RESEARCH AND DEVELOPMENT

Cost-Effective Enhancement of Durability of Concrete Structures by Intelligent Use of Stainless Steel Reinforcement

ARMINOX Stainless

Cost-effective Enhancement of Durability of Concrete Structures by Intelligent Use of Stainless Steel Reinforcement

Knudsen, A. [(1)], Jensen, F.M. [(1)], Klinghoffer, O. [(2)] & Skovsgaard, T. [(3)]

ABSTRACT
Reinforced concrete has been used successfully in the construction industry since the beginning of this Century. At present a large number of reinforced commercial buildings, domestic dwellings, marine structures, bridges, etc., are starting to show serious signs of deterioration, particularly those over 30 years of age. This deterioration is mainly caused by corrosion of the reinforcement. The annual cost of repair work on concrete structures is now in excess of US$ 5 billion in Western Europe alone. Stainless steel reinforcement products have developed considerably and a wide range of alloys today offer a total solution for providing corrosion-free reinforced concrete structures. The paper presents both a capital cost and a life-cycle cost analysis in the case when stainless steel is used in combination with carbon steel bars for concrete repairs. The paper also focuses on the unfounded fear of using both stainless steel and carbon steel in the same concrete structure. Further, the paper shows that **intelligent** use of stainless steel (use of stainless steel in critical areas due to deterioration or accessibility) for repair of deteriorated concrete structures is a cost-effective option when considering various rehabilitation alternatives.

KEYWORDS
Concrete structures, deterioration, repair, rehabilitation, durability, stainless steel, intelligent use and life-cycle cost analysis.

INTRODUCTION
Both stainless steel and carbon steel derive their corrosion resistance from a naturally occurring chromium rich oxide film, which is present on its surface. This invisible film is inert, tightly adherent to the metal, and most importantly in an environment where oxygen is present, even at relatively low levels, the film reforms instantly if the surface is damaged, [1]. There are, however, aggressive environments (e.g. carbonation or ingress of chlorides) which can give rise to breakdown of this passive layer resulting in corrosion of the unprotected surface. When deterioration has developed to a given point, rehabilitation measures are required. Among the various rehabilitation options modern stainless steel has become an attractive alternative when compared to traditional methods with carbon (unalloyed) steel, epoxy coatings, corrosion inhibitors, cathodic protection, etc.

Deterioration of reinforcement is a complex process, where several aspects regarding the concrete, the reinforcement steel, the structural design, and the environment all may have an influence. To find the optimal rehabilitation strategy, it is decisive for the bridge or building owner to include all costs throughout the remaining lifetime of the structure, such as repair costs, maintenance costs, administrative costs, consulting fees, costs of disruptive traffic alterations (road user costs), etc.

[(1)] RAMBOLL, Bredevej 2, DK-2830 Virum, Denmark (E-mail: akn@ramboll.dk - fnj@ramboll.dk)
[(2)] FORCE Institute, Park Alle 345, DK-2605 Broendby, Denmark (E-mail: osk@force.dk)
[(3)] Arminox, Fabriksvej 6, Moensted, DK-8800 Viborg, Denmark (E-mail: bbe7850@vip.cybercity.dk)

Stainless steel is becoming cheaper, although still today (1998) 5-8 times more expensive than uncoated carbon steel (black steel), see Table 5. Therefore, a both economical and technical attractive approach, may be to substitute carbon steel with stainless steel in critical areas, such as the lower section of a column on a highway bridge exposed to de-icing salt, the splash zone for coastal structures, or an edge beam on a highway bridge. This is named ”intelligent use”. See Figure 1, which shows some of the potential critical areas. Additionally, stainless steel may have a higher strength than traditional carbon steel. These aspects should be considered to design slender structures combining the additional strength with the reduction of concrete cover on the stainless steel.

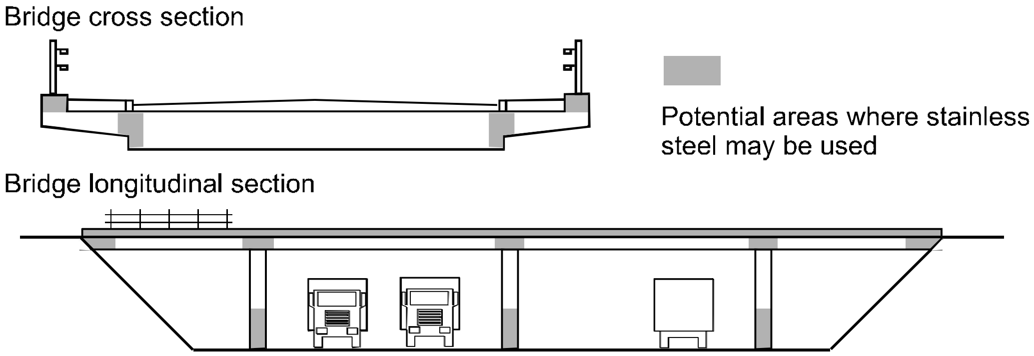

Figure 1 Potential areas where stainless steel may be used intelligently for repair - and in new structures as well.

The cost-effectiveness of the intelligent use of stainless steel will be demonstrated in two examples where stainless steel is compared to traditional carbon steel and cathodic protection. Other repair methods such as surface protection, corrosion inhibitors, etc. are not evaluated in this paper, primarily because they are not realistic options due to the severe deterioration in the examples considered. Based on the two examples, a series of additional sensitivity studies have been carried out to document that the use of stainless steel is beneficial. Experiments have been performed to test the combination of stainless and carbon steel in repair.

USE OF STAINLESS STEEL

Material Aspects

Stainless steel refers to a group of steel with a minimum of 12% chromium. In principle, stainless steel may be divided into 4 major groups - martensitic, ferritic, austenitic, and duplex - all including a large number of alloyed members. The two cheapest alloy groups - martensitic and ferritic steel - will not be evaluated as they do not offer sufficient protection from a corrosion point of view and the price advantage of these steel types, compared to austenitic and duplex types, is continuously decreasing. Austenitic and duplex types are getting more cost-competitive and they provide high corrosion resistance. The stainless steel characteristics are shown in Table 1.

Table 1 Stainless steel types/characteristics.

Type	% chromium	% nickel	% molybdenum
Ferritic	12 - 19,5	0	0
Austenitic	18 - 26	8 – 21	2 - 4
Austenitic / ferritic (duplex)	21 – 28	4 – 6	1,5 – 6,0

Stainless steel rebars are produced with the strength and dimensional properties to meet the requirements of structural concrete codes of practice. Stainless steel bars need only be scaled proportional to carbon steel (with respect to strength) to provide full corrosion and bond strength. Small bar sizes in the 3-16 mm (~1/8-5/8 in.) diameter range are either produced by cold drawing or from 18-40 mm (~6/8-1 5/8 in.) diameter bars by hot rolling.

Austenitic steel, which is resistant to corrosion in concrete with a very high chloride content, and which is the recommended material for rebars, increases in strength when cold or hot worked. The high ductility of stainless steel allows the material to be bent to a tight radius without initiating cracking.

Several investigations have confirmed that stainless steel is much superior to carbon steel in its ability to resist chloride initiated corrosion when embedded in concrete [2], [3] [4] [5] [6]. So far most of the stainless steel used as reinforcement has been of the austenitic types (AISI 304 and 316), which are most readily available and have been shown to have a 5-10 times higher chloride tolerance compared to carbon steel reinforcement [2].

Duplex stainless steel, which has a ferritic/austenitic microstructure, combines a high material strength with increasing resistance to corrosion due to a high molybdenum content, [12]. The commercial grades of stainless steel rebars considered in this paper are shown in Table 2 and Table 3. The recommended type of stainless steel to be used depends on the environment, see Table 4.

Table 2 Stainless steel rebar grades. Cold drawn: 3-16 mm. (Aust. AISI 304/316, Dupl. SAF2205).

Grade (EuroCode /American)	**Chemical Composition**	**0.2% strength** [N/mm^2]	**Tensile strength** [N/mm^2]	**Elongation** A_y	**Elongation** A_{10}
W.1.4301, AISI 304	18Cr, 8Ni	Min. 500	600-800	Min 3%	>15%
W.1.4401, AISI 316	18Cr,10Ni,2Mo	Min. 550	600-800	Min 3%	>15%
W.1.4462, SAF2205	26Cr,8.5Ni,4Mo	Min. 650	800-1000	Min 3%	>15%

Table 3 Stainless steel rebar grades. Hot rolled, 18-40mm.

Grade (EuroCode/ American)	**Chemical Composition**	**0.2% strength** [N/mm^2]	**Tensile strength** [N/mm^2]	**Elongation** A_y	**Elongation** A_{10}
W.1.4311/AISI 304LN	18Cr, 8Ni	Min. 450	600-750	Min 3%	>15%
W.1.4429/AISI 316LN	18Cr,10Ni,2Mo	Min. 500	600-800	Min 3%	>15%
W.1.4462/SAF2205	26Cr,8.5Ni,4Mo	Min. 650	700-850	Min 3%	>15%

Table 4 Stainless steel rebar grades: Typical working environments, [12].

Grade	**Typical working environment**
W.1.4301/AISI 304 /1/LN	Inland, low-chloride environments.
W.1.4401/AISI 316/ /1/LN	Coastal and high-chloride environments
W.1.4462/SAF2205 Duplex	High strength and high-chloride environments

The resistance to corrosion of stainless steel is indicated by the pitting resistance equivalent number (PREN = %Cr + 3.3 %Mo + 16 %N). PREN values are expressed in Table 5.

Table 5 Stainless steel rebar grades: Price comparison (June 1998) and PREN values.

Grade	Price comparison	PREN
Unalloyed steel	1	<1
W.1.4301 / AISI 304	5.5	19
W.1.4311 / AISI 304 LN	5.5	19
W.1.4401 / AISI 316	6.5	25
W.4429 / AISI 316 LN	6.5	25
W.1.4462 / SAF2205	8.5	34

Practical Aspects

The manageability of stainless steel on site is comparable to normal carbon steel, therefore no special precautions need to be taken when using stainless steel. Due to the high cold working properties of stainless steel, somewhat higher bending forces are necessary. For repairs comprising selective replacement of carbon steel with stainless steel in a limited area three methods can be used to connect the stainless steel and carbon steel reinforcement: Traditional unwelded laps, welded laps and mechanical couplers.

The diameter of the main reinforcement is typically in the 15 to 40 mm (~5/8-1 5/8 in.) range, which requires a minimum grip length (~ lap length) of more than 50 cm (~2 ft) at both ends. Therefore unwelded lap joints are not a very competitive option since an additional 1 – 1.5 m (~3.5-5 ft) of concrete is be removed.

Stainless steel bars are weldable on site, but often the weldability of the existing carbon bars is questionable and in some cases unknown. Therefore welding on site may not always be possible.

The corrosion resistance of stainless steel is lowered by welding and by contamination with iron deposits from tools used in handling [7]. However, problems may be avoided by careful post-treatment, e.g. sandblasting and pickling.

Mechanical stainless steel couplers between the carbon steel bars and the stainless steel reinforcement is an alternative to welding. Some of these require that a thread be made on the existing carbon steel, which may be both difficult and time-consuming on site. Another option is to use couplers, which mechanically lock the bars to the coupler, thereby achieving strength higher than the yield strength of the rebar itself. By using mechanical steel couplers, no additional lap length is required. The mechanical couplers may be made using stainless steel. The examples in this paper assume that the mechanical couplers described above are used.

Corrosion Aspects

Stainless steel freely exposed to seawater may, if in galvanic contact with a less noble metal such as carbon steel, initiate a galvanic type of corrosion of the less noble metal. The corrosion rate will depend on the area ratio between carbon and stainless steel. The otherwise slow cathodic oxygen reduction at the stainless steel surface is a catalyst for bacterial slime, which forms after few weeks in seawater.

When stainless steel is cast into concrete, however, the cathodic reaction is a very slow process, since no such catalytic activity takes place on a stainless steel surface. A research project recently conducted at the FORCE Institute [8] 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.

As a consequence, connection between stainless steel and carbon steel should not promote significant galvanic corrosion. As long as both metals are in the passive conditions their potentials will be more or less the same when embedded in concrete. Even if there should be minor differences in potential, both carbon and stainless steel can be polarised significantly without serious risk of corrosion. It is because their potentials will approach a common value without the passage of significant current. Therefore assuming the correct use of stainless steel, which means at all positions where chloride ingress and subsequent corrosion might occur, the two metals can be coupled without any problems.

This behaviour and the fact that stainless steel is far less effective cathode in concrete than carbon steel, makes stainless steel a useful reinforcement material for application in repair projects. When a part of the corroded reinforcement, e.g., close to concrete cover is to be replaced, it could be advantageous to use stainless steel instead of carbon steel. Because of being a poor cathode, the stainless steel should minimise possible problems, which will occur in neighbouring corroding and passive areas after repair.

At the same time it is very important for the intelligent use of stainless steel that it is combined with carbon steel in proportions which guarantee both optimal performance and cost-effective solution. For this reason, tests, which include probable combinations of volume between stainless steel and carbon steel aimed for repair of damaged highway and coastal bridges, have been created as described later in examples a) and b).

EXPERIMENTAL TESTS

The aim of the described experiments in this paper is to define objectives for use of stainless steel in repair of corroding reinforcement. The galvanic couple that is formed between the passive stainless steel and the existing carbon steel, which in some cases is passive and in some cases is corroding, will be studied in order to prove that use of stainless steel for this purpose might even have a beneficial effect.

All test samples have dimensions 300x170x70 mm (~12x7x3 in.) (Length x width x height) and are cast from an ordinary Portland cement concrete of w/c ratio = 0.5 and without addition of fly ash and microsilica. All samples contain 5 reinforcement bar pieces in full sample length. These bars are either made of carbon steel or austenitic stainless steel (AISI 316). Additionally, the test samples contain two small pieces of the austenitic stainless steel or carbon steel, which correspond to 5% to 10% of the total steel volume. The 5% and 10% chosen for the samples represent the percentage of stainless steel to be used in example a) (10%) and example b) (5%). All bars have a diameter of 6 mm (~1/4 in.). In each sample, a reference electrode of the MnO_2 type is embedded. A total of 10 concrete samples are cast and they are divided into groups as follows:

Group 1: Three concrete samples, each containing five carbon steel rebars and two short (6 mm (~1/4 in.)) stainless steel rebars corresponding to 5% and 10% of the total volume of the steel in the sample. The stainless steel rebars are closest to the concrete surface, which will be exposed to the aggressive environment. The carbon steel rebars are located behind the stainless steel, at different but defined depths from the exposed concrete surface.

Group 2: One reference concrete sample containing six pieces of carbon steel rebars located as the rebars in the samples of group 1.

Group 3: Three concrete samples, each containing the same number and volume of carbon steel and stainless steel as the samples of group 1. The only difference is that all rebars are located at the same depth from the exposed concrete surface (cover is approximately 20 mm (~3/4 in.)) to ensure that both the stainless steel and carbon steel are exposed under identical environmental conditions with more or less the same oxygen access.

Group 4: Three concrete samples, each containing five pieces of stainless steel rebars and two short pieces of carbon steel rebars corresponding to 5% and 10% of the total volume of steel. All rebars are located at the same depth from the exposed concrete surface.

Figure 2 and Figure 3 show the above-described samples and the principle of measurements. Figure 4 and Figure 5 show photos of the test setup.

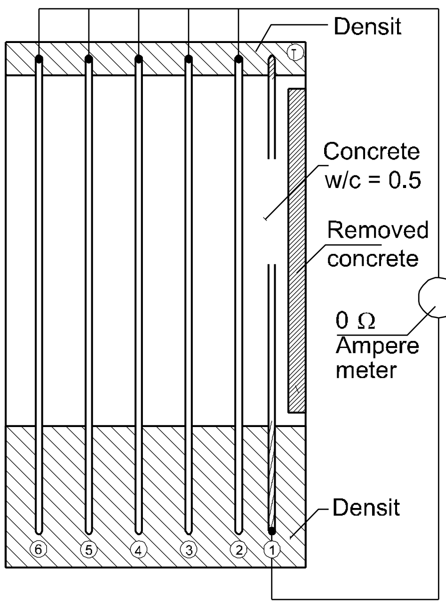

Figure 2 Experimental model. See also Figure 4 and Figure 5.

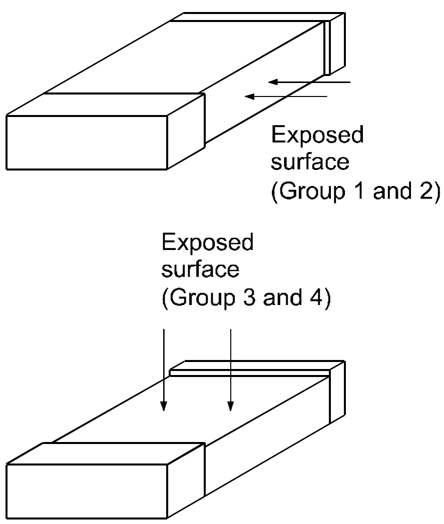

Figure 3 Exposed area for groups 1 and 2 (top) and for groups 3 and 4 (bottom).

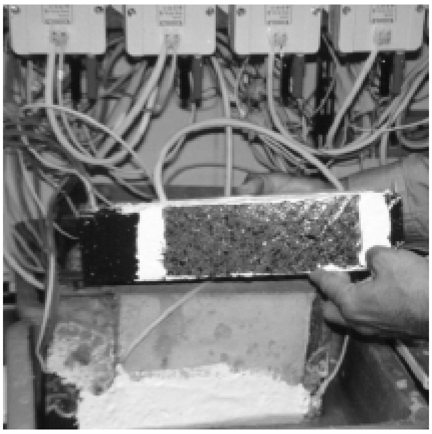

Figure 4 Experimental setup for group 1 and group 2.

Figure 5 Experimental setup for group 3 and group 4. The dark areas are red corrosion products.

One month after casting, all samples were exposed in a concentrated solution of NaCl (165 g NaCl/litre) with addition of $Ca(OH)_2$. In order to accelerate the chloride ingress this exposure is a cycle of two days wetting in the NaCl solution and five days drying in the laboratory atmosphere [9]. The following measurements are conducted:

- macrocouple current between one of the short rebars (usually the one corresponding to the 5% of the total steel volume in the sample) connected to the remaining five rebars. The macrocouple current is measured by means of a specially constructed Zero Ohm Ammeter.
- electrochemical potential of the above mentioned macrocouple by means of an embedded MnO_2 reference electrode.

A significant increase of a macrocouple current when the corrosion process starts is caused by the rapid potential drop of the corroding metal. Thus an electromotive force between two metals with different electrochemical potentials is created and it results in the electrical current (corrosion current) flowing between them, Figure 6.

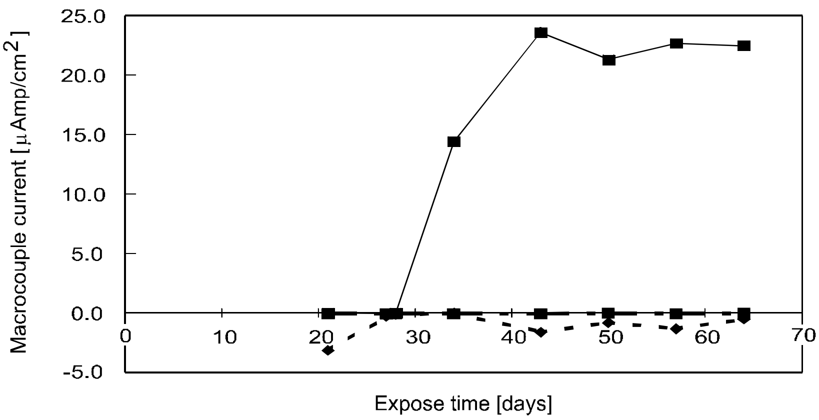

Figure 6 Macrocouple current as function of exposure time for test specimens in group 3, where one specimen has started to corrode, and two other specimens are still passive and therefore the measured macrocouple current for these two specimens is close to zero.

The increase of macrocouple current after initiation of corrosion depends on the type of the passive material (cathode). The current will be much lower when the corroding carbon steel is connected with a passive stainless steel, compared to the current registered between active and passive bars of carbon steel. For this reason, the increase in corrosion rate on carbon steel due to galvanic coupling with stainless steel will be significantly lower than in the case of carbon steel.

The first experimental results, which are shown on Figure 7, confirm this behaviour. When the current is measured between the carbon steel rebar which start to corrode and a small rebar (5%) of carbon steel which is still passive, a value of a current density of approx. 4.3 $\mu A/cm^2$ is registered. If the same corroding carbon steel rebar is connected to the small rebar (5%) of stainless steel, the measured value of current density is reduced to only 0.27 $\mu A/cm^2$. It means a reduction of current density by approx. a factor 15, which will result in the same decrease in the corrosion rate.

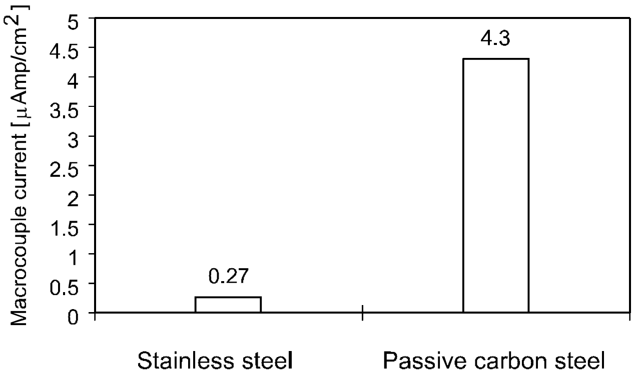

Figure 7 Macrocouple current for stainless steel and passive carbon steel.

For repair work, the combination of corroding carbon steel and stainless steel is therefore more beneficial (with respect to corrosion rate) than replacement of the corroding rebars with a new carbon steel rebar connected to the old and locally corroding reinforcement.

TOTAL COST AND UPDATED LIFE-CYCLE COST ANALYSIS (ULCCA)

In this paper the intelligent use of stainless steel is evaluated by analysing two examples of deteriorated standard reinforced concrete (RC) columns:

a) Highway bridge: RC columns carrying an overpass, and
b) Coastal bridge: RC columns (splash zone).

The rehabilitation of a RC column in these two examples can be analysed independent of the remaining part of the bridge This can be done because the administration, inspections, maintenance, rehabilitation, etc. normally are carried out independently from the rest of the bridge.

The two types of repair will be analysed using the net present value method taking into account all costs (direct and indirect) from the time of repair and onwards. This updated life-cycle cost analysis (ULCCA) will consider all relevant financial and technical aspects. An "U" for "updated" is added to "LCCA" since the life-cycle cost analysis starts when the structure is e.g. 30 or 40 years old and is showing signs of serious deterioration.

The comparison of different strategies – with and without the use of stainless steel - using the net present value method is carried out to determine the repair strategy which is economically optimum for society as a whole, given the premises at the time of decision. This includes taking all costs into consideration: repair, maintenance, administration and indirect cost to society (traffic alterations). This method is a generally accepted method which is approved in Denmark and several other countries.

Example a) repair of highway RC columns of an overpass over a highway

Due to chloride ingress and subsequent pitting corrosion a large number of overpasses have severely damaged RC columns and require either major repair (replacement of reinforcement) on the lower 1.5 – 2.0 m (~5-7 ft.) of the column or a complete replacement.

In this example a 30-year old bridge with several (18) slender columns (see example in Figure 8) with serious corrosion on the reinforcement on the lower 2 m (~7 ft.) of each column is considered. The extensive corrosion requires at least a 50% replacement of the reinforcement at the lower 2 m. The amount of reinforcement to be replaced is 10% of the total reinforcement steel in the column and foundation. Several repairs similar to the one described have previously been carried out in Denmark using carbon steel.

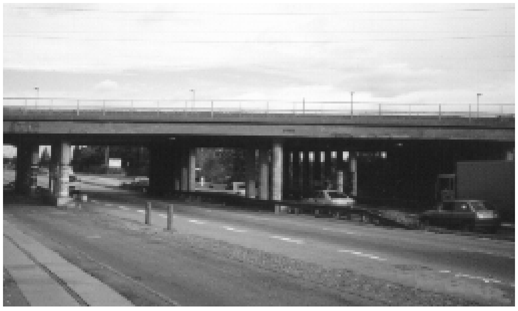

Figure 8 Example a) **:** Typical highway RC columns requiring repair or replacement after 20-30 years.

Figure 9 Example b) : Typical Danish coastal RC columns which may require repair after 20-40 years.

The lifetime for the bridge as a whole is set to 80 years, i.e. a remaining lifetime for the ULCC of 50 years. Four repair strategies are considered for the bridge, see Table 6. The proposed strategies represent typical strategies for similar repair projects in Denmark in the last decade. In Table 7 the 4 strategies are analysed using the net present value method using the cost of repair/replacement as stated in Table 6. The costs of repair and traffic alterations are based on average values from repair projects in Denmark, price level 1998.

As a supplement, when comparing the strategies, a sensitivity analysis is performed for each strategy, taking into account the uncertainties related to both the costs, the time and extent of repairs. Based on previous experience each repair cost and the time of repair is modelled using a distribution function. Using these distributions the uncertainty (here using the coefficient of variation ~ the standard deviation divided by the mean is used for comparisons) on the total net present value can be estimated using simulation.

Table 6 Description of repair strategies. The underpass has 30,000 vehicles daily. Future increase in traffic is ignored.

Strategy	Description of repair strategy
1	**Repair** of all columns using **carbon steel** after 1 year. The repair is done over 2 m (~7 ft.) of each column involving the breaking up of the concrete to behind the reinforcement and replacement of 50% of the reinforcement. This repair takes 12 weeks with 4 lanes narrowed down to 2. After 20 years, the columns are replaced. This replacement takes 16 weeks.
2	**Replacement** of all columns after 10 years and again after 40 years using **carbon steel**. Both replacements take 16 weeks.
3	Repair of all columns using **stainless steel** after 1 year. Same repair as strategy 1 but with replacement of 100% of the existing carbon steel reinforcement with stainless steel. The repair takes 16 weeks. The 4 additional weeks compared to 1 are due to replacement of 50% more steel end establishment of connections.
4	Repair of all columns using carbon steel and installation of a **cathodic protection** system after 1 year and replacement of all columns after 25 years.

Table 7 Net present value analysis of all four major strategies. All values are in 1,000 US $ (100 US $ = 650 DKK, 1998). The value of the maintenance costs shown in the table is included for each year until other maintenance costs are listed in the table.

Example a) : RC-columns, Highway Bridge				
Year	**Strategy 1** Carbon steel Repair year 1 Replac. year 20	**Strategy 2** Carbon steel Replac. year 10 Replac. year 40	**Strategy 3** Stainless steel Repair year 1	**Strategy 4** Carbon steel Rep. + Cath. year 1 Replac. year 25
	Repair / Traffic Maintenance	Repair / Traffic Maintenance	Repair / Traffic Maintenance	Repair / Traffic Maintenance
1	88 490 -	- -	108 585 2	76 408
5	- 4.5	- 11	- 2	- 4.5
10	- 4.5	445 655 -	- 2	- 4.5
15	- 4.5	- 4.5	- 2	- 4.5
20	445 655 -	- 4.5	- 2	- 4.5
25	- 4.5	- 4.5	16 30 -	445 655 -
30	- 4.5	- 4.5	- 3	- 4.5
35	- 4.5	- 4.5	- 3	- 4.5
40	- 4.5	445 655 -	- 3	- 4.5
45	- -	- -	- -	- -
50	- -	-295	- -	-74 -
Total net present value (Repair & Maintenance +Traffic) for given discount rates – and the coefficient of variation. Bold marks the lowest (i.e. best) value of the four strategies.				
0%	1890 - **0.07**	2170 - **0.07**	**830** - 0.11	1780 - 0.09
3%	1320 - **0.09**	1290 - **0.08**	**750** - 0.12	1090 - 0.11
5%	1090 - **0.10**	980 - **0.10**	**730** - 0.12	860 - 0.13
7%	940 - **0.11**	780 - 0.13	**720** - **0.12**	**720** - 0.15
10%	800 - **0.12**	**580** - 0.18	700 - **0.12**	620 - 0.17

As expected the initial cost of using stainless steel is higher than using carbon steel or cathodic protection. The total cost after 1 year alone using stainless steel (strategy 3), is

20% higher than repair using carbon steel (strategy 1) and 42% more costly than installing cathodic protection (strategy 4).

However, when comparing the total net present values (see Table 7 and Figure 10) stainless steel is an attractive option. Despite the high initial cost of using stainless steel, it is seen that for discount rates below 7 % strategy 3 is still the most economical. Between 7 and 8 % cathodic protection and stainless steel are equal, and for discount rates above 8 % postponed repair and cathodic protection are the favourable solutions.

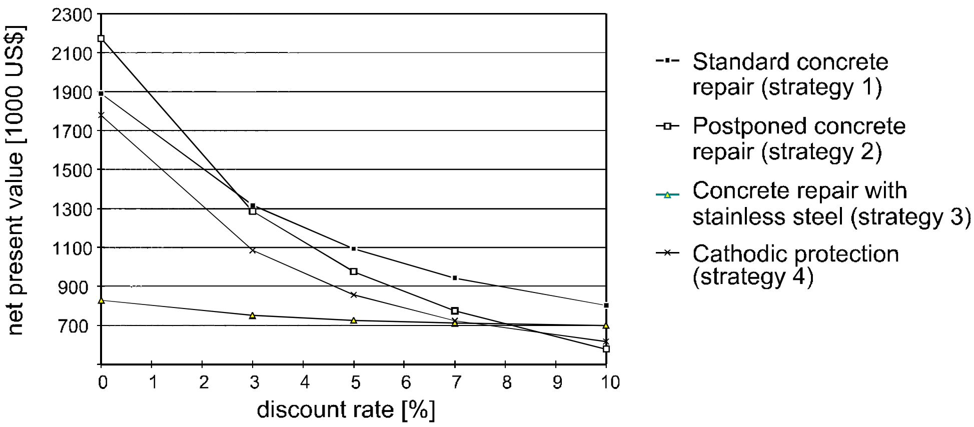

Figure 10 Net present values (50 years remaining lifetime) for different discount rates.

In Table 7 the coefficient of variation (c.o.v.) is shown next to the total net present value. This value is found from simulation of the total net present value using a set of representative distributions for the time and cost of repair and cost of traffic alterations. The time of repair is typically modelled using a uniform distribution around the expected time of repair, and using a normal (or skew) distribution with a c.o.v. ranging from 0.05 to 0.30 for a given repair cost. The distributions are chosen to represent the variation in time of repair and in the corresponding cost.

As seen in Table 7, the uncertainty (here using the c.o.v. as a measure) associated with using stainless steel is slightly higher or equal compared to strategy 1 and 2, but lower than strategy 4 (cathodic protection). This is primarily due to the fact of a higher c.o.v. associated with the initial cost of using stainless steel.

The uncertainties associated with a given strategy may influence the choice of strategy. For strategies with comparable net present values some end-users may want to implement the strategy with the smallest risk of a future unexpected rise in cost. For discount rates between 5 and 7% (which is often used when selecting rehabilitation strategies) it is seen from Table 7 and Figure 10 that using stainless steel is a favourable rehabilitation strategy when compared to the three standard repair methods.

Example b) repair of RC columns of a coastal bridge (splash zone)
An ongoing research project financed by the Danish Road Directorate, [19], has shown that some fairly new coastal bridges need repair in the splash zone due to chloride-induced corrosion, see example of a coastal bridge in Figure 9.

A 30-year-old coastal bridge with 12 columns with severe corrosion of the reinforcement in the splash zone is considered in this example. The repair requires replacement of the outer layers of steel reinforcement due to heavy corrosion. The replacement is in the splash zone, i.e. from 0.5-1 m (~1.5-3.5 ft.) below to 2 m (~7 ft.) above normal sea level. The repair requires that approximately 50% of the outer layer of reinforcement be replaced in this area. The amount of steel to be replaced is approximately 5% of the total reinforcement steel in column and foundation.

The three strategies proposed for rehabilitation of the coastal bridge are shown in Table 8, and the corresponding net present value analysis is shown in Table 9 and Figure 11.

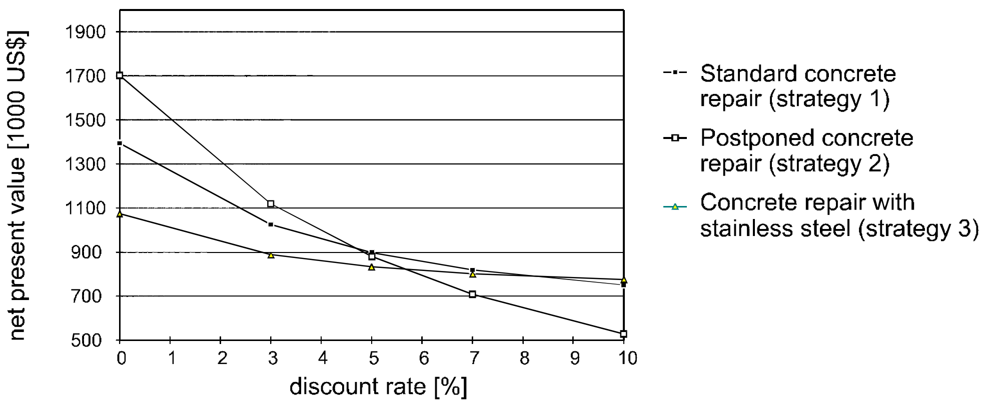

Figure 11 Net present values (50 years remaining lifetime) for different discount rates.

The life-cycle cost analysis shows that for discount rates between 5% and 7% the three analysed strategies are having comparable net present values. Based on this it seems that a postponed repair strategy using stainless steel will be cost-optimal. Taking the uncertainties into account, it is seen that using stainless steel - strategy 3 – is associated with a higher c.o.v. compared to both strategy 1 and 2. It must be noted here that the experience – and the available data for extent of deterioration and associated repair cost - for repair of coastal bridges is lesser than that for repair of highway bridges. This is due to the higher number of repairs performed on highway bridges compared to the number of repairs performed on coastal bridges.

Based on this example, the use of stainless steel for the described type of repair may not be recommended as the preferable choice of repair method in favour of traditional repair using carbon steel. For coastal bridges a better option may be to use stainless steel in selected areas from the time of construction, that is using stainless steel in a new structure.

Table 8 Description of three repair strategies for a coastal bridge.

Strategy	Description of repair strategy
1	**Repair** of all columns using **carbon steel** after 1 year. The repair is done over 2.5-3 m (~8-10 ft.) of each column involving the breaking up of the concrete to behind the first layer of reinforcement and replacement of 50% of the reinforcement. (The columns in the example have two layers of reinforcement – only the outer layer is likely to corrode). At 20 and 40 years minor repair is required on the columns.
2	**Repair** of all columns using **carbon steel** after 10 years. The repair is done over 2.5-3 m (~8-10 ft.) of each column involving the breaking up of the concrete to behind the first layer of reinforcement and replacement of 80% of the reinforcement. At 25 and 45 years minor repair is required on the columns.
3	**Repair** of all columns using **stainless steel** after 1 year. Same repair as strategy 1, i.e. only the outer layer of old carbon steel reinforcement is replaced with stainless steel. At 20 and 40 years minor repair is required on the columns.

Table 9 Net present value analysis of the three strategies. Same organisation as Table 7.

Example b) : RC-columns, Coastal Bridge			
Year	**Strategy 1** Carbon steel Repair year 1 Min.rep. year 20/40	**Strategy 2** Carbon steel Repair year 10 Min.rep. year 25/45	**Strategy 3** Stainless steel Repair year 1 Min.rep. year 20/40
	Repair / Traffic Maintenance	Repair / Traffic Maintenance	Repair / Traffic Maintenance
1	635 -	- -	730 -
5	- 6	- -	- 3
10	- 6	1015 -	- 3
15	- 6	- 6	- 3
20	320 -	- 6	80 -
25	- 6	335 -	- 3
30	- 6	- 6	- 3
35	- 6	- 6	- 3
40	320 -	- 6	125 -
45	- -	320 -	- -
50	-150 0	-235 -	- -
Total net present value (Repair & Maintenance +Traffic) for given discount rates – c.o.v. from the simulation. Bold is used to mark the lowest (i.e. best) value.			
0% 3% 5% 7% 10%	1395 - **0.07** 1025 - **0.07** 900 - **0.08** 820 - **0.08** 750 - **0.09**	1705 - 0.07 1120 - 0.08 880 - 0.08 **708** - 0.09 **528** - **0.09**	**1075** - 0.11 **890** - 0.12 **835** - 0.13 800 - 0.14 775 - 0.14

CONCLUSIONS AND RECOMMENDATIONS

The cost-effectiveness of the intelligent use of stainless steel for new structures has been demonstrated previously, [13]. This paper analyses the cost-effectiveness of the intelligent use of stainless steel for repair of 25 to 30-year-old concrete columns on highway and coastal bridges, by comparing life-cycle costs for different repair strategies with and without the use of stainless steel.

Additionally, experiments have been performed to examine the corrosion aspects when connecting stainless and carbon steel in concrete.

Corrosion Experiments

Experiments with the combination of stainless and carbon steel have shown indications that from a corrosion point of view, it involves no extra risk of corrosion on the carbon steel. Therefore the experiments show that for repair work, the combination between corroding carbon steel and stainless steel is more beneficial than replacement of the corroding rebars with a new carbon steel rebar connected to the old and locally corroding reinforcement. The increase in corrosion rate on carbon steel due to galvanic coupling with stainless steel will be significantly lower than in the case of carbon steel.

The experiments continue and full results will be published separately. It is expected that only carbon steel will corrode under the described accelerated chloride exposure conditions. This will be examined by means of the destructive test and visual inspection, after conclusion of the current investigation. The corrosion current (measured as a macrocouple current), that flows between corroding carbon steel and passive stainless steel is expected to be approximately one magnitude lower than in the case of corroding and passive carbon steel.

Total Cost And Updated Life-Cycle Cost Analysis

The two examples analysed - a highway and a coastal bridge - are comparable in the sense that concrete repairs involve additional costs besides the actual cost of repair. For the highway bridge the traffic flow must be altered, resulting in additional road user cost. For the coastal bridge, concrete repairs are more costly since the repair may require a dry dock and a special raft around columns. Due to this both the number and extent of repairs should be avoided in order to minimise the life-cycle cost. Using stainless steel means a higher initial cost due to the higher material cost of stainless steel, but lowers the requirement for future maintenance, inspection and repair.

As a result of the analyses presented in the paper, stainless steel may be considered a cost-optimal alternative when life-cycle cost is considered, especially for the repair of RC-columns on highway bridges.

REFERENCES

[1] Kilworth, S.R., Fallon, J., "*Stainless Steels for Reinforcement, development of the paper, " Fusion Bonded Coated Reinforcement in the Gulf*", International Conference on Corrosion and Protection of Reinforced Concrete, Dubai, 1994.

[2] Sorensen, B., Jensen, P.B., Maahn, E., "*The Corrosion Properties of Stainless Steel Reinforcement*", from the book "*Corrosion of Reinforcement in Concrete*", ed. Page, C.L., Treadaway, K.W.J., Bamforth, P.B., Elsevier Applied Science, 1990.

[3] Treadaway K.W.J., Cox R.N., Brown B.L., "*Durability of Corrosion Resisting Steels in Concrete*", Proceedings of Institute of Civil Engineers., Part 1, 1989.

[4] Flint, G.N., Cox, R.N., "*The Resistance of Stainless Steel Partly Embedded in Concrete to Corrosion by Seawater*", Magazine of Concrete Research, Vol. 40, No. 142, 1988.

[5] Pastore, T., Pedeferri, P., Bertolini, L., Bolzoni, F., "*Electrochemical Study on the Use of Duplex Stainless Steel in Concrete*" Proceedings of the International Conference Duplex Stainless Steels 92, Beaune Borgogne, France, 1991, pp. 905-913.

[6] Haynes, J.M., "*Stainless Steel Reinforcement*", Civil Engineering, 1984, pp. 23 and 27-28.

[7] Nürnberger, U., Beul, W., Onuseit, G., "*Corrosion Behaviour of Welded Stainless Reinforced Steel in Concrete*", Otto-Graf-Journal, 1993.

[8] "*Materials for Corrosion Cell Cathodes*", Internal Report by the Danish Corrosion Centre as a part of a report regarding Brite/Euram Contract 102 D. 1990.

[9] Nordtest standard, "*Concrete testing, Hardened Concrete. Chloride Penetration*" NT BUILD 443, 1996.

[10] Elsener, B., Klinghoffer, O., Frolund, T., Rislund, E., Schiegg, Y., Böhni, H. , "*Assessment of Reinforcement Corrosion by Means of Galvanostatic Pulse Method*", Proceedings International Conference on Repair of Concrete Structures, Svolvaer, Norway, 1997, pp. 391-400.

[11] Nürnberger, U. (ed.), "*Stainless Steel in Concrete - State of the art report*", European Federation of Corrosion Publications, Number 18, 1996

[12] Cochrane, D. J., "*The role of stainless steel in preventing corrosion in reinforced concrete structures*", Proceedings of the 13th FIB Congress, 1998.

[13] "*Life-cycle Cost Case Study: River Crossing Highway Bridge (Schaffhausen Bridge, Switzerland)*", Published by Euro Inox, 1997.

[14] Poulsen, E. "*Rustskader fra chlorid, Reparation af beton* (in Danish)", Arkitekt og Byggebladet, April 1998.

[15] Abbott, C. J., "*Corrosion-free concrete structures with stainless steel*", Concrete, Vol. 31, No. 5, May 1997.

[16] Bertolini, L, et. al. ,"*Behaviour of stainless steel in simulated concrete pore solution*", British Corrosion Journal, Vol. 31, No.3., 1996.

[17] McDonald, D.B. et al., "*Stainless Steel Reinforcing as Corrosion Protection*", Concrete International, May 1995, p.65-70.

[18] Pedeferri, P., "*Behaviour of Stainless Steels in Concrete*".

[19] Stoltzner, E., Knudsen, A., Birit, B., "*Durability of Marine Structures in Denmark*", Proceedings International Conference on Repair of Concrete Structures, Svolvaer, Norway, 1997, pp. 59-68.

The Intelligent Choice

ARMINOX® stainless steel reinforcement - The maintenance free and the most economical solution for construction all over the world.

ARMINOX® Stainless

Arminox® A/S
Jernvej 22, Mønsted
DK-8800 Viborg, Denmark
tlf. +45 86 64 50 11
fax +45 86 64 57 11
arminox@arminox.com
www.arminox.com