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Guide for the use of stainless steel reinforcement in concrete structures

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Nordic Innovation Centre project – 04118:
«Corrosion resistant steel reinforcement in concrete structures (NonCor)»

Foreword

Premature deterioration of concrete buildings and infrastructure due to corrosion of reinforcement is a severe challenge, both technically and economically. Repair-work on the public transportation infrastructure are causing significant inconveniences and delays for both the industry and the general public, and are now recognized as a substantial cost for the society.

In recent years there has been an increasing interest in applying stainless steel reinforcement in concrete structures to combat the durability problems associated with chloride ingress. However, the use of stainless steel reinforcement (SSR) has so far been limited mainly due to high costs and lack of design guides and standards.

In 2004 a Scandinavian group was established to cope with these challenges and a Nordic Innovation Centre project: "Corrosion resistant steel reinforcement in concrete structures (NonCor)" was formed.

The present report; "Guide for the use of stainless steel reinforcement in concrete structures", is the final document of this project. The scope of this Guide is to increase the durability and service life of concrete structures exposed to corrosive environments by focusing on two issues:

- Eliminating reinforcement corrosion by examining the core of the problem, i.e. the reinforcement itself
- Overcoming the technical knowledge gap for application of stainless steel reinforcement in concrete structures

The foreseen users of this Guide are:

- All parties involved in planning, design and construction of concrete structures to be exposed to corrosive environments, - such as marine structures, coastal-near structures and structures exposed to chloride based de-icing salts
- Owners and Clients who want to reduce or solve the corrosion problem for reinforced concrete structures, in order to obtain a long service life with minimal maintenance

The participants in the Nordic Innovation Centre (NICe) project are:

Norwegian Building Research Institute (Project Manager)
Norwegian Defence Estates Agency [*]
Norwegian Public Roads Administration [*]
Veidekke ASA, Norway
Danish Road Directorate [*]
COWI A/S, Denmark
Force Technology, Denmark
Arminox, Denmark
MT Højgaard a/s, Denmark
Swedish Road Administration [*]
Strängbetong, Sweden

[*] financing partners (in addition to NICe) and members of the Steering Committee

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1   Introduction

1.1   The corrosion problem

Premature deterioration of concrete buildings and infrastructure due to corrosion of reinforcement is a severe challenge, both technically and economically. It has been estimated that Western Europe spends 5 billion Euros yearly for repair of corroding concrete infrastructures. Repair-work on the public transportation infrastructure are causing significant inconveniences and delays for both the industry and the general public, and are now recognized as a substantial cost for the society.

The main sources of chloride ingress stems from seawater splash (on marine based structures) as well as from de-icing salts (on roads, bridges, parking decks and on external staircases and access balconies in large condominiums).

Carbon steel reinforcement embedded in concrete will not normally corrode due to the formation of a protective ion-oxide film, which passivates the steel in the strong alkaline conditions of the concrete pore water. However, this passivity may be destroyed by chlorides penetrating through the concrete, or due to carbonation, reaching the surface of the reinforcement. Corrosion, which is an electrochemical process involving establishment of corroding and passive sites on the steel surface, may then be initiated.

As a result of corrosion reaction, rust forms and occupies a volume of up to 6-7 times that of the original metal, hence generating bursting forces. These forces might exceed the tensile strength of concrete, causing cracking and spalling of the concrete leading to further corrosion and loss of bond between the concrete and the steel. Hazardous situations might occur when pieces of spalled concrete fall and threaten the user or passer-by, or when the structural member looses cross-sectional area and thereby experiences increased stress on the remaining section, which potentially could lead to structural failure.

Examples of damages caused by corrosion are illustrated in *Figure 1-1* through *Figure 1-4*.

Figure 1-1: Corrosion damage on bridge pier.

Figure 1-2: Corrosion damage in car park deck

Figure 1-3: Corrosion damage on marine structure

Figure 1-4: Corrosion damage on New Jersey type crash barrier

The means of addressing the corrosion problems have mainly followed one of five distinct strategies:

- Developing very dense and strong types of concrete to protect the reinforcement against ingress of corrosive substance, particularly chlorides, in combination with sufficient concrete covers

- Inhibiting corrosion through passive protection (corrosion inhibitors) or through active protection (cathodic protection/prevention)

- Developing coatings to the concrete surface or to the carbon reinforcement (particularly epoxy or zinc)

- Develop non-metallic reinforcements (glass fibre, aramid fibre or carbon fibre)

- Developing specially alloyed steel types with higher chloride threshold values for corrosion initiation

Replacing conventional carbon steel reinforcement with corrosion resistant steel reinforcement or with non-metallic reinforcement has only received limited attention in the Nordic countries. While non-metallic reinforcement (carbon-, aramid- or glass fibre) still is in a R&D phase, corrosion resistant steel reinforcement in the form of stainless steel reinforcement (SSR) has been readily available commercially for the last say 10 years. The use of SSR has so far been limited mainly due to high costs and lack of design guides and standards.

A convincing documentation of the performance of stainless steel reinforcement in highly chloride contaminated concrete may be found at Progresso in Mexico. Here you find still operating, a 70 year old 2.2 km long concrete pier leading out into the Gulf of Mexico. This pier was at construction reinforced with stainless steel reinforcing bars (quality 1.4301). No corrosion has taken place within the structure, despite the harsh environment and poor quality materials used in the construction see *Figure 1-5*. The chloride levels, at the surface of the reinforcement were more than 20 times the traditionally assumed corrosion threshold level, see *Figure 1-6, /1/*.

A newer, parallel pier was built in 1972. This structure perished after only 11 years of service due to reinforcement corrosion of the ordinary carbon steel reinforcement used in this structure. The remains of this structure may be seen in the foreground of *Figure 1-7,* and on *Figure 1-8*.

Figure 1-5: Typical picture of breakout exposing non-corroding SSR /1/
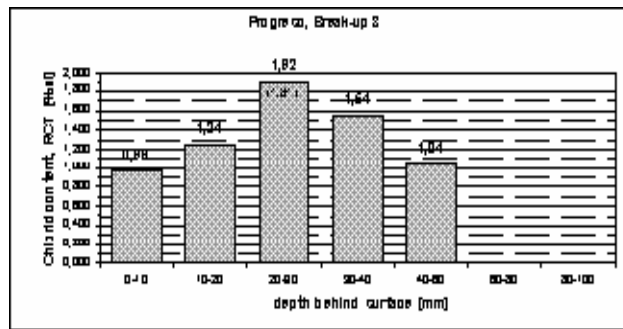
Figure 1-6: Typical chloride profile with 1.92 % chloride-ion by weight of concrete at the level of reinforcement /1/

Figure 1-7: The "new" shorter Progreso Pier built about 1972 and the old one behind /1/

Figure 1-8: Close-up of the land-based remains of the "new" pier as seen on Figure 1-7 /1/

1.2 Scope

Reinforcement corrosion remains the most serious cause of deterioration of concrete structures, and thus reduced service life. The scope of this Guide is to increase the durability and service life of concrete structures exposed to corrosive environments by focusing on two issues:

- Eliminating reinforcement corrosion by examining the core of the problem, i.e. the reinforcement itself

- Overcoming the technical knowledge gap for application of stainless steel reinforcement in concrete structures

The report aims to give some basic information regarding choice of stainless steel reinforcements. In addition, the report provides the basis to understand the state-of-the-art methodologies for service life design of reinforced concrete structures and how stainless steel reinforcements should be applied within this concept.

The foreseen users of this Guide are:

- All parties involved in planning, design and construction of large concrete structures to be exposed to corrosive environments, such as marine structures, coastal-near structures and structures exposed to chloride based de-icing salts

- Owners and Clients who want to reduce or solve the corrosion problem for reinforced concrete structures, in order to obtain a long service life with minimal maintenance

It is assumed that the users of this Guide have a detailed knowledge of both the design and durability aspects of concrete structures exposed to a corrosive environment. The users should therefore be acquainted with the terminology used.

2 Service life design of concrete structures

2.1 General

During recent years Owners of large structures - infrastructures constructions as well as buildings - have focused increasingly on durability. This has repeatedly been expressed as a requirement to a specific design service life ranging typically from 60 - 100 - 120 - 300 years.

A 60 years design service life was specified for the Bahrain Financial Harbour, *Figure 2-1*, located in the most corrosive environment in the World. For the Great Belt Link, Denmark (Tunnel and Bridges), *Figure 2-2* the design service life requirement was 100 years. An extraordinary long design service life of 300 years was specified for the Messina Strait Bridge, *Figure 2-3*, with a world record span of 3000 meters.

Figure 2-1: Bahrain Financial Harbour. Design service life is 60 years.

Figure 2-2: Great Belt East Bridge. Design service life is 100 years.

Figure 2-3: Messina Strait Bridge. Design service life is 300 years. Main span in steel is 3000 m. (Model Photo).

Because reinforcement corrosion remains the most serious cause of deterioration of concrete structures, a particular focus on reducing - or solving - this corrosion problem becomes a key issue for all designers and contractors of concrete structures. Therefore, this Guide starts with this short section on "Service life design of concrete structures". This fact is considered the reference point for the subsequent detailed descriptions on the different aspects of adopting stainless steel reinforcement.

2.2 Durability and service life

The fundamental conditions to be fulfilled in order to ensure durable structures with low maintenance cost are the performance requirements stated by the Owner, when ordering a structure through his Designer. Durability and service life related qualities need primarily to be enforced by the Owner. He - or his representative - must therefore:

- **define** the quality he wants (i.e. the service life and the pre-conditions)

- **check** the quality received (QA, QC, on site supervision)

- **maintain** the structure - or have the structure maintained by the User - to ensure satisfactory performance (foreseen Maintenance Management).

- **pay** for the quality he specifies as well as for the necessary running activities to maintain the structure in satisfactory condition.

Durability is a non-quantifiable term and thereby not operational in design. Therefore the concept of a **service life** - a number of years - has become the Owner's term to specify his long-term functional requirements.

The design service life is assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. In practice there are three different types of service life relevant for structures, depending on the type of performance being considered:

- *Technical service life,* the time in service until a defined unacceptable state of deterioration is reached.

- *Functional service life,* the time in service until the structure becomes obsolete from a functional performance point of view, due to changed requirements.

- *Economic service life,* the time in service until replacement of the structure is economically more attractive than keeping the structure in service.

Only the technical service life and its interaction with the economic service life are relevant for the design of new structures and for the residual service life evaluation of existing structures.

It should be realized that the end of service life is often in reality when a structure becomes functionally obsolete, i.e. shall accommodate higher loads, larger free space under bridges, changed functionality, etc. The structural design shall therefore ensure a technical service life at least as long as the envisaged functional service life.

The design service life defined by the Owner shall be identified through recognizable and verifiable criteria in order to constitute part of the design basis. The required service life shall be supplemented by additional requirements or criteria to be operational, such as:

- The definition of the "end-of-service-life" (i.e. definition of the relevant limit state)

- The type, detailing and frequency of future inspection and maintenance being acceptable to the Owner

- The level of reliability of the service life design

Examples of end-of-service-life situations are:

- Structural safety is dropping below acceptable limits;

- Serious material degradation, such as reinforcement corrosion due to chloride attack, concrete disintegration due to alkali-aggregate reaction, etc. requires extensive or costly maintenance or inconveniences to the Owner or user;

- Appearance becomes unacceptable;

- Functional requirements cannot be fulfilled, i.e. the structure becomes functionally obsolete.

The development in time of nearly all types of deterioration mechanisms in concrete structures may be modelled by the two-phase curve as shown in *Figure 2-4*:

- **The initiation period**, during which the transport and the build-up of aggressive substance occur
    (Carbonation, chloride penetration and sulphate accumulation are examples of such mechanisms determining the duration of the initiation period)

- **The propagation period,** during which the active deterioration develops, often at a high rate
    (Reinforcement corrosion is one such important example of deterioration)

In order to adopt an operational approach to the service life design, the Designer may define a target service life, which may deviate from the Owners design service life (or technical service life). The typical example is illustrated in *Figure 2-4*.
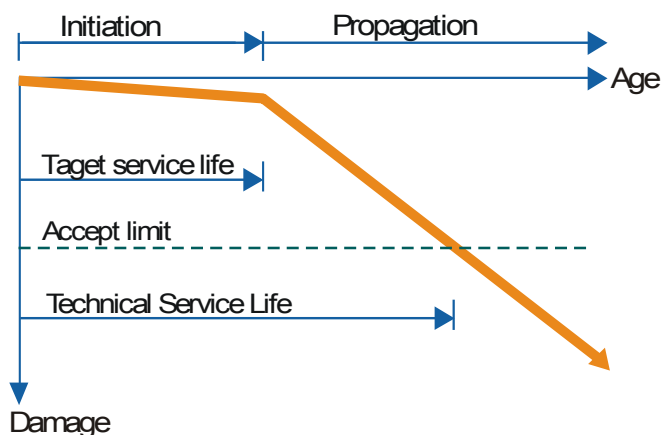
Figure 2-4: Service life of concrete structures. A two-phase modelling of deterioration /2/

The Owner may accept some degree of reinforcement corrosion leading to a defined safety reduction or cracking and spalling, or discolouring of the structure to represent his assumed end of service life.

However, quantifying and verifying this stage during the design is difficult. Therefore, the Designer may adopt the transition from the initiation period to the propagation stage, as illustrated in *Figure 2-4*, to represent the target service life adopted in the design.

2.3 Service life design process

Through the service life design process the Designer shall take the following actions:

- Identify the specific physical actions and aggressive chemical substances, which may be expected from the environment in question during the foreseen service life

- Design the geometrical form of the exposed parts of the structure to be robust in resisting the ingress of the aggressive substance from the environment

- Select the type of reinforcement considered optimal in combination with the chosen concrete and the detailing of the structure and the reinforcement layout

- Select the cement type and concrete quality which would be able to resist the possible deleterious actions identified, provided the design, execution and assumed maintenance is adapted accordingly

- Design the concrete cover and consider crack widths and crack orientations relative to the type of reinforcement chosen

2.3.1   Design strategies

In *fib* Model Code for Service Life Design /3/ two basic design strategies has been adopted, whereof the first introduces three levels of sophistication. In sum four options are available, i.e.:

Strategy 1:   Level 1: Full probabilistic approach (Option 1)

Level 2: Partial factor design approach (Option 2)

Level 3: Deemed to satisfy design approach (Option 3)

Strategy 2:   Avoidance of deterioration design approach (Option 4)

- The *"Full probabilistic design approach"* (Strategy 1/Option 1) will be used only for exceptional structures. This approach has specifically been adopted for tunnels (e.g. The Dutch tunnels: Green Heart Tunnel and Western Schelde Tunnel, the Malmø City Tunnel in Sweden, the Copenhagen Metro and Copenhagen Harbour installation tunnel), and for bridges (e.g. the Busan George Fixed Link Bridges in Korea, the Sutong Bridge in China and the Sitra Causeway Bridges in Bahrain). For all these designs the probabilistic performance based durability and service life design, *DuraCrete/4/*, has been adopted.

- The *"Partial safety factor approach"* (Strategy 1/Option 2) is a deterministic approach where the probabilistic nature of the problem (scatter of material resistance and environmental load) is taken into account by partial safety factors.

- The *"Deemed to satisfy design approach"* (Strategy 1/Option 3) is comparable to the approach which can found in the codes and standards of today.

- The *"Avoidance of deterioration design approach"* (Strategy 2/Option 4) introduces the use of non-reactive materials, e.g. stainless steel reinforcement (SSR).

This Guide focuses on **Strategy 2**, "Avoidance of deterioration", and is an expansion of this strategy.

2.3.2   Life cycle costing

As service life design relates to the structure's performance over a long period of time it is relevant not only to consider the initial construction costs, but also the operation and maintenance costs over the expected design life of the structures. It is now recognized that for many structures the cost, difficulty and operational disruption resulting from both planned and unplanned maintenance and repair are significant burden to the owner of the structures as well as to the users. For example, the user costs due to traffic delays are now being rated so high, that this becomes the dominating basis for selecting the type and timing of maintenance and repairs.

An optimal design strategy should be an economic optimization of the costs throughout the whole life of the structure. In addition to actual financial costs (cost of the construction, repair and maintenance etc), user benefits, environmental effects and other external effects should be included in the economic analysis of the project. Such Life Cycle Cost (LCC) analysis

evaluates whether the project is beneficial to society as a whole. An example of this approach is given in Chapter 7.3.

2.3.3 Environmental aggressivity

In order to address the issue of time dependent deterioration of concrete structures during the design, it is one of the key task and responsibilities of the designer to carefully evaluate aggressivity of the environment that may cause premature deterioration. For instance, a corrosion deterioration mechanism depends on aggressive substance typically chlorides penetrating from the surrounding environment into the outer concrete layer of concrete (the cover). This substance diffuses inward, towards the reinforcement, see *Figure 2-5*. When a threshold value has been reached at the level of the reinforcement corrosion is initiated.
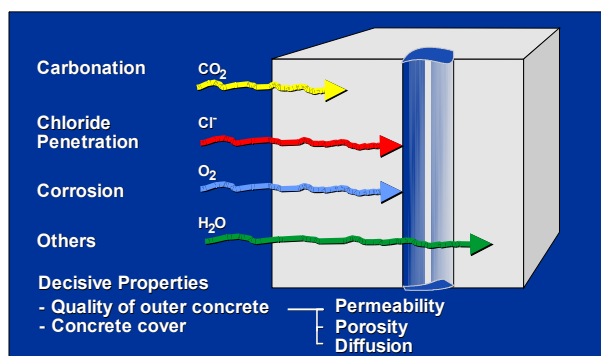
Figure 2-5: Importance of the outer concrete layer to protect the reinforcement /6/

As guidance to determine the environmental aggressivity, Eurocode 2 (EN 1992-1-1 /7/ and reference to EN 206-1 /8/) provides a valuable starting point. The documents also highlight the need to separate the aggressivity of the environment depending on whether it refers to reinforcement corrosion or concrete material deterioration. For certain projects where the Owner has set forward service life requirements exceeding the anticipated service life from national codes, a project specific definition or identification of the factual aggressivity of the environment has to be made.

2.3.4 Codes and standards

The design requirements to be fulfilled are stated in the national codes and standards. They represent the minimum requirements regarding safety and serviceability acceptable to the society.

Historically and for reasons of tradition codes and standards differ from country to country. So far they introduce durability through substitute requirements such as type of cement, minimum cement content, minimum concrete cover, maximum crack widths, but without any requirement to the duration of the performance. As stated in **fib** Model Code for Service Life Design /3/: *"... descriptive rules of today's standards are not based on physically and chemically correct models but more on practical (at times bad) experience. In the future currently applied rules urgently have to be calibrated against the full probabilistic approach…".*

The current Swedish, Norwegian and Danish concrete design standards, SS 137010 /9/, NS 3473 /10/ and DS 411 /11/, all address the service life issue. NS 3473 has criteria for 50 years and 100 years service life, the SS 137010 has criteria for 40, 80 and 120 years service life. DS 411 /11/ assumes a 50 years design life. The difference between the different target service lives are partly represented only by variations in required minimum concrete cover and maximum crack width, with the exception of SS 137010 /9/ where also the water/cement ratio is included in cover requirement in relation to service life. However, it is in addition assumed by all these standards that regular inspection and maintenance is performed - but without being precise in defining details, timing and frequency of this maintenance.

The minimum requirements for cover thickness and maximum calculated characteristic crack width in the existing national standards for design of concrete structures Norway (NO),

Sweden (SE) and Denmark (DK), respectively, are listed in *Table 2-1*. It should be emphasized that none of these standards introduce use of alternative reinforcement, e.g. stainless steel. If SSR is used, both crack width requirements, cover requirements and maybe concrete quality (permeability) could be relaxed.

A number of large public or semi-public owners or concessionaires have developed their own design requirements as supplements to the general codes and standards. This is typically Road and Rail Authorities and developers.

The Norwegian and Danish Design Rules for Roads and Bridges assume 100 years service life for the main structural members. The Danish rules require in addition that the first 25 years shall be without repair works. The Swedish Bridge Design Code /12/ provides concrete covers for assumed services lives of 40, 80 or 120 years for given exposure classes and water/cement ratios. It could be mentioned that the British code for bridges, BS 5400 /13/, claims to provide 120 years service life.

Table 2-1: Water/binder-ratio, minimum concrete cover thickness and maximum calculated characteristic crack width for structures in aggressive environment (Chloride from seawater (XS1, XS2, XS3) and de-icing salts (XD3) according to /8/)

Exposure classes	Codes	Water/binder ratio	Min. concrete cover (mm)[1]			Max crack width (mm)
			Design life (DL) 20 years	Design life (DL) 50 years	Design life (DL) 100 years	
Moderate (XS1)	**NS 3473**	0.45	-	40	50	0.3
	SS 137010	0.45	15	25	30	0.3 (DL 50 years)
		0.40	15	20	25	0.3 (DL 50 years)
	DS 411	0.45	-	20	-	0.4
Aggressive (XS2)	**NS 3473**	0.40	-	40	50	0,4
	SS 137010	0.45	30	40	50	
		0.40	25	35	45	0.3 (DL 50 years)
		0.35	25	30	40	0.3 (DL 50 years)
	DS 411	0.45	-	30	-	0.3
Extra Aggressive (XS3/XD3)	**NS 3473**	0.40	-	50	60	0.3
	SS 137010 [2]	0.40	25	35	45	0.2 (DL 50 years)
		0.35	25	30	40	0.2 (DL 50 years)
	DS 411	0.45	-	40	-	0.2

1) Tolerance on cover: DK +5mm, NO +10 mm, SE +10 mm.

2) Only valid for chloride concentration in sea water lower than 0.4% (East coast). With higher concentrations the cover is to be decided in the individual case.

The Road authorities in the three Nordic countries have, however, tightened the requirements to cover thickness and crack widths in order to increase the reliability of the 100 years service life and reduce future maintenance and repair cost. In Norway where the climate and environment is rather hash the cover requirements for tremie concrete casting for bridge construction as well as in the tidal zone is 100 mm (tolerances +20 mm) and 60 mm (tolerances +15mm) for the structure above the splash zone as well as in zones exposed to de-icing salts. The concrete quality in the splash zone and in the coastal areas requires a water-binder ratio of 0.38. In Sweden and Denmark the requirements are somewhat lower.The Swedish Bridge Code /12/ allows a reduction in concrete covers for structures in marine environment in West cost, if stainless steel reinforcement is used. Further, it is required that the stainless steel reinforcement is in accordance with BS 6744 /14/ number 1.4436 and 1.4462 with a proof stress of 500 MPa

3 Classification and documentation of stainless steel

3.1 Definition

Stainless steels are chromium containing steel alloys. The minimum chromium content of the standardized stainless steels is 10.5%. Steel with lower chromium content should not be termed "stainless". Chromium is the main alloy which provides the steel with improved corrosion resistance. This improved corrosion resistance can be seen in *Figure 3-1 /15/.*
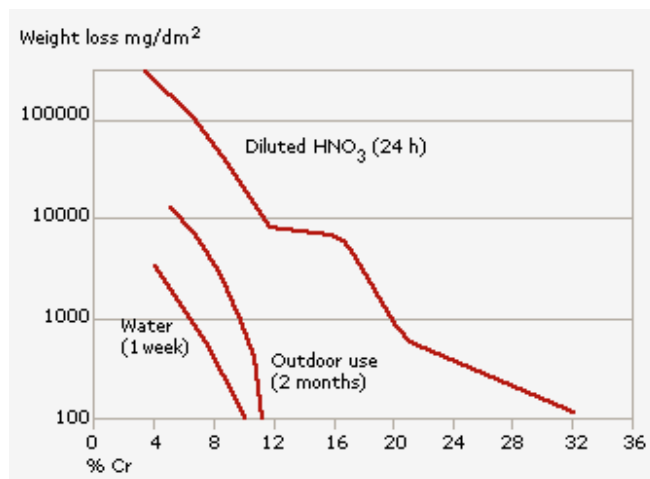
Figure 3-1:   Influence of chromium on the corrosion resistance of stainless steel/15/

The improved corrosion resistance is due to a thin chromium oxide film that is formed on the steel surface and creates a so-called passive condition. It is important to realise that oxygen is required for the oxide film to form. The passivity is a dynamic process which is influenced by the surrounding environment, and especially temperature and humidity. The extremely thin chromium oxide film is also self-repairing under the right conditions, which includes presence of oxygen /16/, /17/.

Besides chromium, typical alloying elements are molybdenum, nickel and nitrogen. Nickel is mostly alloyed to improve the formability and ductility of stainless steel. Alloying these elements brings out different crystal structures to enable different properties of the steel for machining, forming, welding etc. /5/, /18/.

The four major types of stainless steel are:
- Martensitic
- Ferritic
- Austenitic
- Austenitic-Ferritic (Duplex)

Martensitic steel is not of interest as reinforcement.

Ferritic stainless steel has properties similar to mild steel but with the better corrosion resistance. The most common of these steels are 12% and 17% chromium containing steels, with 12% used mostly in structural applications and 17% in housewares, boilers, washing machines and indoor architecture. Currently such steels are rated in the lower range of corrosion resistance for reinforcement.

Austenitic stainless steel is the most widely used type of stainless steel /19/. It has a nickel content of at least 7%, which makes the steel structure fully austenitic and gives it ductility, a large scale of service temperature, non-magnetic properties and good weldability. The range of applications of austenitic stainless steel includes house wares, containers, industrial piping and vessels, architectural facades and constructional structures. Currently such steels are rated in the higher range of corrosion resistance for reinforcement.

Austenitic-Ferritic (Duplex) stainless steel has a combined ferritic and austenitic lattice structure - hence the common name: duplex stainless steel. This steel has some nickel content for a partially austenitic lattice structure. The duplex structure delivers both strength and ductility. Duplex steels are mostly used in petrochemical-, paper-, pulp- and shipbuilding industries. Changing price levels of some of the key alloying elements may at times result in duplex steels being cost effective compared to austenitic steels. In such cases the duplex steels become attractive as reinforcement in the concrete construction industry. Also in highly chloride based corrosive environments at the high temperature range the duplex steel reinforcement is attractive. Currently such steels are rated in the very high range of corrosion resistance.

The growing recognition of the adverse societal impact of needed repair of prematurely corrosion damaged concrete structures, together with the growing environmental impact of such repairs and the growing recognition of the adverse effects of user inconvenience due to repair works, have all together led to a rapid escalation of the use of stainless steel reinforcement to solve the corrosion problem at the origin, /5/ /20/.

This awareness, together with the high costs of the traditional stainless steels - originally developed to solve corrosion problem in other areas that in reinforced concrete structures - have during the past few years made the stainless steel producers to search for methods of producing a robust stainless steel which does not suffer so much from price volatility of the key alloying components. This has in particular led to new products with much reduced nickel content and possibly also low molybdenum content, so-called low-nickel duplex types (**Lean Duplex**), but with sufficient nickel content to maintain the highly corrosion resistant austenitic-ferritic crystal structure. Though none of these products are in large scale running production yet (2006) they seem to have very interesting both corrosion resistance and mechanical properties /21/.

Lean Duplex stainless steels are low alloyed steels but with a duplex crystal structure and with an alloy composition providing PREN values (see section 4.1.6) between the values of 1.4436 and 1.4462. LDX 2101 is produced by Outokumpu /15/ and based on their tests the corrosion resistance of LDX 2101 lies between the resistance of the well known stainless steels 1.4301 (304) and 1.4436 (316). LDX 2101 has a very high Chromium content of 21.5% but a very low Nickel content of 1.5% and a low Molybdenum content of 0.3%, which contributes to the economic competitiveness of the steel. Low-Ni Duplex (1.4362) /22/ has a similar high Chromium content of 23% and the same low Nickel content of 4%, but a Molybdenum content of 0.2%. 1.4362 has a PREN value of 24.

3.2 Classification and chemical composition of stainless steel

Traditionally stainless steels have been classified according to one of the following systems, /5/ /18/:

- The American Iron and Steel Institute (AISI) in which ferritic and martensitic steels are classified, as 400 series alloys i.e. 403 would represent ferritic steel. The austenitic steels are classified as 300 series alloy i.e. 304 or 316. Other than identifying the generic group type these steel grades provided no other information regarding chemical composition or physical and mechanical properties. Traditionally UK standards, such as BS 9705 /23/ and BS 6744 /13/ etc. have followed the AISI classification with the

addition of "S" sub-grades such as Grade 316S33. However, UK standards are being replaced with European standards and those relevant to stainless steel will adopt the current European classification for steels discussed below.

- The German or DIN classification based on the concept of a material number such as 1.44xx.

- The French classification based on a unique material number for a given steel i.e. X18Cr8Ni3Mo would be an austenitic stainless steel with a nominal alloy composition of 18% chromium, 8% nickel and 3% molybdenum. Although a somewhat cumbersome designation this classification has the advantage of providing nominal compositions for each type of steel.

In 1995 a new European standard EN 10088-1 /24/ was issued that provided a uniform method of classification for stainless steels. In effect the standard adopted both the German and French systems. Thus every stainless steel now has a generic number that identifies its grouping and an individual material number referred to as its name giving the nominal alloy composition.

The designation system can be understood for the following example of a stainless steel classified as:

- Material number: 1.4436

- Material name: X3CrNiMo 17-13-3

The material number has the following components:

- 1 Denotes a steel

- 44 Denotes one group of stainless steels

- 36 The individual material identification

The material name has the following components:

- x Denotes a high alloy steel

- 3 Represents 100 times the carbon content (in this case 0.03%), CrNiMo chemical symbols of the main alloying elements 17-13-3 represents the nominal percentage of the main alloying elements.

Additional chemical symbols, for example N for Nitrogen, represent minor but significant alloying elements. The influence of Nitrogen on the corrosion resistance has not been included in the material name though contributing to the PREN value (PREN value is defined in Chapter 4.1.6).

This designation system appears to be more cumbersome than the AISI one it is intended to replace. However, it is more logical and provides an understanding of the alloy composition and therefore material type within the classification.

The number of types of stainless steels is very large, but only a limited number are considered generally relevant as reinforcement for concrete structures. Therefore, this Design Guide has limited the number to the most typical types represented by characteristic groups. These types form the basis for this Design Guide in order to maintain an operational overview as seen form a designer's point of view. Variants of the selected types may very well be relevant in individual cases, and the basic compositions and corrosion characteristics described in the Guide may serve as an indication of the corresponding applicability of the variants.

The new European standards are currently being implemented and the use of this new classification will take over from the more traditional method. *Table 3-1* provides a comparison of the old and new methods of classification for common stainless steel grades and the corresponding pitting resistant equivalent number.

Table 3-1: Classification of stainless steel according to international standards, and corresponding PREN values.

Steel Type	Steel grade		USA	Great Britain	Sweden	PREN-value
	EN 10088-1	Designation	AISI	BS	SS	
Austenitic	1.4301	X5CrNi 18-10	304	314S11/314S15	2332	19
Austenitic	1.4401	X5CrNiMo 17-12-2	316	316S33	2347	25
	1.4429	X2CrNiMoN 17-13-3	316LN	316S63	2375	26
	1.4436	X5CrNiMo 17-12-2	316		2343	26
	1.4571	X6CrNiMoTi 17-12-2	316Ti		2350	25
Ferritic-austenitic (lean duplex types)	1.41xx (LDX 2101)	X3CrNiMo 22-2-0	-	-	-	26
	1.4362	X2CrNiMo 23-4	-	-	-	24
Ferritic-austenitic (Duplex)	1.4462	X2CrNiMoN 22-5-3	-	318	2377	36

EN grades are given steel numbers in groups:

1.40xx for grades with < 2,5% Ni, without Mo, without special additions;

1.41xx for grades with < 2,5% Ni, with Mo, without special additions;

1.43xx for grades with ≥ 2,5% Ni, without Mo, without special additions;

1.44xx for grades with ≥ 2,5% Ni, with Mo, without special additions;

1.45xx and 1.46xx for grades with special additions, such as Ti, Nb or Cu

It is evident, that increasing the level of alloying elements, especially chromium, nickel and molybdenum, will increase the corrosion resistance. However changing the balance of the alloying elements will influence the structure as well as the other properties. Therefore members of the stainless steel family are usually combined in groups having the same metallographic structure. The chemical compositions of the stainless steel grades given in *Table 3-1* are listed in *Table 3-2*.

Summarizing, *Table 3-1* and *Table 3-2* show the composition of a range of stainless steels, which are available in a product form for use as reinforcement. The materials are arranged with increasing corrosion resistance represented by the PREN values in the tables.

Increasing alloy content, particularly Cr, Ni and Mo will increase the corrosion resistance and results in increased cost of materials. The relative cost of these critical alloying elements varies depending on the fluctuating market prices.

Table 3-2: Chemical composition of stainless steel of relevance as reinforcement

Type	Steel grade		Chemical composition									
	EN 10088-1	Designation	C max	Si max	Mn max	P max	S max	Cr min/ max	Ni min/ max	Mo min/ max	N min/ max	Ti min/ max
Austenitic	1.4301	X5CrNi 18-10	0.07	1.0	2.0	0.045	0.03	17.5/ 19.5	8.0/ 10.50	-	max 0.11	-
Austenitic	1.4401	X5CrNiMo 17-12-2	0.07	1.0	2.0	0.045	0.03	16.5/ 18.5	10.0/ 13.0	2.00/ 2.50	max 0.11	-
	1.4429	X2CrNiMoN 17-13-3	0.03	1.0	2.0	0.045	0.015	16,5/ 18,5	11.0/ 14.0	2.5/ 3.0	0.12/ 0.22	-
	1.4436	X5CrNiMo 17-12-2	0.05	1.0	2.0	0.045	0.03	16.5/ 18.5	10.5/ 13.0	2.5/ 3.0	max 0.11	-
	1.4571	X6CrNiMoTi 17-12-2	0.08	1.0	2.0	0.045	0.03	16,5/ 18,5	10.5/ 13.5	2.0/ 2.5	-	5xC/ 0.7
Ferritic-Austenitic	1.41xx (=LDX 2101)	X3CrNiMo 22-2-0	0.03	0.4	5.0	-	-	21.5	1.5	0.3	max 0.22	-
	1.4362	X2CrNiMo 23-4	0.03	1.0	2.0	0.035	0.015	22,0/ 24,0	3.5/ 5.5	0.1/ 0.6	0.05/ 0.20	-
Ferritic-austenitic (Duplex)	1.4462	X2CrNiMoN 22-5-3	0.03	1.0	2.0	0.035	0.015	21,0/ 23,0	4.5/ 6.5	2.5/ 3.5	0.10/ 0.22	-

3.3 Documentation and application of stainless steel reinforcement

In general most of the stainless steels used for reinforcement are within the types 1.4301 and 1.4436. Only in extreme corrosive environments like de-icing salts or marine environments and high temperatures more resistant materials are considered like 1.41xx and 1.4462.

The specifications as listed in *Table 3-3* represent an overview of available national and European standards for the characterization of currently available equalities of SSR. These National Standards specifies requirements and describes methods of test. National certification bodies, certifies producers of SSR according to the national product standards.

Table 3-3: Current Specifications relevant for the documentation and application of SSR

Swedish Standards	
ID. No.	**Title**
SS 14 21 69	Reinforcement steel – SS steel 2169-13
SS 21 25 16	Reinforcement steel bar – Cold drawn and profiled bar Ps 700
SS 14 23 40	Stainless steel reinforcement – steel 2340

Danish Standards	
ID. No.	**Title**
DS 13080	Steel for reinforcement – Steel reinforcement used without pre-stressing in concrete structures

Table 3-3: (cont…)

Norwegian Standards	
ID. No.	*Title*
NS 3576-2	Steel for reinforcement of concrete – Dimensions and properties – Part 2: Ribbed bars B500NB
NS 3576-3	Steel for reinforcement of concrete – Dimensions and properties – Part 2: Ribbed bars B500NC

Relevant European Standards	
ID. No.	*Title*
BS 6744	Stainless steels bars for reinforcement of and use in concrete – Requirements and test methods
BS 8666	Specification for Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete
EN 10204	Metallic products - Types of inspection documents
EN 10088-1	Stainless steels - Part 1: List of stainless steels
EN 10088-3	Stainless steels - Part 3: Technical delivery conditions for semi-finished products, bars, rods and sections for general purposes

The national standards do not set up the same specifications for the mechanical properties of the SSR. In general all the standards operate with ductility classes and for SSR Class B &C are relevant. Extracts from some codes are given below:

Norwegian standard:

"NS 3576-2, ductility class B: Table 5 – Ductility requirements"

A_{gt}	R_m/R_{eH}	
Characteristic value	Characteristic value	Single values
5,0%	1,08	1,05

"NS 3576-3, ductility class C: Table 5 – Ductility requirements"

A_{gt}	R_m/R_{eH}	
Characteristic value	Characteristic value	Single values
8,0%	1,15	1,10

Danish standard:

"DS 13080:2001: Table 2- Ductility classes"

Ductility class	A_j		R_m/R_{eH}	
	Characteristic values	Minimum value	Characteristic value	Minimum value
A	•3,0%	2,5%	•1,08	1,05
B	•8,0%	7,5%	•1,08	1,05

3.3.1 Availability of grades, bar sizes and bar lengths

The stainless steel listed in *Table 4-1*are those available for use in the Scandinavian countries. The stainless steel can be delivered in the same bar diameters as carbon steel reinforcement. The standard stock bar length is 12 meter as for carbon steel. In addition stainless steel welded mesh is available /18/.

It is possible to order stainless steel in coils and produce other bar lengths on site. If reinforcement steel from coil is straightened on site, it must be done by a producer with 3rd party approval. Furthermore, the mechanical properties might change and tests have to be carried out according the specified standard.

3.3.2 Fabrication

Austenitic and duplex stainless steels can be bent to shape using the methods commonly used for carbon steel, providing that allowance is made in the loading rating of the equipment used, as more force is required to bend stainless steel than carbon steel because of the increase in its strength when it is being worked (strain hardening) /18/.

Stainless steel bars should be cut and bent in accordance with requirements of BS 8666 /25/. Stainless steel reinforcement tends to have more spring than carbon steel, which needs to be taken into account especially when bending small dimension links.

Cutting and bending of stainless steel requires 3rd party approval to ensure the minimum bending formers are used.

3.3.3 Costs

The often-stated barrier to use stainless steel reinforcement is the high initial cost. Over the past few years, the price of stainless steel reinforcement has reduced and further reductions are expected.

In comparison with the unit price of carbon steel, the stainless steel bar is about six to ten times higher, depending on bar size and steel type, based on the currently rather high price level (2006). However, the cost of cutting and bending, transportation and fixing stainless steel reinforcement remains the same as for carbon steel.

The price level of the two types of *Lean duplex* steels is on similar relatively low level compared to the classic stainless steels. With a price level 25-35% below the price for the types 1.4436 and 1.4462, and apparently a more stable price structure these steels seem very interesting provided they provide the high chloride corrosion resistance in concrete as indicated by the PREN values. According to factory information 1.4362 is currently (2006) in running production as plates, bars and steel angels, and is expected to be available as ribbed reinforcing bars ultimo 2006 primo 2007.

In /26/ the costs of replacing some of the carbon reinforcement with SSR has been investigated for three real marine constructions made during the years 1995-1996. Although the material cost of SSR in this case was 5 times the cost of ordinary reinforcement the effect on the total construction costs of introducing SSR in the most critical zones of the turned out to be marginal (by replacing 10 % of carbon steel with SSR the initial construction costs increased by 1- 2 %) .

Such experience will have a very positive economic effect if a life cycle cost (LCC) optimization is performed. The increase in initial costs arising from the use of SSR should be offset against possible reduction in future maintenance costs, both scheduled and unscheduled. Examples of are given in /5/ and Chapter 7.3.

4   Corrosion properties of stainless steel reinforcement

4.1   Corrosion types

The passive film ensuring the corrosion resistance of stainless steel can be broken down completely or partly with corrosion as a result. However, the passive layer has the ability of repassivation in an environment containing oxygen, such as in air or in aerated solutions. This environment is sufficient for both the creation and the maintenance of the passive layer of stainless steels. There are, nevertheless, environments that cause permanent breakdown of the passive layer. This could be with high chloride concentrations above a so-called chloride corrosion threshold value or in oxygen-free environments like permanently water saturated conditions. Seawater in the tidal and splash zones will usually have adequate oxygen content to maintain the passive film on built-in stainless steel reinforcement. Under circumstances where the passive layer cannot be rebuilt, corrosion occurs on the unprotected surface /16/, /27/.

The types of corrosion, which in general can occur on stainless steels, are the following:

- Uniform Corrosion
- Galvanic Corrosion
- Pitting Corrosion
- Stress Corrosion Cracking
- Crevice Corrosion
- Atmospheric Corrosion
- Intergranular Corrosion
- Corrosion Fatigue

From those above mentioned, uniform corrosion, galvanic corrosion, pitting corrosion, crevice corrosion and stress corrosion cracking could in principle be expected for stainless steel reinforcement in concrete. Atmospheric corrosion, intergranular corrosion and corrosion fatigue are not generally relevant for concrete structures and therefore not mentioned further.

4.1.1   Uniform corrosion

Uniform corrosion occurs when the passive layer on a stainless steel surface partly or completely breaks down.

The corrosion then propagates at a rate determined by a combination of the corrosive environment and the alloy composition. Uniform corrosion or general corrosion occurs on stainless steel in acid environments or hot alkaline solutions. Severe environments from a corrosive point of view are high concentrations of hydrochloric or hydrofluoric acid in which the corrosion may propagate at a rate that can be detrimental to a construction.

4.1.2   Galvanic corrosion

When two dissimilar metals are connected electrically and immersed in a conductive liquid, an electrolyte, their corrosion performance might differ significantly when compared with the metals, uncoupled. As a rule, the less noble material, the anode, is attacked, whilst the more noble metal, the cathode, is essentially protected from corrosion. This phenomenon is called galvanic corrosion.

If the passivity of one of the steel grades breaks down and corrosion occurs, the corrosion rate increases further by the galvanic coupling. In addition the galvanic series are dependent on temperature and the composition of the conductive solution. Knowledge about the series in the specific environment is necessary to predict galvanic corrosion.

4.1.3 Pitting corrosion

Pitting is a form of localized corrosion and is characterized by attacks at small discrete spots on the steel surface. Pitting occurs mainly in the presence of neutral or acidic solutions containing chlorides or other halides. Chloride ions facilitate a local breakdown of the passive layer, especially if there are imperfections in the metal surface.

Initiation sites may be non-metallic inclusions, e.g. sulphides, micro crevices caused by coarse grinding, or deposits formed by slag, suspended solids, etc. When the metal corrodes in the pit, dissolved metal ions generate an environment with low pH and chloride ions migrate into the pit to balance the positive charge of the metal ions.

A higher chromium, molybdenum and nitrogen content in the steel increases the resistance to pitting. The relative resistance to pitting is often measured as a critical pitting corrosion temperature, CPT, in a selected media.

4.1.4 Stress corrosion cracking

A material failure may be accelerated by the combined effect of a corrosion process and a mechanical stress. Two examples of such processes are stress-corrosion cracking and corrosion fatigue. The most common type is trans-granular stress-corrosion cracking, SCC, that may develop in concentrated chloride-containing environments. Previously, it was generally considered that an elevated temperature was necessary for SCC to occur.

In recent years, however, SCC has been experienced at ambient temperature on standard grade steels like 304(L), which corresponds to 1.4306 and 1.4307, or 316(L), which corresponds to 1.4404, 1.4432 and 1.4435, that were exposed to high tensile stresses. In these cases the steel surface was contaminated with solid salt deposits and the humidity of the atmosphere was rather high. These two factors resulted in a thin liquid film saturated with chloride. Other contaminants, such as H_2S, may increase the risk of SCC in chloride containing environments.

4.1.5 Crevice corrosion

Crevice corrosion is a form of localized corrosion and occurs under the same conditions as pitting, i.e. in neutral or acidic chloride solutions. However, attack starts more easily in a narrow crevice than on an unshielded surface. Crevices, such as those found at flange joints or at threaded connections, are thus often the most critical sites for corrosion.

In narrow crevices, capillary forces make liquid penetrate into the crevice. Oxygen and other oxidants are consumed for the maintenance of the passive layer in the crevice just as on the unshielded surface. However, in the stagnant solution inside the crevice, the supply of new oxidant is restricted, causing a weakened passive layer, hence an anodic site.

Small amounts of dissolved metal ions inside the crevice cause a decrease of the solution pH and the presence of chlorides facilitates the break-down of the passive layer. Thus the environment inside the crevice gradually becomes more aggressive and repassivation becomes less likely. As a result, crevice corrosion attacks often propagate at a high rate, thereby causing corrosion failure in a short time.

4.1.6 Corrosion resistance

Austenitic steels are more or less resistant to general corrosion, crevice corrosion and pitting, depending on the quantity of alloying elements. Resistance to pitting and crevice corrosion is very important if the steel is to be used in chloride-containing environments not protected by a high pH-value. Resistance to pitting and crevice corrosion increases with increasing contents of chromium, molybdenum and nitrogen /28/.

The austenitic- ferritic steels (duplex) are quite superior to the common austenitic steels in respect to corrosion resistance and especially to stress corrosion cracking. Today's modern steels with correctly balanced compositions, for example Duplex 1.4462, also possess good pitting resistance properties and are not sensitive to intergranular corrosion after welding.

Whereas chromium is the main alloying element, molybden and nitrogen has more effect on the localized corrosion resistance. In order to compare stainless steel grades with different alloying, correlation of the influence of the different elements has been made resulting in the expression of pitting resistance equivalent number (**PREN**). This expression can be considered as a relative measure of the total resistance resources for the steel grade and thus as a comparable value for ranking the corrosion resistance against chloride pitting corrosion. The expression is calculated from the content of the alloying elements in the steel grade.

For austenitic steels the expression is:

PREN = %Chromium + 3,3 * %Molybdenum + 16 *%Nitrogen

For duplex steels the effect of nitrogen is considered higher resulting in the expression

PREN = %Chromium + 3,3 *%Molybdenum + 30 *%Nitrogen

The susceptibility to pitting corrosion increases with the decrease in PREN value. Examples of commercially available standard grades, which can be used in concrete, and their pitting resistance equivalent number, are shown in *Table 3-1*.

The PREN values have been developed to represent the level of corrosion resistance of different grades of stainless steel to be used directly exposed to a corrosive environment. Therefore the values cannot be directly transferred to represent the absolute pitting corrosion resistance of SSR cast into alkaline concrete. But the values represent the best available relative measure of the pitting corrosion resistance of the SSR exposed to chlorides in concrete /18/ /28/.

4.2 Resistance to chloride attack and carbonation

The corrosion resistance required for use in concrete is primarily resistance against localized corrosion in chloride containing media. This resistance depends on the main alloying elements of chromium, nickel, molybden and nitrogen. The onset of corrosion is dependent on the critical chloride concentration at the level of the reinforcement triggering corrosion by eliminating the passive layer locally. This so-called threshold value for chloride corrosion initiation depends on the degree of alloying of the steel, the level of alkalinity of the surrounding concrete and the level of the ambient temperature /29/.

Reduced alkalinity may be a result of carbonation of concrete, where the carbonated concrete normally has a pH of about 9. In this connection it should be mentioned that "Blended" cements may cause reduced alkalinity of the concrete, hence lower pH value and thereby buffer of hydroxide in the pore solution against carbonation. Leakage will also reduce the pH-value of concrete.

At the UK Building Research Establishment tests (BRE) /30/, /31/, /32/ studies have been performed to assess the risks of corrosion of SSR in chloride contaminated concrete. A range

of the most typical types of SSR were examined by casting them into concrete prisms with varying levels of cast-in chloride concentrations and immersed in seawater.

The conclusion of these tests was that austenitic SSR were virtually immune to corrosion attack. A further interesting observation from the seawater immersion was the limited level of pitting attack observed on the 316-type SSR (1.4436 and 1.4429) in the unfavourable condition of having bare steel projecting from the concrete. In these tests, the corrosive attack of the exposed areas was non-existing or superficial, even after 22 years in marine conditions. All the austenitic steels tested showed very high corrosion resistance with no serious corrosion of any bars, but recommendations were made that the molybdenum bearing alloys (1.4436 (316) and higher 1.4462 (duplex)) should be used in chloride-contaminated conditions to minimise the risk of corrosion, especially where high chloride contents and/or carbonation to the full depth of cover were anticipated.

Similar studies were carried out at the Politecnico di Milano /33/, /34/, /35/ and /36/ on several types of SSR in concrete and in solutions simulating the concrete pore liquid. These tests evaluated the corrosion conditions in concrete exposed to various environments, i.e. different temperatures, chloride levels and alkalinity. Testing temperatures of 20°C and 40°C were used to simulate temperate and hot climates. Equivalent types of steels, 1.4301, 1.4429, 1.4529 and 1.4362 were tested. The duplex steel 1.4362 contains only 0.2% molybdenum compared to the most common type duplex SSR today, 1.4462, containing 2.5% molybdenum. As the contents of molybdenum is expected to have a major influence on the corrosion resistance, the results of the 1.4362 is not included in the guidance presented in Figure 4-1. As SSR grade 1.4436 (316) also was not included in the tests, but the corrosion resistance expected to be between the value of 1.4429 and greater than 1.4301, it has been included in the band covered by the same band as 1.4429 in *Figure 4-1*.

- These test programmes led to factual conclusions regarding the chloride corrosion resistance of the different grades of SSR:

- As the alkalinity increased, chloride induced corrosion decreased

- The critical chloride threshold levels exceeded 10% Cl⁻ by weight of cement in the highly alkaline solution (pH 13.9), at both 20°C and 40°C

- At a pH of 12.75 the steel requires a chloride ion concentration by weight of cement in excess of 4% Cl⁻ by weight of cement to initiate corrosion; an increase in pH to 13 raises the concentration for corrosion initiation to approximately 6%. These chloride concentrations are very high compared to those found in actual structures. It would indicate that for most general applications this grade of steel would be a suitable material.

- Tests in nearly neutral pore water solutions (pH 7.5) showed a lower resistance to chloride induced corrosion, especially when the chromium content was low

- In the case of carbonated concrete in which the pH has fallen to a value in the order of approx. 9 stainless steels will also be of benefit. Under these conditions the stainless steel will unlike carbon steel, remain passive in this condition. Stainless steel will also be of benefit where the concrete is both carbonated and chloride contaminated to the level of the reinforcement. In this case the threshold concentration to corrosion initiation is lower, than in higher pH conditions, but would still be satisfactory provided the appropriate grade of stainless steel is selected.

- By increasing the temperature to 40°C, a general reduction of corrosion resistance was observed, except with the super austenitic steel 1.4529 (this steel not specifically included in this report). The duplex type 1.4462 is expected to have similar good corrosion resistance under these circumstances.

Following the data from these different test regimes but analyzing the same issues, the chloride corrosion resistance of different grades of SSR can be presented schematically in the informative *Figure 4-1* where the curves indicate the conditions of alkalinity and chloride content in which it is considered safe to use the different steels.

The corrosion threshold level is defined as the level at which corrosion of the reinforcement starts; below this level the reinforcement is passive. In this comparison it is essential to note that carbon steel reinforcement is susceptible to localized corrosion at low chloride contents even in alkaline concrete, as illustrated in *Figure 4-1*. This corresponds well with the normal considered threshold value for chloride content for corrosion of black steel in concrete on approximately 0.4 % Cl⁻ by weight of cement.

Further in *Figure 4-1*, the effect of temperature on the threshold values is illustrated based on comparative test results performed at 20°C and 40°C, respectively. The effect of the alkalinity, pH value is pronounced in both situations. Special attention shall be drawn to the considerable effects observed within the range of pH=12.5-13. This relatively small difference in alkalinity of the pore water would represent the normal difference in pH of noncarbonated concrete made with either a highly blended cement (lower pH value) or with an OPC cement (higher pH value).

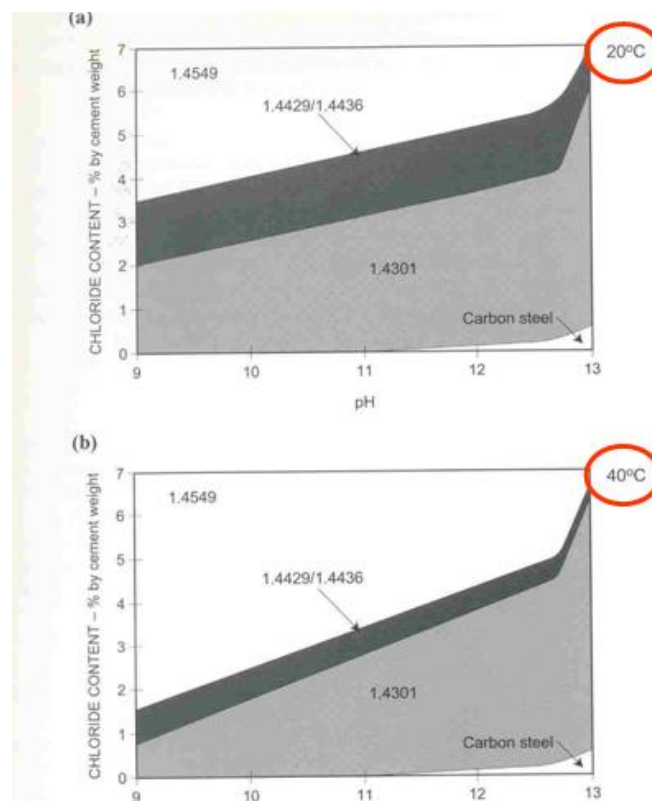*Figure 4-1:   Illustrative representation of the application of different grades of SSR in chloride containing environments. The threshold levels are indicative only, as local conditions may increase as well as reduce the indicated values. The effect of temperature on the threshold values is illustrated from the presentation of comparative test results performed at 20°C and 40°C, respectively /36/*

A long-term site exposure performed in the onerous climate conditions in the Arabian Gulf using SSR is interesting in the sense that this environment is considered the most aggressive in all aspects regarding chloride induced reinforcement corrosion /37/. The reason is the adverse synergetic effect of the high ground and ambient salinity, the high average temperature, the high humidity level and even the high aridity. Austenitic steel type 316 after natural exposure for 12 years in concrete with low cement content of only 220 kg/m^3, poorly cured concrete and with a 3.2% chloride ion content by weight of cement in the original mix, showed no significant loss in weight due to corrosion. This performance was clearly much better than that of the high yield carbon steel reinforcement, which has been known to start corroding after just a couple of years even in high quality concrete with large cover in these conditions.

4.3 Surface finish of stainless steel reinforcement

The surface of stainless steel can vary from matt chemically de-scaled finish to a bright as-rolled or cold drawn finish. The presence of a scale or oxides on SSR should be avoided because it can increase the risk of galvanic corrosion.

In addition, the surface of SSR may be polluted with carbon steel particles when being stored together with carbon steel or handled on equipment also handling carbon steel reinforcement. Such extensive superficial surface pollution should be avoided as there is a small risk that galvanic corrosion may develop. However, this is of very minor importance in practice, as the effect of the corrosion of the carbon steel particles is negligible from a structural point of view, as these particles cannot cause any cracking or damage to the concrete cover, only result in unsightly discoloration of the SSR surfaces when stored in most atmosphere or when exposed through breakouts in the concrete polluted with chlorides.

One unresolved issue at present is the risk of having carbon steel particles pressed deeply into the crystal lattice of the SSR through handling with carbon steel equipment, and then these particles corrodes when critical amounts of chlorides reach the steel surface. For sensitive grades of SSR it could be considered whether this corrosion inside the crystal lattice could initiate pitting or crevice corrosion. This theoretical type of corrosion has not been reported and the issue remains a suggestion for testing.

4.4 Classification of corrosion resistance of stainless steel reinforcements

In practice it may be cumbersome to operate continuously with the many different steel grade designations available as SSR. Only in the detailed design and in the specifications will it be necessary to be precise in the designation of the steel grade required. Also the variable price levels of the different grades may warrant a somewhat more simple corrosion resistant classification until the final quality is adopted.

Therefore, such an operational classification is suggested in *Table 4-1*. A recommendation of the application of the different corrosion resistant classes is related to the corrosivity of the operating environment of the structure and its assumed longevity /28/, /38/, /39/ .

When adopting the corrosion resistant classes it shall be taken into account that different steps of the processing and handling of the steel, such as cold working, heat treatment, welding and bending, may have adverse influence on the corrosion resistance.

It may be noted, that the *Lean duplex* types of SSR, are not included in *Table 4-1*. This shall not be interpreted as a de-classification of their corrosion resistance in structural application. They have been left out because these grades of reinforcement are not yet in running production (2006)

Table 4-1: Classification of corrosion resistance of available ribbed SSR (2006)

Corrosion resistance class	Steel Type	Steel grade		PREN-value
		EN 10088-1	Designation	
Class 0	Carbon steel	(-)	(-)	(-)
Class 1	Austenitic (without Mo)	1.4301	X5CrNi 18-10	19
		1.4541	X6CrNiTi 18-10	17
Class 2	Austenitic (with Mo)	1.4401	X5CrNiMo 17-12-2	25
		1.4429	X2CrNiMoN 17-13-3	26
		1.4436	X5CrNiMo 17-12-2	26
		1.4571	X6CrNiMoTi 17-12-2	25
Class 3	Ferritic-austenitic (Duplex)	1.4462	X2CrNiMoN 22-5-3	36

There are no hesitations regarding adequate corrosion resistance. If independent testing confirmed the producers' own extensive testing, then such Lead duplex types could become part of the recommended types of SSR in the future.

4.5 Resistance to galvanic corrosion

When stainless steel is cast into concrete, however, the cathodic reaction is a very slow process, since no catalytic activity takes place on a stainless steel surface. A research project conducted at the FORCE Technology /40/ has indicated that the cathodic reaction is inhibited on stainless steel embedded in concrete, as compared to the cathodic reaction on carbon steel reinforcement in galvanic contact with corroding carbon steel /41/. Publications of Pedeferri et. al, /36/ and Jägi et. al. /42/ provides also results, which confirmed the above mentioned findings.

The fact that stainless steel is a far less effective cathode in concrete than carbon steel, makes stainless steel a useful reinforcement material for application in repair projects. When part of the corroded reinforcement, e.g. close to the concrete cover, is to be replaced, it could be advantageous to use stainless steel instead of carbon steel. In being a poor cathode, the stainless steel would minimize any possible problems that may occur in neighbouring corroding and passive areas after repair. The experimental results *(Figure 4-2)* where the carbon steel (initially passive and later corroding) was connected to stainless steel, has confirmed this behaviour.
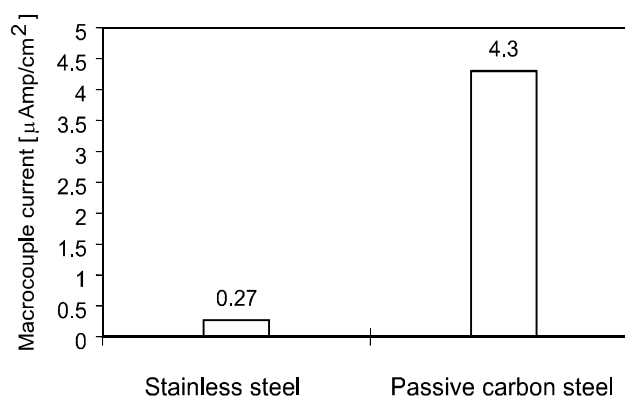

Figure 4-2: Macro-couple current for carbon steel starting to corrode coupled to either stainless steel or passive carbon /41/

Stainless steel freely exposed to seawater may, if in galvanic contact with less noble metal such as carbon steel (also exposed to sea water), initiate a rapid galvanic type of corrosion of the less noble metal. The otherwise slow cathodic oxygen reduction at the stainless steel surface is catalyzed by a bacterial slime, which forms after a few weeks in seawater.

When the current is measured between the carbon steel rebar starting to corrode and a rebar of carbon steel that is still passive, a current density value of approximately 4.3 µA /cm^2 is recorded. If the same corroding carbon steel rebar is connected to the stainless steel with the surface area equal to the passive carbon steel, the measured current density value is reduced to only 0.27 µA/cm^2, se *Figure 4-3*. This means a reduction in current density by a factor of approximately 15, which will result in the same decrease in corrosion rate. In addition where the carbon steel is placed outside the most aggressive exposure conditions the potential difference between the carbon steel and SSR will be relatively small and so will the corrosion rate. On top of this is the inefficient cathodic action of the SSR as described above.

Therefore, it is very important for the intelligent use of stainless steel that it be combined with carbon steel in proportions that guarantee both an optimal performance and cost-effective solution.

It seems to be a fact that most of civil engineers have an unfounded fear of using stainless steel and carbon steel together in the same concrete structure. In Denmark, FORCE Technology (The former Danish Corrosion Centre) has given advice to more than 100 clients on the use of stainless steel in concrete. Nearly always the clients had to be convinced, that it is in the fact good and safe practice to use stainless steel in the most chloride exposed concrete, with the stainless steel in good - often welded - connection with the carbon steel as the main reinforcement. On *Figure 4-3* and *Figure 4-4* examples of the application in practice of combined carbon steel and SSR are shown.

A significant macro couple corrosion threatening the SSR can only arise under very particular conditions. For example, when the SSR in concrete is in heavy chloride contaminated conditions and is deprived from the access of oxygen, i.e. in water saturated condition, and the carbon steel is in aerated condition and in non-chloride contaminated condition. The risk of pitting corrosion of the SSR increases to the level typical of the aerated condition for carbon steel.

Figure 4-3: Combination of stainless steel reinforcement and ordinary black steel reinforcement. Only the deck part later to be exposed to a marine environment is reinforced with SSR. The remaining part of the deck, later to become part of the indoor structure, is reinforced with ordinary carbon steel reinforcement. No special precautions are needed where the two types of steel meet.

Figure 4-4:   Bridge column in marine environment reinforced with two or more layers of reinforcement. The outer layer is SSR and the subsequent layers are ordinary carbon steel reinforcement.

Stainless steel is also an excellent material to be used for all components, which are only partially embedded in concrete, especially connected to the reinforcement. Examples are blots, binders, ladder rungs, inserts, electrical connectors, sanitary piping and bushings.

4.6 Corrosion resistance of welded stainless steel reinforcement

In the presence of chlorides the corrosion resistance of stainless steel in concrete can be adversely influenced in the region of the weld and the heat affected zone /43/. This is because welding results in the formation of high temperature oxides on the surface of the steel, often referred to as heat tint, or welding scale, and these oxides do not remain as stable (passive) as the oxide layers on the bare stainless steel when exposed to chloride environments.

An investigation of the effect of resistance on welding on the corrosion resistance of carbon steel and stainless steel reinforcement types 1.4301 and 1.4436 has been conducted /29/. The effect of ingress of chlorides as well as cast-in chlorides was investigated. The studies showed that the stainless steel reduced the chloride threshold level from 10, in the not-welded case, to three to six times that of the carbon steel as welded, due to the combined effect of oxide and insufficient compaction of concrete around the weld

The corrosion resistance can be reinstated by the complete removal of all heat tint scale after welding. This is not easily done under conditions prevailing on construction sites. Primarily because the heat tint scales are very adherent and difficult to remove, in practice the only methods that can guarantee removal are either abrasive blast cleaning or the use of pickling pastes both of which are difficult to carry out on site. However, welding in factory conditions, where welding condition can be closely controlled, can be carried out successfully.

Where bars need to be joined alternative methods of connection, such as lapping or mechanical couplers, should be used. If welding is unavoidable then a post cleaning process should form part of the welding procedure qualification. The quality procedures should also include accelerated testing to demonstrate that the cleaning process reinstates the corrosion resistance of the stainless steel surface.

5 Mechanical and physical properties of stainless steel reinforcement

5.1 Stress-strain relationships

The stress-strain relationship for different types of stainless steels is illustrated in *Figure 5-1 /15/*. Note that only the austenitic and the ferritic-austenitic (duplex) steel are relevant as reinforcement. Austenitic and duplex (and ferritic) grades of steels show early plastic deformation in test, and continue to sustain increasing load with increasing strain.

Cold working will increases the strength of the steels and is therefore used to meet the requirements for use as reinforcement in concrete. Cold working usual results in martensite formation in 1.4301 types, whereas in 1.4401/1.4436 and duplex materials, this is not the case. For the austenitic types cold working results in a reduction of the elongation from 40% to 20-25%.

For small dimensions (<16 mm) also warm working (at temperature somewhat lower than normal for such process) may be used for increasing the strength, resulting in mechanical properties similar to those obtained by cold working.

Another way of increasing strength is addition of nitrogen (0.15-0.2%). This is however not sufficient to reach the required strength and must therefore be combined with either cold or warm working.

To characterize the design strength of such strain hardening materials, proof strength are defined and determined as the tensile stress ($R_{P0,2}$) at elongation (strain) 0,2 %. Ultimate tensile strength is defined at the maximum load the tested reinforcement can withstand.

As listed in *Table 5-1*, stainless steels can be produced as ribbed bars within the normal range of strength and deformability required for application in concrete.

The modulus of elasticity (E-modulus) for the relevant SSR is about 200 kN/mm^2, in the same range as for carbon steel reinforcement (210 kN/mm^2).

Owing to their excellent mechanical properties in the as-rolled conditions, duplex steels are of particular interest as material for reinforcement. For example, the duplex steel of grade 1.4462 (X2CrNiMoN 22-5) as cold rolled, has proof strength of 950 MPa, tensile strength of 1059 MPa and elongation of 14 % for 10 mm bars.

Figure 5-1: Stress-strain curves for different stainless steels at room temperature /15/

Table 5-1: Mechanical properties of SSR

	Steel grade	Dimension [mm]	Proof strength $R_{P0,2}$ [N/mm^2]	Tensile strength R_m [N/mm^2]	Elongation		$R_m/$ $R_{P0,2}$	E-modulus at 20°C [kN/mm^2]
					$A_{\alpha t}/$ A_j	A_5		
Cold drawn	1.4301	3-16	≥550	≥600	≥5/ ≥8	≥15	≥1.10	200
	1.4436	3-16	≥550	≥600	≥5/ ≥8	≥15	≥1.10	200
	1.4571	3-16	≥550	≥600	≥5/ ≥8	≥15	≥1.10	200
Hot rolled	1.4301	20-40	≥500/ ≥550	≥700	≥5/ ≥8	≥15	≥1.10	200
	1.4571	20-32	≥500/ ≥550	≥700	≥5/ ≥8	≥15	≥1.10	200
	1.4462	20-50	≥500/ ≥550	≥700	≥5/ ≥8	≥15	≥1.10	200

5.2 Application at extreme temperatures

Carbon steel experience a considerable drop in proof stress at elevated temperatures, particularly at temperatures above say 500 °C. Similarly, they experience a considerable loss in ductility and increased brittleness at temperatures below zero. Below -20 °C most carbon steels behave very brittle and would not be adequate as reinforcement in structures exposed to sudden impact loading or seismic actions.

This situation is very different for SSR. The austenitic stainless steels maintain their strengths at considerable higher temperatures than carbon steel. Therefore such steels are more resistant and robust under fire loading than carbon steel. Similarly, austenitic steels maintain their strength and ductility at very low temperatures, so-called cryogenic temperatures, which may reach as low as -196 °C. In addition their strengths increase slightly under decreasing temperatures.

Although stainless steels are most commonly used for their corrosion resistance stainless steels are often used for their high temperature properties. Stainless steels can be found in applications where high temperature oxidation resistance is necessary and in other applications where high temperature strength is required. The high chromium content which is so beneficial to the wet corrosion resistance of stainless steels is also highly beneficial to their high temperature strength and resistance to scaling at elevated temperatures.

The particular crystalline structure of the austenitic steels allows them to maintain a high degree of toughness (Charpy impact test) with a large temperature range, from elevated temperature to far below the freezing point. Due to the difference in crystalline structure between austenitic steel and duplex steel, (where duplex is in part ferritic) duplex steel undergoes a marked decrease in toughness at low temperature, an effect which starts at about -50°C.

5.3 Fatigue

SSR has fatigue properties similar to that of carbon steel reinforcement when tested in the atmosphere. However, the critical issue of fatigue for reinforcement is the special situation of a fatigue loading in a corrosive environment. In this case the much increased corrosion

resistance of SSR compared to carbon steel reinforcement results in a marked increased corrosion fatigue resistance of SSR compared to carbon steel reinforcement.

The fatigue limit (upper limit of fatigue stress to be supported indefinitely) for SSR, is related to the tensile strength of the steel. In this case the increased strength of several of the types of SSR compared to carbon steel reinforcement leads to a corresponding increased fatigue limit.

5.4 Physical properties

The density of SSR varies only marginally from normal carbon steel reinforcement, as seen in *Table 5-2*, and in all practical applications the small variations cannot be of concern.

SSR has a thermal expansion which for the austenitic steels is approximately 16×10^{-6}, and the austenitic-ferritic duplex steels have a thermal expansion of approximately 13×10^{-6}, compared to the carbon steel with a thermal expansion of 12×10^{-6}, see *Table 5-2*.

Although the difference between the thermal expansion of austenitic and carbon steel is not negligible this issue is not of concern in the structural design. The issue is to compare the thermal coefficient between the steel and the concrete, and there classical concretes have a coefficient close to the value of carbon steel. However, depending of the type and quality of aggregates (with granite and gabbro at the low end, and limestone at the high end regarding thermal expansion) the concrete itself can have a thermal expansion coefficient which may vary say 20%, and the elastic modulus of the concrete may also vary 20-30% or more, depending on the mix. These variations in strain properties have never been reported giving structural or performance related difficulties in concrete structures.

Table 5-2: Physical properties of stainless steel /18/

Steel grade	Density kg/m^3	Thermal expansion [10^{-6}/°C]
Carbon steel	8000	12
1.4301	7900	16
1.4436	8000	16
1.4462	7800	13

In general austenitic type stainless steels are considered to be practically non-magnetic. However, cold drawn bars gain some magnetic permeability. The magnetic permeability decreases in the designation order 1.4301>1.4436>1.4429>1.4529. Therefore SSR required to have a low magnetic permeability (not magnetic) must be hot-rolled and be of a specific compositions or alternatively tested. As an example cold drawn bars of 1.4571 has been tested to a permeability of $\mu = 1,00800$.

Duplex stainless steels are magnetic, as are carbon steels.

6 Designing and constructing with stainless steel reinforcement

6.1 General

When considering the adoption of SSR to eliminate the corrosion risk problems within a stipulated long service life, the additional costs for the SSR obviously becomes a key issue and alternative corrosion preventive measures are frequently tabled. Due to the cost implications and the level of reliability of the alternative solutions, these issues should be addressed at a very early stage of the design process in order for the owner and client to select the final solution being the optimal for him. However, only a profound knowledge of the governing deterioration mechanisms can assist in such evaluations, together with long-term documentation and experience.

A number of corrosion preventive measures, apart from concrete quality, concrete cover and SSR, may be considered when ensuring long service life of concrete structures, such as:

- Surface treatment of concrete

- Corrosion inhibitors

- Cathodic prevention

- Non-metallic reinforcement

- Corrosion resistant steel (not stainless) reinforcement

Some of these methods are discussed in Chapter 6.8.

Life cycle cost analyses have shown that the use of SSR in zones exposed to high chloride concentrations is an economic beneficial solution (/5/ and Chapter 7.3). Following the "Avoidance of deterioration approach"/3/ (see Chapter 2.3.1) this approach can ensure a very long problem-free service life in the structure, provided the concrete itself is made sufficiently resistant to avoid other types of deterioration (sulphate attack, salt scaling etc.).

Selecting SSR of the types listed in *Table 4-1* may in principle lead to a simple replacement of ordinary carbon steel reinforcement with SSR in the ratio 1:1 as the structural properties are the same regarding strength and ductility (or better for several of the available types of SSR). Several other advantages by using SSR may be adopted, as described in Chapter 6.3 and 6.4.

SSR can be combined with black steel cast into concrete without risks of galvanic corrosion due to bi-metal - or galvanic - action. This is an important observation, and is the precondition for general economical application of SSR used only in the parts of the structure where this protection is needed, so-called selective use. If the stainless steel protrudes from the concrete, such as for inserts and fastenings, then this will also be satisfactory as long as the contact to the carbon steel is within the concrete section.

The key design parameters to be addressed in the design with SSR are:

- SSR grade in relation to the prevailing environmental aggressivity

- Concrete section design

    - Concrete cover

    - Concrete mix representing the required quality

    - Design crack widths

- Execution conditions and after-treatment

- Plastic shrinkage cracking

- Thermal cracking

6.2 Selection of stainless steel reinforcement grade

Available ribbed stainless steel reinforcement (2006) is listed in *Table 4-1*. Research in Europe has shown that all grades of austenitic steels will perform well in chloride contaminated concrete exposed to harsh environments and other research has shown that the tolerance of stainless steels to chloride induced corrosion is at least an order of magnitude greater than for carbon steels

In *Table 4-1* an attempt has been made to classify the various available ribbed stainless steel material grades into corrosion resistance classes. Such a classification system should have been based on chloride threshold values determined for different grades of SSR cast in concrete, but due to lack of such appropriate test data the classification is based on PREN values. The selection of the reinforcement grade should also take account of the beneficial effect of the high pH environment surrounding the SSR and the effect this has on the passivating layer, see *Figure 4-1*. A classification based on PREN values is not able to take into account this effect but then the risks of carbonation of the concrete reaching the reinforcement within the design life do not influence the classification either.

The choice of material grade depends both on the design service life and the environmental aggressivity. The results of research over the last 30 years has provided a rational basis for the selection of material grades for a given application, based on probable chloride levels within concrete structures. In general a specific evaluation of the relevant parameters should be performed for the specific structure and environmental conditions.

In order to sum up this experience related to the proposed SSR classification an attempt on exemplification is performed (Table 4-1). For concrete structures located in marine environment in the Nordic countries (moderate temperature and relative humidity) with a low design service life (10 – 30 years), carbon steel reinforcement (Class 0) may be used. For concrete structures located in the same environment with a moderate design service life (50 – 100 years), Class 1 SSR (Table 4-1) should be used in the most exposed zones to gain a reliable solution. For long design service life (200 - 300 years) in the same environment, the use of Class 2 SSR may be appropriate. Further for concrete structures in moderate temperature and relative humidity exposed to high chloride concentrations from e.g. de-icing salt, Class 2 SSR should be adapted to gain a moderate design service life. The same recommendation could be given for structures located in marine environment combined with high temperature and relative humidity and a request for moderate design service life and for cases where the steel is only partially embedded (dowel bars, inserts, fastenings and holding down connections). The use of Class 3 SSR should be evaluated if the design request is long service life for structures in marine environment with high temperature and relative humidity.

If material costs are included in the evaluation, the distinction between Class 2 and Class 3 materials become a bit irrelevant as the prices today are quite equal. The Lean duplex steel reinforcement (a Class 2 material) may be an option in the near future, pending the needed documentation and commercial production of ribbed reinforcement.

The selective use of stainless steel shall include both the recommendation of when and where this procedure may bring the desired results and also the precautions to be taken against the opposite effect. This should include guidance for type of stainless steel to be chosen for certain application and environment, proportioning of mix between stainless steel and carbon steel, and description of recommended connection type (physical by bending or welding including all precautions).

Particular specifications for works with stainless steel reinforcement should as a minimum include the following headings if not covered by the general specifications:

- Location; - where shall SSR be used

- General; - type of steel

- Relevant standards should be mentioned

- Material Properties; - physical properties

- Workmanship

- Bending of SSR

- Fixing of SSR

- Anchorage and lapping of SSR

- Welding; - Specify if welding is allowed

- Testing; - Additional testing

- Storage; - Separation from carbon steels and marking

- Spacers; - Requirements to spacers (for instance non-metallic spacers) and tying wire

6.3 Concrete section design

The inherent corrosion resistance of SSR allows room for considerable changes in the design for durability compared to current designs based on carbon steel reinforcement. These changes reflect relaxations which also will result in overall cost savings on the individual items, an issue which will compensate for the prevailing high costs for the SSR compared to the carbon steel reinforcement.

These specific design issues are addressed below.

6.3.1 Concrete cover

Concrete cover has three main purposes in the structure:

- Ensure adequate anchorage of the reinforcement at bar ends and at lap joints. This is a structural issue independent of durability issues

- Protect the (carbon steel) reinforcement against corrosion caused by ingressing chlorides or from an approaching carbonation front. This determines the quality required of the concrete, represented by the penetrability to the aggressive substances in question, in combination with the cover thickness. In addition, the level of alkalinity of the concrete is an issue as higher pH leads to higher chloride threshold values, for carbon steel as well as for stainless steel

- Protect the reinforcement against excessive heat from fires

Anchorage: The structural aspect of anchorage is the same for the two types of reinforcements. Hence minimum covers allowed for carbon steel reinforcement shall also govern for SSR. Based on current code requirements the minimum concrete cover to the outermost reinforcement bars can be relaxed to 30 mm with a tolerance of typically +5 mm, which determines the dimension of the spacers to minimum 35 mm.

Protection of the reinforcement: No additional corrosion protection of the SSR is required. Only in zones where the SSR and carbon steel reinforcement is coupled or lapped, or where only carbon steel reinforcement is used shall the traditional covers

prevail, but adjusted to the environment (exposure conditions) governing in these zones. This latter environment should in general be considerably less onerous than where the SSR is applied - that is the whole idea of using SSR selectively.

Fire protection: The need for fire protection is similar for the two types of reinforcements. However, the SSR is more tolerant to high fire-induced temperatures by loosing strength only at higher temperatures than carbon steel. In addition, the relaxed concrete denseness needed in the case of adopting SSR reduces the risks of explosive spalling, and thus the selection of SSR introduces indirectly higher resistance - or safety - against damage by fire. This may however not be the case, where the design needs to take both water tightness and corrosion into account.

6.3.2 Concrete mix and concrete quality

Having solved the corrosion problem through the selection of an appropriate grade of SSR then the selection of the concrete mix can be optimized by taking a number of other properties into account. With reference to the type of mix usually adopted to protect carbon steel reinforcement, these additional properties would typically lead to relaxations regarding the concrete denseness and permeability properties towards deleterious substance penetrating into the concrete in gaseous and aqueous form.

Such relaxation of the usual purely corrosion preventive properties would lead to the following:

- The type and quantity of pozzolanic additives may be optimized regarding concrete durability and cost. The more the cementitious binder consists of OPC the higher the pH - raising the chloride corrosion threshold values - and the larger the calcium hydroxide buffer capacity of the mix to resist carbonation

- The water-cement ratio may be raised to say 0.45. This leads to a reduction in plasticizers and super-plasticizers needed to ensure dispersion of the pozzolanic admixtures - if any - and ensuring workability of the concrete. Following from this the risk of early age cracking (plastic shrinkage cracking and thermal cracking) and autogenous shrinkage is reduced. Fire resistance may also be enhanced due to the more porous concrete reducing the risks of explosive spalling.

All in all these possibilities will usually lead to savings in the cost of the concrete.

It shall be recalled, that all additives and admixtures added to concrete to protect against reinforcement corrosion will for practical reasons realistically have to be mixed into the whole bulk of the concrete although only needed in the outer concrete cover zone. This represents not only a considerable waste in materials and an environmentally unfriendly solution, but introduces also difficulties and added cost in execution.

6.3.3 Crack widths

The crack widths needed to be controlled according to prevailing codes and standards refer to the visible and measurable crack widths on the concrete surface. The control of cracking on a concrete structure is determined by the amount, size, strength and distribution of steel reinforcement, the concrete cover and the concrete strength, together with the level of strain (stress) in the main reinforcement.

It is today acknowledged within the profession that the at times very strict crack width limitation imposed on designs in corrosive environments may lead to the consumption of large quantities of reinforcement, far beyond what is required from an ultimate limit state calculation. Such large quantities of reinforcement may induce execution difficulties with

casting and compaction of the concrete, leading to reduced quality of the concrete cover, which in fact should provide the best quality from a durability point of view.

Where SSR is used an unlimited value of maximum crack can theoretically be tolerated from a purely corrosion point of view. With SSR visually acceptable cracks widths and tightness - together with possible effects on deflections and vibrations - will govern the tolerable size of crack width. A relaxation of the crack width requirements from typically 0.10 - 0.20 mm for concrete protecting carbon steel reinforcement to 0.30 - 0.40 mm, would be acceptable from a structural point of view according to several codes.

Where watertight structures are concerned special cases can apply.

6.3.4 Minimum reinforcement

Also with SSR care shall be taken to ensure sufficient reinforcement to ensure a good distribution of cracks and thus avoid large size single cracks. If the design has adopted relatively small concrete covers, it facilitates fulfilment of the minimum reinforcement requirement.

6.3.5 Example of the benefits by replacing carbon steel with SSR

In connection with Sitra Causeway project in Bahrain, which includes marine bridges and highway interchanges, the detailed design was made adopting SSR in the critically exposed concrete sections. For comparative reasons the Client requested an overview of the initial cost savings if the SSR was replaced with normal carbon steel reinforcement. Therefore a re-design with carbon steel reinforcement was made.

The design basis was BS 5400 /13/, with a design service life of 120 years, of which the first 40 years shall be maintenance free. The design crack widths and required minimum cover were only moderately modified compared to the suggestions above:

- SSR (1.4462) : maximum crack width: 0.25 mm, minimum cover: 45 mm

- Carbon steel reinforcement: maximum crack width: 0.10 mm, minimum cover: 75 mm.

As the bridges were prestressed longitudinally the amount of longitudinal mild steel reinforcement was limited, and the increased amount of reinforcement due to the shift from SSR to carbon steel reinforcement was only a factor of 1.1 - 1.2. However, the transverse reinforcement should be increased by a factor of 3.0 due to this shift. Using the BS 5400 as design basis for crack width control also introduced the effect of using only the nominal cover for designing the crack width, not the required minimum cover. Hence the 75 mm minimum cover (+10 mm tolerance) was only introduced as 55mm in the cracks width calculation. The 45mm cover was only introduced as 25 mm in the calculations. Without this effect of limiting crack width design cover to these smaller nominal values, the difference in reinforcement consumption in the two situations analyzed would have been even greater.

Due to the distribution of the reinforcement in longitudinal and transverse direction and at mid span zones and over the supports, the overall increased amount of carbon steel reinforcement compared to stainless steel reinforcement was a factor of 2.05. This detailed analysis highlighted the main effect of the more relaxed design requirements regarding crack widths and cover when adopting SSR. The relatively higher cost level for the SSR compared to carbon steel reinforcement was shown to be halved just by this exercise. At the same time a harmonic balance between the amounts of reinforcement needed for structural reasons and for durability - or serviceability - reasons was achieved.

Further savings would be attributed to the reduced dead load due to saved cover, possible savings in concrete additives and admixtures as particularly dense high performance concrete to protect the reinforcement is not needed as only the protection of the concrete itself from

deterioration is needed. The reduced quantity of reinforcement will allow easier casting and compaction of the concrete. In countries with high unit labour costs the savings in this large reduction in reinforcement needed to be handled on site would also become a non-negligible factor of saving.

All in all, the whole concept of solving the reinforcement corrosion problem at the origin of corrosion, namely through the chosen type of steel reinforcement, introduces a new perspective in the design and execution of long-term durable structures in highly corrosive environments and at the same time this approach raises the level considerably of the reliability of the structural and durability performance of such structures.

6.4 Execution and after-treatment

The selection of a dense concrete to protect the reinforcement against ingress from aggressive substance will have a number of adverse effects from an execution point of view. Apart from the increased demand for experience with such concretes when casting and compacting the concrete, the early age properties and after-treatment with respect to controlling evaporation and temperature gradients become key durability issues.

The adoption of SSR may in general lead to considerable relaxations in the demand for special competence in execution. In other words a much more tolerant and robust concrete can be adapted to the overall benefit of the structural durability and performance.

6.4.1 Plastic shrinkage cracking

The very dense concretes experience no bleeding when cast and the surface has been finished. This means that any evaporation of water from the freshly cast surface cannot be compensated by a slight migration of water inside the dense structure of the concrete. Hence, such concretes are very prone to plastic shrinkage cracking. Due to the denseness of the concrete traditional sprayed curing membranes are not sufficiently effective to avoid evaporation. The Danish road authorities allow maximum 6% micro silica in the concrete mix for the same reasons.

Adopting SSR together with the corresponding relaxations in the concrete mix will reduce the plastic shrinkage problem.

6.4.2 Thermal cracking

The heat generation during hardening of the concrete introduces temperature gradients across the casting section as well as between a former and a new cast section across a construction joint.

Adopting dense concrete for corrosion control reasons introduces also a very brittle and sensitive concrete from a deformation point of view. The autogenous shrinkage and the dense packing of the concrete particles leaves very little strain capacity in the concrete.

Adopting SSR together with the corresponding relaxations in the concrete mix will reduce the thermal cracking problem considerably; especially the relaxation in water/cement ratio requirement and cement content is of importance.

6.5 Transport, storage and handling

Stainless steel reinforcement should in general not be stored in contact with carbon steel because of the risk of rust staining.

It shall be stressed that such rust staining is only the small carbon steel particles on the surface of the SSR that corrodes. This results often in the misconception that the SSR is corroding, but this is not the case. Similarly, leftover microscopic remains of the mill scale from the

production of the stainless steel might give cause to similar harmless discolouring prone to misunderstandings. The observation is only of visual - and psychological - importance.

A different situation may occur in the case that SSR is physically forcefully treated with cutting and bending equipment of ordinary carbon steel. Theoretically small carbon steel particles may be pressed into the lattice of the stainless steel. If they start to corrode it could be assumed that this corrosion might develop into the lattice of the stainless steel and cause further damage such as inter-crystalline corrosion, or maybe pitting corrosion. However, this is an issue which to the authors' knowledge never has been reported, but the situation has also not been investigated through targeted tests.

It seems that further research is needed in order to clarify reality of the theoretical concern of a possible corrosion risk from carbon particles pressed into the lattice of the different grades of SSR, see *Figure 6-1*. This activity is therefore proposed in the Chapter 8 "Further investigations".

Apart from that, no additional precautions beyond those for carbon steel reinforcement are required.

Figure 6-1: Stainless steel bar with iron contamination (contamination greater than 10 mm in length) due to spray of metal particles from adjacent black bar cutting operation.

6.6 Installation, welding and coupling

The self-repairing characteristics of the oxide film on stainless steel mean that the integrity of the film is maintained, even if the stainless steel reinforcement suffers mechanical damage during handling.

Stainless steel reinforcement should always be fixed with softened stainless steel tying wires of the same quality as the structural steel. Spacers made with concrete, or cement mortar should be used. It is recommended to use stainless steel chairs for support.

SSR should only be cut with equipment designed solely for that purpose. If SSR is cut with a disc cutter or angle grinder any "blueing" of the steel, i.e. thermal oxides caused by the cutting, must be removed with a proprietary pickle paste, otherwise the corrosion resistance of the steel may be impaired.

Welding of stainless steel is possible but not recommended in site conditions, because if the weld products is not completely removed, corrosion resistance is reduced. Pickling or shot-blasting the weld can often solve the problem, but is not practical on construction sites. In the factory or at precasting yards welding is no problem provided the general rules for welding stainless steel and stainless steel in contact with carbon steel is ensured.

Stainless steel couplers are available for connecting lengths of bar longitudinally, providing in most cases a direct alternative to welding. They can be used to connect two lengths of carbon steel, carbon and stainless, or of stainless steel. The risk of galvanic corrosion is virtually non-existing under most realistic conditions.

6.7 Use of cover meters

Traditional cover meters cannot detect austenitic stainless steel reinforcement, as they work using an induction method requiring magnetic reinforcement. Special cover meters have been developed claiming that measurements can be conducted on non-magnetic reinforcement.

Duplex stainless steel is magnetic but has a poor conductivity. They are detectable by conventional cover meters but the signal received will be weak. Thus, the measured cover depth should be checked by visually control of the spacers (number, cover and distribution) and spot checks by drilling after the reinforcement has been tentatively located, and the readings calibrated with this measurement.

6.8 Other corrosion preventive measures

6.8.1 Surface treatment of concrete

Surface treatment and coatings of concrete are available in many types and qualities, from invisible hydrophobic silane impregnations to thick pigmented acrylic, polyurethane or epoxy based coatings.

The experience with silane impregnations vary from country to country. In some countries such treatment is compulsory for bridges exposed to de-icing salts. In other countries where the hydrophobic effect cannot be verified, silanes are not generally used.

Coatings may have durability enhancing effects by slowing down the rate of carbonation or chloride ingress. However, the verification of the beneficial effects is very difficult in practice. It has been demonstrated that pinholes in a thick carbonation resistant coating reduces the protective ability to just fractions of the effect of a pinhole-free coating - and it is not realistic to make pinhole-free coatings on concrete - even after having applied a surface wash using a smooth polymer modified cementitious material.

In addition, surface treatment to concrete is exposed to degrading effects from the environment and from UV light. Hence such coatings shall be maintained or re-applied after a limited number of years, depending on the type of material and exposure.

6.8.2 Corrosion inhibitors

The newest addition to the concrete mix is the introduction of corrosion inhibitors. They can be anodic-, cathodic- or mixed inhibitors. The corrosion inhibitor technology is well-known within the chemical industries. However, adding inhibitors to concrete has resulted in diverging reports on their efficiency. It also seems that adding too small quantities of nitrate corrosion inhibitor compared to the future concentration of chlorides may even cause increased corrosion rates. The most usual inhibitors also act as concrete accelerators, which are not always an advantage, particularly in hot environments, where high temperatures often cause casting difficulties due to rapid loss of workability and early hardening.

The risk of the inhibitor being washed out of concrete exposed to splashing water is also an issue under discussion as the inhibitors seems more mobile than chlorides. The effect of washing out of inhibitors has not been verified in the laboratory tests, and in situ verifications after long-term exposure are still missing. In this connection it shall be recalled that inhibitors

are dormant in the concrete until inhibitor and chlorides are present at the same time at the surface of the steel reinforcement.

The inhibitor must, for several reasons, be added to all the concrete in the exposed components although it is only needed in the vicinity of the reinforcement. This has cost implications for the construction.

With these uncertainties the level of long-term reliability of the inhibitor solution is questioned.

6.8.3 Cathodic prevention

Cathodic prevention has become an interesting option to protect the parts of structures to be exposed to high chloride concentrations. In this way a cathodic protection system is installed already within the new structure ready to be used to protect the reinforcement against chloride corrosion. This option seems most promising in marine structures as a sacrificial anode system may be installed simply by placing anodes in the water near or on the structure and link them up to the reinforcement. Similarly, impressed current systems may be used, but they will impose on the owner or operator a permanent monitoring of current and of protection level. The impressed current system can comprise inert anode meshes or bands cast into the concrete structure, when new or added later, when needed.

However, installing cathodic protection at a later stage will only stop further corrosion; there will be no restoring of lost steel. Therefore such a solution shall be implemented promptly when the need has been identified.

The unavoidable life-long monitoring of the installations is also an issue needing careful consideration. With required design lives in excess of 50 to 100 years the reliability of such a solution seems to diminish with time.

6.8.4 Non-metallic reinforcement

Fibre reinforcing bars have been developed and it is recognized that corrosion resistance is achieved with such reinforcing bars. At present such non-metallic bars are based on glass, aramid or carbon fibres.

However, the different mechanical characteristics between fibre reinforcement and steel reinforcement, together with the different conditions for practical use on site, seems to indicate only rather limited or special applications in concrete structures. Fibre reinforcing bars have very limited strength when loaded transversely to the direction of the fibres. This is an issue for the connection between main bars and stirrups. Some bars are also very sensitive to hard impacts. Carbon fibre bars may fracture if hit by a falling hammer or a dropped vibrator. During casting of concrete the workers cannot walk on the reinforcement, temporary work bridges must be used.

All such fibre reinforcing bars with very low mass must also be anchored to the form to prevent floating due to their buoyancy.

Durability aspects have also been questioned for glass fiber reinforcement.

Currently the precast industry seems to present the largest potential use of fibre reinforcing bars.

6.8.5 Corrosion resistant steel reinforcement

Epoxy coating of reinforcement

Epoxy coating of reinforcement was a new protective measure against chloride induced corrosion introduced in North America in the mid 70´ies. Fusion bonded epoxy coating has

since then been used extensively in North America, and later in other countries. However, from the early 90´ies reports began to emerge indicating that an undercutting chloride induced corrosion may develop from local pinholes and damages in the coating without necessarily causing cracking and spalling of concrete cover, just slowly disintegrating the reinforcement. The first examples were from the bridges on the Florida Keys, At that time the technology was slowly spreading to Europe, the Middle East, and to some parts of the Far East.

Hence, before the problems with epoxy coated reinforcement became public in North America, a viable and reliable technology for precasting plants was developed in Europe to eliminate cutting, bending and patch repairing of coated bars, and at the same time maintaining the possibility of introducing cathodic protection if corrosion was initiated after all.

Figure 6-2: Illustration of the adverse corrosion protection performance of epoxy coated reinforcement in the vicinity of the saline waters of the Arabic or Persian Gulf under hot and humid environments.

The North American experience with the traditional technology of epoxy coating together with additional testing and site investigations, among others in Ontario, Canada, has lead to this technology not gaining foothold in Europe, and the technology is now slowly being phased out, also in the Middle East and Gulf Countries. Examples from a marine structure in the Gulf reinforced with epoxy coated reinforcement are illustrated on *Figure 6-2*.

Hot-dip galvanised reinforcement

With respect to hot-dip galvanized reinforcement (HDG) not much practical experience is available regarding enhanced performance in very aggressive environment. However, it has been shown in tests that HDG steels may have a higher threshold value for chloride induced corrosion (initiation) compared to black steel. The protective ability depends on the reactions at the iron-zinc interface.

An evaluation of the available date and practical experience indicate that galvanizing of well controlled quality and thickness and applied to appropriate reinforcing steel alloy, will provide:

- Reliable corrosion protection of reinforcing steel in concrete exposed to carbonation but without chlorides and sulphates

- Slightly increased threshold level for chloride induced corrosion compared to non-coated steel. However, once corrosion starts the rate of pitting corrosion may be higher, and at times much higher, than for uncoated black steel. For immersed conditions this effect may be further aggravated

As stated previously stainless steel reinforcement can be mixed with ordinary black steel (carbon steel) reinforcement without causing galvanic corrosion. However, zinc coated steel

reinforcement (galvanised steel, grey galvanizing) cannot be mixed with carbon steel reinforcement as this will cause galvanic corrosion of the black steel. Therefore, a simple replacement of Stainless steel reinforcement with galvanized steel reinforcement is not a viable solution. Realistically all reinforcement should then be galvanized.

Nevertheless any post coating mechanical treatment like cutting and bending will reduce the reliability of the galvanizing to provide adequate corrosion protection.

Stainless steel cladded black steel reinforcement

Recently a hybrid solution has emerged, this being stainless steel cladded carbon steel reinforcement, "Nuovinox" /44/. The stainless layer provides it with the corrosion resistance and the inner carbon steel core gives the necessary physical and mechanical properties. The outer stainless layer can be made from different grades of stainless steel.

The composite nature of the Nuovinox product is clearly seen in the cross-section of the rebar. The outer stainless steel appears brighter than the carbon steel core. The cladding itself is of uniform thickness with an average variation between 0.5mm to 1.0mm around the perimeter and along the length for a 16mm rebar.

Core composition and finishing rolling temperatures can be adjusted to achieve the tensile properties required. The stress strain curve for Nuovinox is the same as for a carbon steel bar.

Three main drawbacks can be identified:

- The reinforcement can only be produced in short lengths: Usually 6m and by special order 11.6m.

- The reinforcement cannot be produced in coils, which would have given major simplifications for the on site automatic production of prescribed lengths and bending of stirrups and links.

- The cut ends are exposes the carbon steel core and would therefore be exposed to local chloride ingress. Therefore it is recommended by the producer to provide such bar ends with a stainless steel cap. The bars shall then be cut by means that does not deform the ends. Usual scissor type cutting cannot be applied. This could be considered a major practical drawback of this technology.

- Currently available maximum dimensions are limited to 20mm.

Microcomposite Multistructural Formable Steel (MMFX)

Microcomposite Multistructural Formable Steel, MMFX steels, are manufactured without the use of coating technologies as a result of a patented chemical composition and proprietary steel microstructure that is formed during the production associated with controlled rolling and cooling of the steels. This physical feature should minimize the formation of micro galvanic cells in the steel structure, hence minimizing corrosion initiation /45/.

MMFX steel has low carbon content, less than 0.15%, and contains around 8-10% chrome.

For the MMFX steel reinforcement to be acceptable to the concrete community, a number of design and detailing characteristics must be determined. The mechanical characteristics of the reinforcement by themselves, and in conjunction with concrete must be independently validated.

7 Applications for stainless steel reinforcement

7.1 Selective application of stainless steel reinforcement

Stainless steel reinforcement has been used in a wide range of applications, such as bridges, tunnels and underpasses, retaining walls, foundations, marine structures, historic buildings and other structures with special long service lives in the last 30 years /5/ and /18/.

The most cost optimal solution is to use SSR in the most exposed zones/parts of the structure. For bridges it could be in edge beams, expansion joint sections, piers and piers tops and bridge deck soffits. Typical potential application of SSR in marine and offshore structures includes piers and soffits of jetties, part of oil rigs in the tidal or splash zones, exposed face of building close to the shoreline, seawalls etc.

The selective use approach is illustrated in *Figure 7-1* and *Figure 7-2*, showing a large building complex in the Gulf region. SSR (1.4462) was here introduced from somewhat below groundwater level to level +5m in the outermost structures directly exposed to the salty ground water and seawater spray of the Gulf. All remaining parts are reinforced with normal carbon steel. At the same time the reinforced concrete seawall protecting the new building complex is being replaced using SSR (1.4462) throughout.

Figure 7-1: A large building complex under construction in the Gulf directly facing the sea

Figure 7-2: New precast concrete seawall elements to replace an existing seawall adopting SSR (1.4462)

Figure 7-3: Stonecutters Bridge, Hong Kong. Pylons reinforced with SSR (1.4462) in the outer layer of the multi-layer of reinforcement

Figure 7-4: Sheik Zayed Bridge, Abu Dhabi. Lower part of supports are of reinforced concrete with SSR (1.4462) in the outer layer of reinforcement

Recently a number of very large and prestigious bridges in highly corrosive marine environments have adopted SSR in the outermost horizontal and vertical reinforcement layer of the most exposed parts of the structures. These are the Stonecutters Bridge, *Figure 7-3*, and the Shenzhen Corridor in Hong Kong or the Sheik Zayed Bridge in Abu Dhabi, *Figure 7-4*. All these bridges are currently under construction. The remaining layers of the multi-layer of reinforcement are ordinary carbon steel reinforcement, but now with a larger concrete cover and with effective crack width control due to the outer SSR layer.

An intermediate possibility could also be to use SSR only as the stirrups in more moderate cases of exposure.

7.2 Repair works

Stainless steel reinforcement is now being introduced into more repair projects. As stainless steel is a much poorer cathode than carbon steel, SSR can be beneficial in those repair cases where ordinary carbon steel has corroded to such an extent that local replacement or added reinforcement is needed as part of a repair.

A growing application area for SSR is repair and renovation of historical buildings where very long design lives are required.

Welded mesh reinforcement of stainless steel are being used extensively, both for new constructions like parking decks etc, but also in connection with repairs of reinforced concrete. In the latter case such fine mesh is used extensively to control cracking where cover is low in the repaired zones.

A typical example is the replacement of parts of the seawall in Sydney Harbour following extensive reinforcement corrosion of the reinforcement. The structural safety was below acceptable level and the structure became visually unpleasant. Stainless steel reinforcement is introduced to avoid future repairs, see *Figure 7-5*.

Figure 7-5: Reconstruction of the seawall in Sydney Harbour using SSR (1.4462) following extensive corrosion damage of the reinforcement.

7.2.1 Replacement of central crash barrier

In 2005 and 2006 the existing 5.7 km of concrete crash barrier on the Free Way between Copenhagen and Lyngby was replaced. The existing crash barrier consist of two parallel rows of concrete elements - New Jersey type, each with a length of 4m, join with a concrete bottom slab in between the rows of concrete elements, and closed with concrete top slab elements.

The existing barrier was 35 years of age and has extensive reinforcement corrosion and damage to the concrete cover, due to chloride ingress, frost and delaminating of the concrete.

Based on aesthetic and safety considerations the new crash barrier was designed with similar geometry and colour as the existing old barrier.

The new concrete crash barrier was designed according to the standards EN 1317 /46/, DS 2426 /47/ and EN 206 /8/, and the use of white concrete was required.

Due to the severe aggressive environment in which the crash barrier is situated, the concrete must fulfil the requirements for an extra aggressive environmental class in compliance with DS 2426, hence the concrete specifications and concrete cover was defined. Further more surface cracks are not accepted due to aesthetic reasons.

In this case a cost/benefit analysis was made considering the following aspects:

- aesthetic

- durability

- limited experience with durability for white concrete in severe aggressive environment

- additional cost for stainless steel (AISI 316-quality) compared with black steel

- local reduction in the concrete cover due to geometry of concrete elements

Based on this analysis stainless steel reinforcement was deemed preferable.

7.2.2 Replacement of concrete edge beams on bridges

Due to the extensive damages to concrete edge beams on Danish high way bridges a great number has been replaced during the last 5 years.

The aggressive environment has coursed large damage such as cracking, reinforcement corrosion and spalling of the edge beams.

Based on the damages seen on the existing edge beams, replacement is designed according to the requirement for extra aggressive environment according to DS 2426 /47/.

However, experience with in situ casting with high performance concrete on the existing concrete structures, have shown difficulties in avoidance of cracks in the new concrete. Such cracks, which are caused both by temperature and short and long term shrinkage conditions, may thus leave the reinforcement exposed to water and chlorides from de-icing salt.

Figure 7-6: Replacing edge beams and parapets on bridges in Denmark exposed to de-icing salt.

With the relative small amount of steel needed in these projects the additional cost for using stainless steel is acceptable as shown in Chapter 7.3.

7.3 Life cycle costing

Life cycle costing (LCC) is a technique developed for identifying and quantifying all costs (initial and running) associated with a project over a given period of time. LCC uses the standard accountancy principle of discounted cash flow, so that all costs incurred during a life cycle period are reduced to present day value.

The life cycle costing is, for instance, by the Danish Road Directorate handled in connection with special investigations in a systematic and rational manner. The methodology can be used also where selection of durability strategy for structures to be built are concerned. A time span of 50 years is looked upon, but can in principle be extended to cover any required period of

time. In a spread sheet different strategies A, B and maybe a third strategy C is selected. In relation to strategy for a new structure this can be:

A: Construction using stainless steel and carbon steel reinforcement, selective as appropriate

B: Carbon steel reinforcement with high performance concrete and relatively high concrete cover.

In the spread sheet all costs during the anticipated service life are inserted, the construction costs in year zero, and inspection, operation and maintenance costs in the years to come. In case an expected repair before onset of corrosion is expected after 50 years of operation for strategy B, the price for repairs are inserted corresponding to year 50.

The costs are converted to present value costs using the real rate of interest. The real rate of interest used by the Danish Road Directorate is set at 7 %.

Together with an evaluation of the actual costs (total and present value) also indirect costs are included in the calculations. The indirect costs include traffic hazards, where the disturbance made to the Traficant's is priced in relation to the time of disturbance.

This means that a life cycle planned with two or more repairs during the service life in traffic heavy areas will have considerate indirect costs included before the present cost value is calculated, and could result in a conclusion that strategies including repair during the service life is not cost optimum. Correspondingly, marine structures will often not have indirect costs during repairs (except for water tanks on the roadway used for water jetting) and construction and repair strategies where stainless steel and/or preventive repair strategies are considered will often turn out to be the optimum solution.

The methodology etc. is described in more detail in /48/ . It should be noted that the real rate of interest has a significant influence on the calculated optimum strategy.

In the following an example is given where the life cycle costs of replacement of approx. 100 meter edge beams have been calculated using stainless steel and carbon steel, respectively (see 7.2.2). The calculation is based on an estimated service life of 50 years for both types of reinforcement. Maintenance costs for carbon steel are further estimated at an extra 200.000 DKK every 5th year during the last 30 years of the service life. The indirect costs are secondary costs caused by the estimated traffic disturbances.

The calculations are based on the Danish Road Directorate's present value program /48/. *Figure 7-7* shows the net present value as function of various real rate of interest a result of the calculations (A0 : Stainless steel reinforcement and B0 : Carbon steel reinforcement).

In *Table 7-1* the key figures are summarized for the total costs and the costs of a present value calculation using a real rate of 7 %. As seen the use of stainless steel is the most financially advantageous solution both with respect to total costs as well as present value calculation.

Table 7-1: Costs in mill. DKK

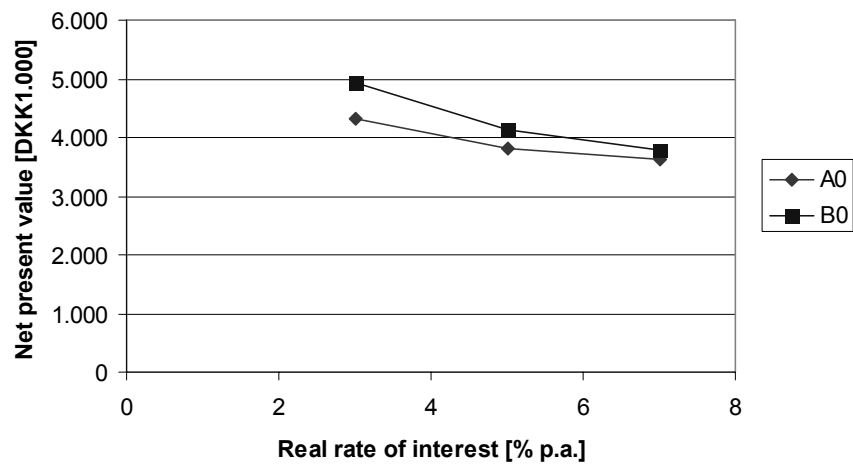|	Strategy A Stainless Steel	Strategy B Carbon Steel
Total costs	7,0	8,6
Present value, real rate 7 %	3,6	3,8

Figure 7-7: Net present value as function of real rate of interest

8 Further investigations

There remains a number of investigations and test that would be convenient to have made when discussing the rapidly growing adoption of the nearly foolproof solution of stainless steel reinforcement to the worldwide problems of corrosion of carbon steel reinforcement.

Among such tests are:

- Objective determination of the chloride threshold values for the different grades of SSR as a supplement to the relative level of corrosion resistance represented by the PREN values developed for a different situation. Of particular interest will be methods clarifying the uncertainties related to the more cost effective types SSR like the low grades of stainless steel and the new types SSR like the lean duplex types. Currently it is a general impression that some "overshooting" governs the selection of grades, to be on the safe side so not to destroy this very promising solution to a serious problem.

- Testing to clarify reality of the theoretical concern of a possible corrosion risk from carbon particles pressed into the lattice of the different grades of SSR.

- Testing to clarify the influence of coupling between corroding carbon steel and stainless steel when the unexpected corrosion behaviour occur (case when low alloyed stainless steel used as skin reinforcement maybe forced to corrode by high amount of chlorides penetrating concrete and which is electrically connected to the passive carbon steel located behind).

- Testing influence of welding on the coupling between carbon steel and stainless steel. It should be expected that welding will reduce the corrosion resistance by reducing the chloride threshold levels, which is due to the combined effect of oxide and insufficient compaction of concrete around the weld. For this reason a post cleaning process should form a part of welding procedure qualification. This procedure should also include the accelerated testing to demonstrate the influence of cleaning process on the corrosion resistance properties.

- Comparative study of corrosion properties of different types of stainless steel reinforcement and cladded steel by determination of chloride threshold values under accelerated exposure conditions.

9 Summary and conclusions

In recent years there has been an increasing interest in applying stainless steel reinforcement (SSR) in concrete structures to combat the durability problems associated with chloride ingress.

A convincing documentation of the performance of stainless steel reinforcement in highly chloride contaminated concrete is presented by the 70 year old concrete pier at Progresso in Mexico. This pier was reinforced with stainless steel reinforcing bars (quality 1.4301) and no corrosion has taken place within the structure yet despite the harsh environment. The chloride levels, at the surface of the reinforcement are more than 20 times the traditionally assumed corrosion threshold level for carbon steel.

It may be concluded that designing structures with SSR may in principle be performed by a simple replacement of ordinary carbon steel reinforcement with SSR in the ratio 1:1 as the structural properties are the same regarding strength and ductility (or better for several of the available types of SSR). Further, using SSR in design other advantages should also be utilized, such as: relaxation of concrete cover requirements, crack width requirement, and maybe concrete quality (permeability) requirements.

An important fact is that stainless steel reinforcement can be combined with black steel cast into concrete without risks of galvanic corrosion due to bi-metal - or galvanic - action. In fact, this is the precondition for general economical application of stainless steel reinforcement used only in the parts of the structure where this protection is needed, - so-called selective use.

Ribbed stainless steel reinforcement is available in a number of different material grades. Choice of material grade should depend on the design service life and the environmental aggressivity.

A more operational classification system is proposed in this guide introducing corrosion resistance classes (Class 0-3). For concrete structures located in marine environment with moderate temperature and relative humidity and with a design service life below 100 years, Class 1 SSR should be used to gain a reliable solution. For long design service life (200 - 300 years) or for concrete structures exposed to high chloride concentrations (e.g. de-icing salt) or for concrete structures exposed to high temperature and relative humidity, Class 2 SSR should be adapted. For concrete structures exposed to a combination of above criteria (e.g. long service life combined with high temperature and relative humidity) use of Class 3 SSR should be evaluated.

The main conclusion of this guide is that if SSR is to be used cost effectively, it is essential that the design and specification of reinforcement minimizes the impact of the initial cost increase resulting from the use of stainless steel. This can be achieved in a number of ways:

- by adapting life cycle costing (LCC) as an integral part of the service life design process

- by correct selection and specification of stainless steel material grade

- by changing the design approach for durability requirements developed for carbon steel by a new approach adapted that reinforcement corrosion is solved through the use of SSR

- by using stainless steel selectively only on structures or elements or at surfaces of structures that are at risk of degradation from chloride induced corrosion

- by using higher strength steels, as available with the SSR, to reduce the overall quantity of reinforcement - when structurally and performance based acceptable

The adoption of SSR in concrete structures is the first clear indication that changes in the steel formulations is needed. The competitiveness of the relatively expensive SSR has led to the development of the different types of reinforcements mentioned in this Guide. "Lean Duplex" is one such promising renewal within the corrosion resistant types of reinforcement. One concern has been that in several instances the traditional types of SSR will in some situations be considered "overkill" for concrete structures. It is assumed that this development is just in its infancy, and a keen eye should be kept on this development in the years to come.

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