18}$$

Equation (4) is used to derived the splitting bond strength given as:

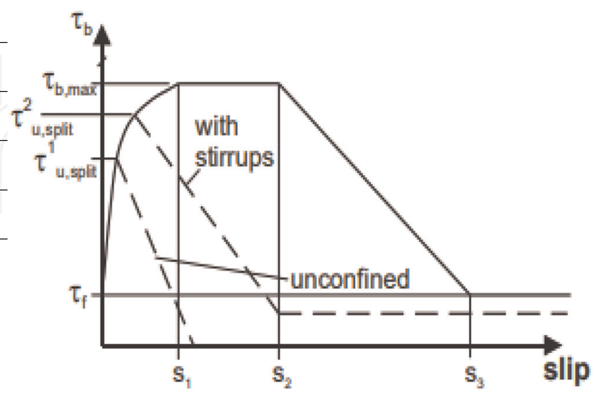
$$\tau_{u,split} = \left(\frac{\varnothing}{l_b}\right)\left(\frac{f_{stm}}{4}\right) = \left(\frac{\varnothing}{l_b}\right)\left(\frac{1}{4}\right)54\left(\frac{f_{cm}}{25}\right)^{0.25}\left(\frac{25}{\varnothing}\right)^{0.2}\left(\frac{l_0}{\varnothing}\right)^{0.55} \tag{19}$$

In addition, ref. [44] provides a coefficient for the influence of cyclic loading, longitudinal cracking, yielding, transverse cracking and stress.

$$\tau_{b,m} = \tau_0 \Omega_y \Omega_{p,tr} \Omega_{cr} \Omega_{cyc} \tag{20}$$

Where
Ω_{cyc} is the effect of cyclic loading
Ω_{cr} is the effect of longitudinal cracking $\Omega_{cr} = 1 - 1.2w_{cr}$
$\Omega_{p,tr}$ is the effect of transverse pressure $\Omega_{p,tr} = 1 - \tanh\left(0.2p_{tr}/0.1f_{cm}\right)$
Ω_y is the effect of yielding

	Splitting		Pull-out
	Stirrups	Unconfined	
τ_{max}	$8(F_{cm}/25)^{0.25}$	$7(F_{cm}/25)^{0.25}$	$2.5\sqrt{f_{cm}}$
s_1	$s(\tau_{max})$	$s(\tau_{max})$	1 mm
s_2	s_1	s_1	2 mm
s_3	$0.5c_{clear}$	$1.2s_1$	C_{clear}
α	0.4	0.4	0.4
τ_f	$0.4\tau_{max}$	0	$0.4\tau_{max}$



Where:
$\tau_0 = \tau_{max}(s/s_1)^{\alpha}$ *for* $0 \le s \le s_1$
$\tau_0 = \tau_{max}$ *for* $s_1 \le s \le s_2$
$\tau_0 = \frac{\tau_{bmax}(\tau_{max} - \tau_f)(s - s_2)}{(s_3 - s_2)}$ *for* $s_s \le s \le s_3$
$\tau_0 = \tau_f$ *for* $s_s < s$
c_{clear} *is the clear distance between the ribs*
τ_f *is the residual bond stress*

Table 7.
Bond slip relationship based on [44].

1.3.3 Eurocode 2

The Eurocode 2 design model for laps is based on [51]. In order to determine the design lap length in Eurocode 2, the design bond strength must first be determined, then the anchorage length. The bond strength is used to calculate the anchorage length with the following expression:

$$f_{bd} = 2.25\eta_1\eta_2 f_{ctd} \tag{21}$$

In this equation, η_1 is a coefficient associated with the bar position and bond conditions during concreting, with 0.7 representing all other conditions unity indicating good bond condition. The bond strength to tensile concrete strength ratio is described by the coefficient 2.25. When the diameter of the bar is smaller than 32 mm, the coefficient η_2 is considered as unity; otherwise, the following equation is used:

$$\eta_2 = \frac{132 - \varnothing}{100}\ \text{for}\varnothing > 32\ \text{mm} \tag{22}$$

The concrete design strength is f_{ctd} is obtained from $\frac{f_{\text{ctk,o.005}}}{\gamma_c}$ where is $f_{\text{ctk,o.005}}$ restricted to concrete grade C60/75. The concrete tensile strength is determined as a function of the compressive strength of concrete:

$$f_{ctd} = \frac{f_{ctk,o.005}}{\gamma_c} = \alpha_{ct} = 0.21(f_{ck})^{\frac{2}{3}}/\gamma_c \tag{23}$$

The coefficient α_{ct} which takes into consideration the loading and long-term effect of tensile strength is a nationally defined factor, the value recommended is 1.0. Equation (24) gives the safety considered in the Eurocode 2 anchorage design model. The bond strength is calculated using a 5%- fractile of the concrete tensile strength divided by the concrete's partial safety factor $\gamma_c = 1.5$. Therefore, the average bond strength required for comparing the test results is:

$$f_{bm} = 2.25\eta_1\eta_2 0{,}3 f_{ck}^{\frac{2}{3}} \tag{24}$$

The required basic anchorage length is obtained as:

$$l_{b,rqd} = \left(\frac{\varnothing}{4}\right)\left(\frac{\sigma_{sd}}{f_{bd}}\right) \tag{25}$$

The stress at the cross-section in which the anchorage length begins is the design stress of the bar σ_{sd}. The design length of the anchorage is determined as:

$$l_{bd} = \alpha_1.\alpha_2.\alpha_3.\alpha_4.\alpha_5.l_{b,rqd} \geq l_{b,\,min} \tag{26}$$

α_1 and α_2 are coefficient associated with bars shape and cover concrete, respectively. $\alpha_1 = 1.0$ for straight rebars.

$$\alpha_2 = 1 - 0.15\left(\frac{(c_{min} - \varnothing)}{\varnothing}\right) \leq 1.0$$

α_3 is the coefficient of shear link if present, with

$$0.7 \le \alpha_3 = 1 - k\lambda = 1 - \frac{k(\sum A_{st} - \sum A_{st,min})}{A_s} \le 1.0 \tag{27}$$

If there is no transverse pressure and welded transverse reinforcement, α_4 and α_5 can be taken as unity. However, when transverse pressure is present, the anchorage length can be decreased by:

$$0.7 \le \alpha_5 = 1 - 0.004p \le 1.0. = 1 - 0. \tag{28}$$

K accounts for the transverse reinforcement's efficacy in relation to its position inside the section. The difference in cross-section area between the minimum transverse reinforcement $\sum A_{\mathrm{st,\,min}}$ and the transverse reinforcement provided along the anchorage length is described by the coefficient λ with $\sum A_{\mathrm{st}} = 0.25 A_{\mathrm{s}}$ for anchorage and $\sum A_{\mathrm{st,\,min}} = 1.0 A_{\mathrm{s}} \left(\frac{\sigma_{\mathrm{sd}}}{f_{\mathrm{yd}}}\right) \ge 1.0 A_{\mathrm{s}}$ for laps.

Eurocode 2 recommended that laps should be positioned in a low moment region and staggered. The clear lapped spacing between bars should not exceed 50 mm or 4∅. If all bars are in the layer, the permissible proportion of lapped bars in tension is 100% and should not exceed 50% for lapped bars in several layers.

The basic required anchorage length and the coefficients are included in the design of lap length $l_{\mathrm{b,rqd}}$, which takes into account the key impacting parameters.

$$l_0 = \alpha_1.\alpha_2.\alpha_3.\alpha_4.\alpha_5.\alpha_6 l_{b,rqd} \ge l_{0,\,min} \tag{29}$$

The coefficient α_1 to α_5 described above. For the proportion of bars lapped at a section, the coefficient α_6 can be taken from **Figure 11**.

The coefficient α_6 may also be determined as follows:

$$1.0 \ge \alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5} \le 1.5 \tag{30}$$

		Bond condition, (see Figure 1)	Reinforcement in tension, bar diameter, ϕ (mm) 8	10	12	16	20	25	32	40	Reinforcement in compression
Anchorage length, l_{bd}	Straight bars only	Good	230	320	410	600	780	1010	1300	1760	40ϕ
		Poor	330	450	580	850	1120	1450	1850	2510	58ϕ
	Other bars	Good	320	410	490	650	810	1010	1300	1760	40ϕ
		Poor	460	580	700	930	1160	1450	1850	2510	58ϕ
Lap length, l_0	50% lapped in one location ($\alpha_6 = 1.4$)	Good	320	440	570	830	1090	1420	1810	2460	57ϕ
		Poor	460	630	820	1190	1560	2020	2590	3520	81ϕ
	100% lapped in one location ($\alpha_6 = 1.5$)	Good	340	470	610	890	1170	1520	1940	2640	61ϕ
		Poor	490	680	870	1270	1670	2170	2770	3770	87ϕ

Notes

1 Nominal cover to all sides ≥ 25 mm, [i.e. $a_2 \le 1$]. At laps, clear distance between bars ≤ 50mm.

2 $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$. For the beneficial effects of shape of bar, cover and confinement see Eurocode 2, Table 8.2.

3 Design stress has been taken as 435 MPa. Where the design stress in the bar at the position from where the anchorage is measured, σ_{sd}, is less than 435 MPa the figures in this table can be factored by σ_{sd}/435. The minimum lap length is given in cl 8.7.3 of Eurocode 2.

4 The anchorage and lap lengths have been rounded up to the nearest 10 mm.

5 Where 33% of bars are lapped in one location, decrease the lap lengths for '50% lapped in one location' by a factor of 0.82.

6 The figures in this table have been prepared for concrete class C25/30; refer to Table 13 for other classes or use the following factors for other concrete classes.

Concrete class	C20/25	C28/35	C30/37	C32/40	C35/45	C40/50	C45/55	C50/60
Factor	1.16	0.93	0.89	0.85	0.80	0.73	0.68	0.63

Figure 11.
Lap and anchorage length for concrete class C25/30 (mm). (Extract from [45]).

with

ρ_1 is the proportion of lapped bars at a section

The placing of shear links at the outer section of the lap length is required by Eurocode 2 for the concentration of splitting forces at lap ends. Shear links required for other reasons might be assumed sufficient if the proportion of lapped bars is less than 25%. The cross-sectional area of the shear link must not be lower than the cross-sectional area of one lapped bar for laps with a diameter higher than or equal to 20∅.

Equation (29) must be rearranged to obtain the bar developable stress. This enables the computation of experimental data and compared the various design model.

$$\sigma_{sd} = (l_0/\varnothing)\left(\frac{4f_{bd}}{\alpha_1\alpha_2\alpha_3\alpha_3\alpha_5\alpha_6}\right) \tag{31}$$

For straight bars without shear pressure and good bond condition, the average bar stress may be calculated as:

$$\sigma_{sd} = (l_0/\varnothing)\left(\frac{2.7f_{ck}^{2/3}\eta_2}{\alpha_2\left(1-k\left(\frac{\sum A_{st}-\sum A_{st,min}}{A_s}\right)\right)\alpha_6}\right) \tag{32}$$

1.3.4 American concrete institute

Reference [52] developed the design model that is used in American Concrete Institute (ACI). The design anchorage length is determined as:

$$\sigma_{sd} = \left(\frac{3}{40}\right)\left(f_y/\lambda\sqrt{f_c'}\right)\left(\frac{\psi_t\psi_c\psi_s}{\frac{c_b+k_{tr}}{\varnothing}}\right)\varnothing \tag{33}$$

f_y is the bar yield stress

$c_b = \min\left\{\frac{(c_s+\varnothing)}{2}; c_y+\frac{\varnothing}{2}; c_x+\frac{\varnothing}{2}\right\}$, because the cover values are connected to the bar centre in ACI, the values $\frac{\varnothing}{2}$ have to be added to comply with the notion.

f_c is the cylinder concrete strength limited to 69 MPa

ψ_c is the coefficient for coated reinforcement (for uncoated bars $\psi_c = 1.0$)

ψ_s is the coefficient for bar diameter (1.0 for bars ≥ 22 mm, otherwise 0.8)

ψ_t is the coefficient for bond (for good bond condition $\psi_t = 1.0$)

k_{tr} is shear link index

$$k_{tr} = \frac{40A_{st}[in.^2]}{s[in.]n_b} \tag{34}$$

$$\frac{c_b+k_{tr}}{\varnothing} \tag{35}$$

Where

S is the shear link spacing

A_{st} is the cross-sectional area of all shear links within the spacing

n_b are the number of lapped bars

Pull-out failure becomes more probable for confinement ratios $\frac{(c_b+k_{tr})}{\varnothing}$ above 2.5, and increasing transverse reinforcement or concrete cover does not result in enhanced bond capacity. [53] reported that for $\frac{c_b+k_{tr}}{\varnothing} > 3.75$, the bond capacity did not increase, thus the limit of 2.5 provides extra safety [50].

For normal weight concrete, uncoated reinforcement, and good bond conditions, Eq. (33) can be simplified to

$$l_b = \left(\frac{3}{40}\right)\left(\frac{f_y}{\sqrt{f'_c}}\right)\left(\frac{\psi_s}{\frac{c_b+k_{tr}}{\varnothing}}\right)\varnothing(in.) \tag{36}$$

Rearranging for the anchorage strength gives

$$\sigma_s = \left(\frac{40}{4}\right)\left(\frac{l_b}{\varnothing}\right)\sqrt{f'_c}\left(\frac{c_b+k_{tr}}{\varnothing}\right)\left(\frac{1}{\psi_s}\right)(\text{psi}) \tag{37}$$

If confinement by a compressive reaction is present at the simple support, the development length can be decreased by around 30%. It is worth noting that the ACI design guide permit bar diameters of up to 57 mm. For reinforcing bars with 43 and 57 mm diameters, the minimum allowable cover concrete is 38 mm in beams and 19 mm in slabs.

It must be noted that ref. [50] allows for bar diameters up to 57 mm. The minimum permissible concrete cover is 38 mm in beams and 19 mm in slabs (for Ø 43 mm and Ø 57 mm reinforcing bars: 38 mm). The smaller of the two values, the bar diameter or 25 mm, is the minimum clear bar spacing.

Due to a lack of sufficient experimental evidence, ACI does not allow laps of ACI does not permit laps of 43∅ and 57∅. **Table 8** shows the relation between the required and provided reinforcement and the coefficient for the proportion of bars lapped based on ACI recommendation. The coefficient 1.3 is not established on bond stress investigations, however, it is intended to promote the placement of laps away from high tensile stress regions to places where the reinforcement cross-sectional area provided as a minimum is twice that required by analysis. As a result, this coefficient contains a level of safety.

Smaller bar diameters with short lap lengths, the majority of which were less than 300 mm, were used to validate the design model. As a result, a factor for bar size γ is 1.0 and 0.8 for larger bar diameters and smaller bars not more than 22 mm were introduced. The ACI committee 408 advices against a size effect factor of less than 1.0.

1.3.5 German national annex

The coefficient for the minimum permissible bar spacing c_s and the tensile strength α_{ct} are the nationally determined parameters for lap and anchorage calculation for

Lap length	$A_{s,prov}/A_{s,req}$	**Percentage of A_s spliced**
max $\{1.0.\ l_b; 305\ \text{mm}\}$	≥ 2.0	50
max $\{1.3.\ l_b; 305\ \text{mm}\}$		100
	< 2.0	All cases

Table 8.
Spliced length coefficient for the percentage of lapped bars and the provided reinforcement [50].

Eurocode 2. The National annex defines the bars spacing as $c_s = 1.\varnothing$, and the tensile strength $\alpha_{ct} = 1.0$ for bond strength calculation.

The anchorage length at direct supports may be computed using $\alpha_5 = 2/3$ taking transverse pressure into consideration. For anchorages, National annex suggests using a simple cover concrete coefficient of $\alpha_2 = 1.0$. The cover concrete orthogonal to the lap plane c_y is not taken into consideration for spliced rebars with straight ends, based on the National annex. For laps, $c_{\min} = \min\left\{\frac{c_s}{2}0; c_x\right\}$ is the cover used for the calculation of the coefficient α_2. The lap factor α_6 is determined by the bar diameter and the proportion of bars lapped at a location. The recommended values for α_6 are shown in **Table 9**.

If more than 50% of the reinforcement is spliced at one location, according to Eurocode 2, the transverse reinforcement in laps shall be made by links. The National annex relaxes this requirement by specifying that the transverse reinforcement does not need to have a longitudinal spacing of $0.5l_0$ between the adjacent laps centres or consist of links with the distance between adjacent laps greater than 10∅.

1.3.6 PTI working draft

The PTI working draft offers a revised design model based on Fib Bulletin 72 for the next generation of Eurocode 2. For ease of use, the exponents were simplified. The required bond strength for good bond conditions comes from

$$l_{bd,req.} = 40\left(25\,MPa/f_{ck}\right)^{1/2}\left(\frac{\sigma_{sd}}{435MPa}\frac{\gamma_c}{1.5}\right)^{3/2}(\varnothing/20\;mm)^{1/3}\left(1.5\varnothing/c_{d,c\;onf}\right)^{1/2} \le l_{b,min} \tag{38}$$

with

$l_{\mathrm{b,\,min}}$ is the minimum lap length with 20∅ for laps

$$c_{d,c\;onf} = c_d + \left(30k_{conf}\frac{n_l A_{st}}{n_b \varnothing s_{st}} + 8\sigma_{ctd}/\sqrt{f_{ck}}\right)\varnothing \le 3.75\varnothing \tag{39}$$

σ_{ctd} is the design value of the mean compression stress perpendicular to the potential splitting plane

K_{conf} is the effectiveness factor. K_{conf} is taken as 0.25 for shear links within the cover c_y with cover spacing greater than 8∅, and 1.0 for confinement reinforcement crossing the potential splitting plane (the maximum recommended distance from the leg to the lapped bar is less than 5∅). In other circumstances, K_{conf} is taken as zero.

Percentage of bars lapped at a section in one layer		**Bar diameter ∅**
$\geq 33\%$	$\leq 33\%$	
1.4 (1.0)	1.2 (1.0)	< 16 mm
2.0 (1.4)	1.4 (1.0)	≥ 16 mm
The values in brackets are valid for $c_s \geq 8\varnothing$ *and* $c_x \geq 4\varnothing$.		

Table 9.
German National Annex for the percentage of a spliced bar under tension based on [45].

The PTI working draft recommended the laps be designed for $1.2 \times \sigma_{sd}$ if tension laps are located in regions where the yield strength may be exceeded. Therefore, a reduction of lapped bars or confining reinforcement is required. Reliability analysis of the coefficients 8,30, and 40 in expressions (38) and (39). Rearranging for the developable stress in anchorages gives

$$f_{std} = 435(1.5/\gamma_c)^{1/3}(20/\varnothing)^{2/9}(l_b/40\varnothing)^{2/3}\left(c_{d,conf}/1.5\varnothing\right)^{1/3} \tag{40}$$

1.3.7 Canbay and Frosch

Four hundred and eighty experiments with and with shear links were used by [54] to verify their model for ultimate lap load. The ultimate lap strength at splitting failure, taking shear link into account is defined as

$$\sigma_{sd} = F_{split} + F_{st}/n_b A_s \tan\beta = 2.75/n_b A_s\left(F_{split} + F_{st}\right)(\text{ksi}) \tag{41}$$

Where

θ inclination of struts commencing at the rib flanks (20 degrees provided optimal results)

F_{st} is the splitting resistance by a shear link [kip]

F_{split} is the splitting resistance by cover concrete along the lap length [kip]

[54] distinguish side (**Figure 12**, middle) and face splitting types (**Figure 12**, left).

It is assumed that the tensile concrete stress surrounding the bars is linear along the lap length. The bond strength nonlinearity along the splice length is taken into account by using an efficient lap length l_0^*. The side-splitting force $F_{split,side}$ is

$$F_{split,side} = l_0^*\left[2c_x^* + 2c_x^*(n_b - 1)\right]6f_c^{1/2}(\text{kip}) \tag{42}$$

For face-splitting, the splitting force $F_{split,face}$ is

$$F_{split,face} = l_0^*\left[2c_y^*\left(0.1\frac{c_x}{c_y} + 0.9\right) + 2c_y^*(n_b - 1)\left(0.1\frac{c_s}{2c_y} + 0.9\right)\right]6f_c^{1/2} \tag{43}$$

With

$c_y^*, c_x^*, (\frac{c_s}{2})^*$ coefficients for the efficiency of concrete cover at linear stress distribution

$$l_0^* = l_0\frac{9.5I}{\sqrt{l_0/\varnothing f_c^{\frac{1}{4}}}} \le l_0$$

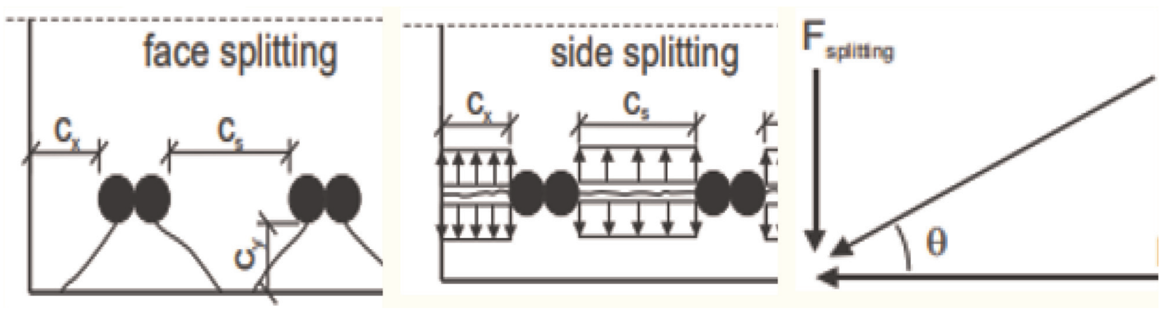


Figure 12.
Face splitting (left), side splitting (middle) and force distribution at bond forces (right). [Adapted from [54]].

l_0 coefficient for the efficiency of lap length at linear stress distribution

In the failure of splitting, shear links provide extra resistance. The splitting force $F_{st,\ side}$ provided by the shear link, is given by expression (44). For side splitting, this expression includes the number of legs crossing the splitting plane in

$$F_{st,side} = n_{st} n_l A_{st} f_{yt} \left[\frac{n_b f_c^{1/2}}{170} \right] (\text{kip}) \tag{44}$$

Because the shear link crosses the splitting plane at each bar in face split failure, expression (45) incorporates the bar number instead of the leg number in determining the splitting force.

$$F_{st,face} = n_{st} n_b A_{st} f_{yt} \left[\frac{n_b f_c^{1/2}}{170} \right] (\text{kip}) \tag{45}$$

62 MPa is the suggested transverse bar yield strength. A safety factor of 1.2 was utilised to develop the design expression (41). 50% of the calculated strength was unsafe without the safety factor. Only 16% of the confined and 10% of the unconfined test meet their yield strength when the factor of 1.2 was used. This take into consideration the lap length designed based on the proposed model and nominal yield strength of 414 MPa.

2. Conclusions

This chapter provides the context and presents the state of art for using stainless steel as a structural material. It is demonstrated that stainless steel is a remarkable building material that is becoming a more desirable option for RC structures because of its recyclability, long life cycle favourable mechanical properties, ductility and excellent corrosion resistance. There are numerous factors preventing stainless steel reinforcement from being used more frequently in reinforced concrete structures. Firstly, mainly due to this is a new topic in structural engineering, there is a lack of performance data and design guidance available in the public domain. The second issue is that engineers have the perception that stainless steel reinforcement is expensive. Despite having a higher initial cost than carbon steel, stainless provides efficient and a very competitive design option throughout the duration of a structure's lifetime when rehabilitation and maintenance expenses are taken into account.

It is important that structurally effective design solutions are available that take into account and exploit the advantageous and distinctive properties of stainless given its high initial cost. Unfortunately, current design guidance, such as Eurocode 2, does not have an effective method for designing structures with stainless steel reinforcement; rather, they include inadequate material models for reinforcing bar that do not make use of the unique properties of stainless steel. While this premise could be valid for concrete with carbon steel reinforcement, it provides widely erroneous estimates when suing stainless steel reinforcement. This is mostly due to stainless steel's early stage of nonlinear behaviour and its high strain hardening. The design of stainless steel reinforced concrete members using the current design criteria for carbon steel RC structures is thus neither accurate nor efficient.

Additionally, it has been reported that there is little discussion of the bond behaviour of stainless-steel reinforcement in the literature and that the research that is available is both conflicting and limited. The existing design guidelines, such as Model Code 2010 and Eurocode 2, generally recommend employing the same standard as for traditional carbon steel reinforcement and do not provide particular lap length design guidelines for designing stainless steel RC structure without appropriate test data may not be a safe option because it has been observed that stainless steel reinforcement may develop a lower bond strength than carbon steel reinforcement.

Conflict of interest

The authors declare no conflict of interest.

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References

[1] Shamass R, Cashell K. Analysis of stainless steel-concrete composite beams. Journal of Constructional Steel Research. 2018;**152**:132142

[2] Baddoo NR. Stainless steel in construction: a review of research, applications, challenges and opportunities. Journal of Constructional Steel Research. 2008;**64**(11):1199-1206

[3] Gardner L. The use of stainless steel in structures. Progress in Structural Engineering and Materials. 2005;**7**(2): 45-55

[4] Eladly MM. Behaviour of stainless steel beam-to-column bolted connections–Part 1: simplified FE model. Journal of Constructional Steel Research. 2020;**164**:105-784

[5] Rabi M, Cashell KA, Shamass RJES. Flexural analysis and design of stainless steel reinforced concrete beams. Engineering Structures. 2019;**198**:109432

[6] EN 10088-2. Stainless steels—Part 2: technical delivery conditions for sheet/ plate and strip of corrosion resisting steels for general purposes. European Committee for Standardization (CEN). 2014

[7] Evans K. RB rebak in corrosion science—A retrospective and current status in honor of Robert P. Corrosion Science. 2002;**13**:344-354

[8] Ramberg W, Osgood WR. Description of Stress-Strain Curves by Three Parameters. Technical Note No. 902, National Advisory Committee for Aeronautics, Washington DC. 1943

[9] Rabi M, Cashell KA, Shamass R. Ultimate behaviour and serviceability analysis of stainless steel reinforced concrete beams. Engineering Structures. 2021;**248**:113259

[10] Markeset G, Rostam S, Klinghoffer O. Guide for the use of stainless steel reinforcement in concrete structures. Norway. 2006

[11] Sharif AM, Al-Mekhlafi GM, Al-Osta MA. Behavior of circular stainless steel stub columns internally strengthened by longitudinal carbon steel bars. Engineering Structures. 2019;**199**: 109-617

[12] Rabi M, Cashell KA, Shamass R. Analysis of concrete beams reinforced with stainless steel. In: Proceedings of the Fib Symposium 2019: Concrete-Innovations in Materials, Design and Structures. 2019. pp. 690-697

[13] Nationwide Stainless. 2021. Stainless Steel Rebar. Available from: https://www.nationwidestainless.co.uk/products/stainless_steel_rebar/

[14] Rabi MM. Analysis and Design of Stainless Steel Reinforced Concrete Structural Elements [doctoral dissertation]. London: Brunel University; 2020

[15] Baddoo R, Burgan A. Structural Design of Stainless Steel SCI, Publication No. P291. 2012

[16] Rabi M, Cashell KA, Shamass R, Desnerck P. Bond behaviour of austenitic stainless steel reinforced concrete. Engineering Structures. 2020; **221**:111027

[17] Rabi M, Shamass R, Cashell KA. Structural performance of stainless steel reinforced concrete members: A review. Construction and Building Materials. 2022;**325**:126673

[18] Cramer S, Covino B, Bullard S, Holcomb G, Russell J, Nelson F, et al. Corrosion prevention and remediation strategies for reinforced concrete coastal bridges. Cement and Concrete Composites. 2002;**24**(1):101-117

[19] Rabi M, Shamass R, Cashell KA. Experimental investigation on the flexural behaviour of stainless steel reinforced concrete beams. Structure and Infrastructure Engineering. 2022: 1-13

[20] British Stainless Steel Association. Classification of stainless steel types. 2000. Available from: https://www.bssa.org.uk/cms/File/SSAS1.2-Classification%20of%20Stainless%20Steels.pdf

[21] Aalco. Specifications for Stainless Steel Products. 2013

[22] Metals4U. 2021. Stainless steel reinforcing bar. Available from: https://www.metals4u.co.uk/stainless-steel/c8/reinforcing-bar/c1875

[23] Gardner L, Bu Y, Francis P, Baddoo NR, Cashell KA, McCann F. Elevated temperature material properties of stainless steel reinforcing bar. Construction and Building Materials. 2016;**114**:977-997

[24] Gedge G. Rationale for using stainless steel reinforcement in the UK construction industry. UK. 2003

[25] McGurn J. Stainless steel reinforcing bars in concrete. In: Proceedings of the International Conference of Corrosion and Rehabilitation of Reinforced Concrete Structures. Orlando. 1998

[26] British Highways Authority. Advice note: design manual for roads and bridges BA84/02: the use of stainless steel reinforcement in highway structures. 2003. Available from: https://www.bssa.org.uk/cms/File/REBar%20report.pdf

[27] Fan S, Ding X, Sun W, Zhang L, Liu M. Experimental investigation on fire resistance of stainless steel columns with square hollow section. Thin-Walled Structures. 2016;**98**:196-211

[28] Castro H, Rodriguez C, Belzunce F, Canteli A. Mechanical properties and corrosion behaviour of stainless steel reinforcing bars. Journal of Materials Processing Technology. 2003;**143**: 134-137

[29] Medina E, Medina JM, Cobo A, Bastidas DM. Evaluation of mechanical and structural behavior of austenitic and duplex stainless steel reinforcements. Construction and Building Materials. 2015;**78**:1-7

[30] DMRB. DMRB 1.3.12. Design manual for roads and bridges (DMRB). 2003

[31] BA 84/02. Design manual for roads and bridges part 15: use of stainless steel reinforcement in highway structures. Highways Agency UK. 2003

[32] Bautista A, Blanco G, Velasco F, Gutiérrez A, Palacín S, Soriano L, et al. Passivation of duplex stainless steel in solutions simulating chloride contaminated concrete. Materiales de Construcción. 2007;**57**(288):17-32

[33] Pérez-Quiroz J, Terán J, Martínez M, Genescá J. Assessment of stainless steel reinforcement for concrete structures rehabilitation. Journal of Constructional Steel Research. 2008;**64**(11):1317-1324

[34] Huang Y, Young B. Stress–strain relationship of cold-formed lean duplex stainless steel at elevated temperatures. Constructional Steel Research. 2014;**92**: 103-113

[35] Lopes N, Real PV, da Silva LS, Franssen J. Numerical analysis of stainless steel beam-columns in case of fire. Fire Safety Journal. 2012;**50**: 35-50

[36] Tondini N, Rossi B, Franssen J. Experimental investigation on ferritic stainless steel columns in fire. Fire Safety Journal. 2013;**62**:238-248

[37] Helland S. Design for service life: implementation of fib Model Code 2010 rules in the operational code ISO 16204. Strcural Concrete. 2013;**14**(1):10-18

[38] García-Alonso M, Escudero M, Miranda J, Vega MI, Capilla F, Correia M, et al. Corrosion behaviour of new stainless steels reinforcing bars embedded in concrete. Cement and Concrete Research. 2007;**37**(10): 1463-1471

[39] Nurnberger U. Stainless Steel in Concrete: State of the Art Report. 1996

[40] Serdar M, Žulj LV, Bjegović D. Long-term corrosion behaviour of stainless reinforcing steel in mortar exposed to chloride environment. Corrosion Science. 2013;**69**:149-157

[41] Bond AJ, Brooker O, Harris AJ, Harrison T, Moss RM, Narayanan RS, et al. How to Design Concrete Structures to Eurocode 2. 2nd ed. London: MPA The Concrete Centre; 2018

[42] Zhou Y, Ou Y, Lee GC. Bond-slip responses of stainless reinforcing bars in grouted ducts. Engineering Structures. 2017;**141**:651-665

[43] Ahlborn T, DenHartigh T. Comparative bond study of stainless and highchromium reinforcing bars in concrete. Transportation Research Record: Journal of the Transportation Research Board. 2003;**1845**:88-95

[44] Model Code for Concrete. 2013. Fib Model Code for Concrete Structures 2010. fib Model Code for Concrete Structures 2010. Wiley. DOI: 10.1002/9783433604090

[45] EN 1992-1-1. Eurocode 2: design of concrete structures part 1-1: General rules and rules for buildings. European Committee for Standardization (CEN). 2004

[46] PT.SC2.T1. PTI working draft prEN 1992-1-1:2018. European Committee for Standardization. European Committee for Standardization; CEN, European Committee for Standardization. 2018

[47] Fib Bulletin 72. Bond and anchorage of embedded reinforcement: Background to the fib Model Code for Concrete Structures 2010. Lausanne, Switzerland. 2014

[48] Eligehausen R. Übergreifungsstöße zugbeanspruchter Rippenstäbe mit geraden Stabenden. Stuttgart Germany: Wilhelm Ernst & Sohn; 1979

[49] Ferguson PM, Briceno EA. Tensile lap splices part 1: Retaining wall type varying moment zone. No. 113-2. Centre for Highway Research. Texas. 1969

[50] ACI committee 318. Building Code Requirements for Structural Concrete and Commentary (ACI 318M-11). American Concrete Institute, Farmington Hills, MI. 2011

[51] CIB-FIP Model Code. CEB-FIP model code 1990: design code [online]. 1991. Bulletin d'Information. London: Thomas Telford Services Ltd. Available from: http://books.google.com/books?hl=en&lr=&id=IRG9GTPJ7s8C&oi=fnd&pg=PR5&dq=CEB-FIP+model+code+1990:+Design+Code&ots=756sbOWzgo&sig=x2_W4HCZLIqwahl62aMrKidAcTU [Accessed: July 22, 2014]

[52] Orangun CO, Jirsa JO, Breen JE. A reevaulation of test data on development length and splices. ACI Journal Proceedings. 1977;**74**(3):114-122

[53] Darwin D et al. Development length criteria for conventional and high relative rib area reinforcing bars. ACI Structural Journal. 1996;**93**(3):512-516

[54] Canbay E, Frosch RJ. Bond strength of lap-spliced bars. ACI Structural Journal. 2005;**102**(4):605-614. DOI: 10.14359/14565