

COMITE EURO-INTERNATIONAL DU BETON

COATING PROTECTION FOR REINFORCEMENT

This State of the Art Report was prepared by Task Group VII/8 of Permanent Commission VII: 'Reinforcement: Technology and Quality Control'.

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STATE OF THE ART REPORT

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Foreword to the 1992 edition, CEB Bulletin 211

Among the activities of the Permanent Commission VII 'Reinforcement: Technology and Quality Control' the subject of reinforcement protection systems was considered to be of great interest.

This was for two reasons. The first was the need for a technical survey of the most common protection systems, which are at present rapidly finding new uses. The second reason was to evaluate the practical effect of these systems.

The Task Group VII/8 was established with these objectives. It has concluded its work with the state-of-the-art report presented here.

Professor J. Calavera, *Chairman of PC VII*

J.D. Holst, *Convenor of TG VII/8*

Foreword to the 1995 edition, 'Coating protection for reinforcement'

This report, produced by CEB as Bulletin 211 in 1992, was taken up by Thomas Telford Services Ltd for formal publication. It should be noted that the protection systems referred to here only cover a limited range, those that were principally of interest at the time that the Task Group was operating.

Naturally, development in protection systems was, and still is, going on and in due course the CEB will again be considering the latest developments.

Roy E. Rowe, *President of CEB*

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1. Introduction

1.1. General

Reinforced concrete has been regarded as a durable construction material. There are many examples of structures which have performed well and consequently have represented an economic solution for the owners. Lately, however, the durability of concrete has been a topic of general concern. An increasing number of damages are being reported, many due to corrosion of the reinforcing steel. Reasons for this to have become more prominent recently might be

- (a) the environment is more polluted and aggressive
- (b) utilization of higher material strengths has resulted in more slender design
- (c) economic pressures have led to a reduction in construction time, often at the expense of good workmanship
- (d) cement characteristics have changed.

CEB has taken this problem seriously, publishing some relevant bulletins.¹⁻³

1.2. Corrosion mechanisms

Below, sections are quoted from reference 3 (comments from the Task Group are added in square brackets).

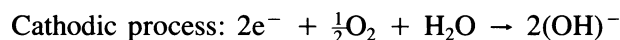
‘Steel in concrete is protected against corrosion by passivation. This passivation is due to the alkalinity of concrete: the pH-value of the pore water runs up to $\text{pH} > 12.5$. In the case of such high pH-values, a microscopic oxide layer is formed on the steel surface, the so-called passive film. This passive film impedes the dissolution of iron. Thus, corrosion of reinforcement is impossible, even if all other preconditions for corrosion are fulfilled (mainly the presence of moisture and oxygen).’

‘Due to carbonation of the concrete or by the action of chloride-ions, the passive film may be destroyed locally or over greater surface areas. A third mechanism is a reduction of alkalinity due to the leaching out of alkalis by streaming water. In practice, this may happen in the region of weak points of the structure (e.g. leaky construction joints, wide cracks) in combination with bad concrete quality (gravel pockets, high w/c-ratio). If the pH-value of concrete drops below 9 at the reinforcement, or if the chloride content exceeds a critical value, the passive film and the corrosion protection will be lost. Consequently, corrosion of reinforcement is possible, if sufficient moisture and oxygen is available. This can be assumed to be the case for structures in the open air.’

‘As a simplified model, the corrosion process can be separated into two single processes, the cathodic and the anodic process. The anodic process is the real dissolution of iron. Positively charged iron ions pass into solution:



The surplus electrons in the steel will combine at the cathode with water and oxygen to form hydroxyl ions:



After some intermediate stages, the iron and hydroxide ions will combine to form rust which, at least theoretically, can be written as Fe_2O_3 (under practical conditions, rust products are more or less water-containing compounds). This means that only oxygen is consumed to form rust products. This oxygen must normally diffuse through the concrete cover towards the reinforcement. Water is only necessary to enable the electrolytic process to take place.

[The Task Group does not completely agree to this last sentence.]

As a consequence of the interrelations described, corrosion will not occur either in dry concrete (where the electrolytical process is impeded) or in water-saturated concrete (loss of oxygen), even if the passive layer at the surface of the reinforcement has been destroyed. On the other hand, the highest corrosion rate will occur in concrete surface layers, subjected to highly changing wetting and drying conditions.

In the anodic areas, the passive film must be destroyed; the cathodic process, however, can take place even if the passive layer is intact.

In the case of chloride-corrosion, this effect causes the so-called pitting corrosion, because the passive layer will be dissolved only over small surface areas, so that small anodic areas and huge cathodic areas will exist on the surface — a fact that causes substantial local reductions in sections of the reinforcement. In addition, the chloride ions will act as a catalyst in the pit and accelerate the dissolution of iron in the anodically acting pit.

At the steel surface, anodically and cathodically acting areas may be situated either close together (micro-cell corrosion) or at locally separated places (macro-cell corrosion) even over relatively great distances.

Consequently, corrosion may occur in areas of the structure where the direct access of oxygen to the surface of the reinforcement is impeded if the concrete is wet enough to render the electrolytical connection possible.'

[The Task Group finds the last two paragraphs represent a controversial point. Although micro-cell action is the most common mechanism supporting the corrosion process, sometimes macro-cells may be developed. Whether the action of these macro-cells is significant or not, depends on the ohmic resistance between anode and cathode and the ratio between cathode and anode areas].

1.3. Economic considerations

The costs of a structure consist both of the construction costs for erecting a new structure and the costs involved in keeping it serviceable in a safe manner during its lifetime.

The life of a structure can be divided into four phases.²

- Phase A:* The structure is rather new. Durability is achieved by design, construction, curing, protective coatings, quality control.
- Phase B:* Some carbonation has taken place and may be chlorides and/or other aggressive agents have penetrated into the cover. Durability can then be bettered by cleaning, coating and/or impregnation, local increase of cover by shotcreting.
- Phase C:* The carbonation front has reached the reinforcement at a number of spots. Corrosion has started locally and the cover has started to spall. To ensure continued use during the desired lifetime, repair and maintenance is required: removal of bad concrete, use of repair mortar or shotcreting, coating.
- Phase D:* The reinforcement has started to corrode in large areas. The cover has spalled off in many places, and cracking is occurring on a large scale. Major repair and renovation is necessary, involving
 - (i) almost complete removal and replacement of the cover
 - (ii) demolition, removal and replacement of parts of the structure.

As the amount of repair and maintenance work increases heavily from phase A to D, so also do the costs. As a rough indication, not to be taken too exactly,

the 'law of fives' can be introduced. This states that: '1 dollar spent in phase A, equals 5 dollars in phase B, equals 25 dollars in phase C, equals 125 dollars in phase D'. In short, this means that a certain lifetime may be achieved by some extra attention to 'good engineering practice' in phase A, costing one dollar per unit area. But waiting until phase D has developed, the repairwork to achieve the same lifetime would cost roughly 125 dollars. The conclusion: it is more economical to spend some added resources in phase A when building a structure and by this to delay or avoid the onset of reinforcement corrosion.

1.4. Different protection methods

To reduce the risk of corrosion of the reinforcement, several methods are advocated. Some methods are based on the properties of the concrete, others attack the problem by protecting the reinforcement.

1.4.1. Bettering the concrete

- (a) Concrete cover. Carbon dioxide, as well as chloride, penetrates to the interior of concrete at a rate given approximately by a square-root time function, that is, if the thickness of the cover of a good quality concrete is doubled, the time to reach the critical state for incipient danger of corrosion will increase more than four-fold. Then, it is supposed, serious cracking does not appear.
- (b) Impermeable concrete. The permeability of concrete regulates the inflow of aggressive agents. Lower permeability is achieved by using a low water/cement ratio, good compaction, good curing and a sufficient cement content.
- (c) Inhibitors. The addition of chemical inhibitors to the concrete can prevent steel corrosion. However, there is still some work to do before laboratory results can be applied in practice. Uncertainty prevails as to the correct dosage and the long-term effect of inhibitors.
- (d) Surface sealants. The idea is to seal-off the concrete surface against ingress of oxygen, humidity and agents. The problem is to be sure the sealants are durable and are not worn down or punctured. Neither must the sealants crack due to movement of the concrete. Then a local attack may result.

1.4.2. Protection of the reinforcement

- (a) Other reinforcing material. The use of non-corrosive materials as ordinary reinforcement has, to the Task Group's knowledge, not yet reached any practical solution. Even if aramids have been used as prestressing reinforcement, the strain is regarded as too large for ordinary reinforcement.
- (b) Stainless steels. Austenitic stainless steels have been used with success technically, but the price is prohibitive for general use.
- (c) Cathodic protection. Either sacrificing anodes are used or a system of impressed current. By sacrificing anodes the electrical connection to the reinforcement must not be broken, and the anodes have to be renewed. With the system of impressed current there are problems in acquiring good electrical contact and regulating the current. Both methods are in use, and practical applications may be developed further.
- (d) Protective coating of the steel. One method is to give the steel a sacrificing coating to protect the steel against corrosion, as with hot-dip galvanizing. Another method is to give the steel an insulating barrier coating which restricts contact of the steel with aggressive environments and inhibits the electro-chemistry of the corrosion process. Such coatings are fusion-bonded epoxy and PVC. Both are in widespread use.

1.5. Scope of this report

This report will be limited to the last of the methods described above. Both hot-dip galvanizing and epoxy- or PVC-coatings are covered. In this report the term 'reinforcement' is used to mean ordinary reinforcement, as prestressing steels are not included. The report is based on results reported in the literature and on the experience of the members of the Task Group. No special tests were performed by the Task Group as considerable time is required to achieve practical results.

2. Hot-dip galvanizing

2.1. Manufacture: influences on materials and application

In a corrosive medium, zinc coating retards the onset of steel corrosion if the zinc creates stable protective films. This reduces wastage of the normally unstable zinc (passive corrosion protection). In concrete the galvanized coating first dissolves in the alkaline pore water to create a protective layer of high durability. Zinc corrosion is reduced to a very low rate.

In a chloride-containing concrete, zinc may corrode completely in localized areas. Then the steel may be protected because of cathodic protection by sacrificial dissolution of zinc (active corrosion protection).

2.1.1. *Manufacture of the coating*

In the hot-dip galvanizing process, cleaned bars are immersed in a bath of molten zinc at about 450°C. This process produces a durable, resistant layer of metallic zinc and zinc–iron alloys bonded metallurgically to the steel base, completely covering the steel bar.

The manufacturing process may vary locally, but in general the work is carried out in the following way: Thorough preparation of the base material is essential. Soluble organic compounds on the bar are removed in a hot alkaline solution based on caustic soda. Some types of paint, as epoxy, must be removed mechanically, for example by sand blasting. After a water rinse, the bars are then descaled by pickling in either hydrochloric or sulphuric acid.

Pickle control is an important aspect. Care is necessary to ensure complete removal of rust and mill scale without excessive attack of the underlying steel. Since scale is never uniformly thick and continuous over the surface of the bar, there will be sections that are more resistant to pickling than others, and thus require a longer treatment. Over-pickling of the cleaned areas is avoided by the use of an inhibitor in the acid.

A further water rinse follows to free the surface of residual acid and iron salts. Then the bar is dipped in an aqueous preflux solution of zinc ammonium chloride. The flux solution removes the oxide film which forms on the highly reactive steel surface after acid cleaning, and prevents further oxidation before galvanizing. The bars are dried and are ready for galvanizing.

When the bars are submerged in the zinc bath, the two metals react to some depth. After removal and cooling, the coating has several layers of iron–zinc alloys metallurgically linked to the base steel, and an external layer of nearly pure zinc. The sequence of layers from the steel substrate to the exterior surface of the coating is

- (a) gamma layer: very thin and usually not discernible, it has a cubic structure with 21–28% Fe
- (b) delta layer: usually consisting of two layers, an internal ‘compact’ and an exterior ‘barrier’; the structure is hexagonal with 7–12% Fe
- (c) zeta-layer: formed by very asymmetric monoclinic crystals with 5.8–6.8% Fe; their presence diminishes the ductility of the coating
- (d) eta layer: an external skin of almost pure zinc.

The thickness of these layers varies according to the composition of the steel, the temperature of the bath, the composition of the zinc bath and the immersion time.

As to the steel composition, the silicon content is the greatest influence. If this is either between 0.02% and 0.15% or more than 0.3%, the reactivity between steel and zinc is higher and the alloy layers may become considerably thicker. If the carbon content is below 0.3%, it has little influence on the steel–zinc reactions. The usual contents of reinforcing steels, elements such as manganese, chrome, nickel, niobium, titanium, vanadium, sulphur and phosphorus, do not influence the reactions to any considerable degree.

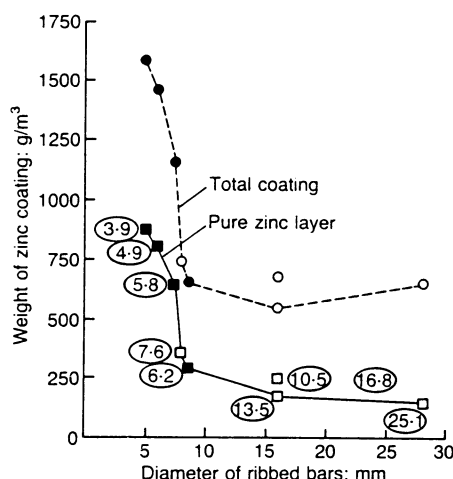


Fig. 2.1. Weight of zinc coating as a function of the diameter of ribbed bars (from Nürnberger) — solid symbols for welded fabric; open symbols for ribbed bars; ringed figures show rib distances in mm

If the temperature of the zinc bath is between 440°C and 470°C, there is no difference in the speed of the steel–zinc reaction worth mentioning. However, as the viscosity of the bath decreases as the temperature increases, the thickness of the coating is reduced. Regarding the composition of the bath, elements such as aluminium may result in considerably thicker alloying layers.

The immersion time is usually between 1.5 and 5 min. In the first minute the thickness of the layers increases rapidly. Later the growth decreases and, finally, is very slow. Thicknesses of iron–zinc layers vary depending on immersion time and silicon content. On average the thickness is usually 50 μm to about 300 μm .

The galvanized bars are withdrawn from the bath slowly and at a uniform rate to allow excess zinc to flow back into the bath. During withdrawal the bars are agitated and their underside scraped with skimmers to eliminate heavy drips and runs. Experience shows that ribbed bars of smaller diameter get thicker coatings of pure zinc, for two reasons: first, thin bars cool much faster, and second, they have shorter distances between their ribs (see Fig. 2.1). Purer zinc is then taken from the bath.

Ribbed bars automatically get an extra-thick coating in the transition zone from the bar mantle to the ribs, especially for small diameter bars (see Fig. 2.1). At normal stresses these transition zones represent critical areas; in bending the strains are concentrated at these spots. Cracks in the layers must be expected, even scaling in thick layers. The immersion time should therefore be as short as possible (about 1 min), and the zinc allowed to drip during withdrawal from the bath. The speed of withdrawal should be about 5 m/min for bars of 10 mm diameter or less, and for larger bars could be 10 m/min.

As a last step, the reinforcing bars are often chromate treated by quenching in a water–chromate solution. The intention is to reduce any reaction between the alkaline cement paste in fresh concrete and the zinc surface during initial curing of the concrete. This treatment may impart a light yellow haze to the surface.

2.1.2. Properties of the zinc coating

The galvanized coating is composed of several layers of zinc–iron alloys and an external layer of pure zinc, as described. In concrete, zinc is effectively passivated for pH-values up to 13.2 ± 0.1 due to the formation of a layer of calcium hydrozincate. This passivating process results in an homogeneous zinc wastage of about 10 μm . If the thickness is not homogeneous, the coating must have a sufficient 'reserve' of pure zinc to avoid the formation of the passivating layer on the alloyed layers. Zinc dissolution from the alloying layers will lead to its disintegration, which weakens the protective character

Table 2.1. Hardness tests

Layer	Hardness
Base steel	159 DPN
Delta layer	244 DPN
Zeta layer	179 DPN
Eta layer	70 DPN

of the galvanized coating. A pure eta-layer (pure zinc) thicker than 10 μm and a total galvanized thickness of at least 60–100 μm is the minimum recommended to provide sufficient protection in concrete.

Galvanizing produces a rather tough coating as the alloying layers show mostly greater hardness than the base steel. Typical results of hardness tests are shown in Table 2.1.

2.1.3. Influences on the steel

The question is whether the process of hot-dip galvanizing influences the properties of the steel itself. As steels are of different grades, of different metallurgical composition and sometimes are cold worked, the possibility of changes cannot be excluded. Due to the usual bath temperatures between 450°C and 470°C and the immersion time of several minutes, ageing and tempering are possible. Ageing increases strength and decreases ductility, while tempering can give the opposite results.

For hot-rolled steels, German tests show that strength and ductility is not influenced to any practical extent. The same applies to Australian tests with Tempcore bars of grade 410. Cold worked ribbed bars tested in Germany showed minor increases in both 0.2 proof strength and ultimate tensile strength as well as in elongation. From Finland a minor decrease of 0.2 proof stress was reported. These changes, however, are regarded as of nearly no practical consequence.

Welded fabrics were tested in Germany. Strength values of galvanized fabrics were found to be practically the same as those of fabrics which were not galvanized and tempered, while elongation values were slightly better for the galvanized fabrics. The strength of the welding points was not influenced.

Cold worked steels with a tendency for ageing might be embrittled after a certain time. Hot-dip galvanizing of such steels will accelerate the embrittlement, and they should be normalized or tempered before galvanizing. Such steels are no longer in common use.

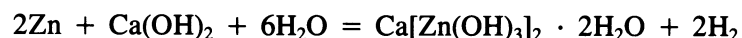
Hydrogen embrittlement is not a problem with normal, low-alloyed reinforcing steels. The hydrogen eventually included through pickling is expelled when immersed in the hot zinc bath. Some quenched and tempered prestressing steels, however, might be embrittled by inclusion of hydrogen. Tests should be carried out before such steels are hot-dip galvanized.

Generally it is recommended that ordinary reinforcing steels with a tensile strength above 800 N/mm² should not be galvanized.

2.1.4. Bond to concrete

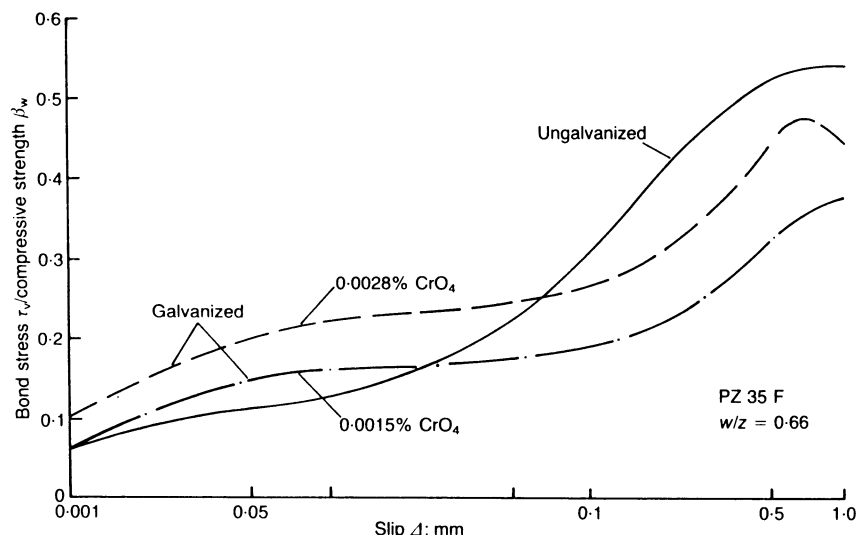
Generally, galvanizing makes the surface of ribbed bars smoother. Several reports have studied the bond behaviour of hot-dip galvanized steel in concrete, for both plain and ribbed bars. The results are rather contradictory, due to the use of different types of tests (pull-out or beam tests), different cements and the different ages of the test specimens.

The galvanized coating will be attacked by the alkaline pore water (calcium hydroxide) of the concrete. Basic zinc hydroxide $\text{Zn}(\text{OH})_2$ is formed and hydrogen released. The zinc hydroxide is immediately converted into calcium hydroxyzincate:



An important aspect of the use of galvanized reinforcement is that the formation of zinc oxide and zincates and the simultaneous hydrogen evolution reduce the adhesive bond at an early age. The formation of zinc oxide and zincates delays the setting and reduces the early strength of the concrete close to the bar. This could delay the stripping of the formwork by several days. The formation of hydrogen bubbles results in a reduction of the bond.

Fig. 2.2. Bond of ungalvanized and galvanized ribbed reinforcing steels BS 420/500 RU (German code), diameter 16 mm, after 7 days (Mang and Müller, 1982)



Through further reactions the zincates disappear, evidently included in the hydration products. Then the strength of the concrete close to the coating surface increases. Therefore the bond strength is time dependent, increasing with time. At the end, setting of the zincates in the concrete might result in an even higher bond than with ungalvanized bars.

The formation of zincates and the evolution of hydrogen seem to occur only when using cements containing small amounts of chromates. If the cement contains 0.022% CrO₄ or more, the process seems to be suppressed. When using cements containing too little chromate, addition of 50–70 ppm of CrO₄ in the mixing water is recommended to avoid hydrogen evolution. Fig. 2.2 shows the effects. However, as some people can develop allergic reactions from contact with chromates, the trend is to reduce the chromate content of cements, while the use of chromate inhibitors is not popular either. In several countries, therefore, after galvanization the reinforcing steel is quenched in a bath of water-chromate. A solution of 0.2% minimum of sodium dichromate in water is used. In this way reaction of the zinc with the alkaline concrete and hydrogen evolution are avoided, resulting in a better bond.

In short, it can be concluded that

- (a) the development of the bond between steel and concrete is dependent both on age and environment. Hence, the time required for developing a full bond is usually longer for galvanized than for ungalvanized bars
- (b) the bonding of galvanized bars may be lower than that of ungalvanized bars when both are of the same age after concreting; but the bond must always reach the level prescribed by standards
- (c) chromation improves the bonding of galvanized bars, reaching even higher values than those of ungalvanized bars under similar conditions.

Under corrosive conditions bond strength is maintained for a longer period by galvanized bars due to the delay in the formation of rust.

2.1.5. Extreme temperatures

The zinc coating can be continuously exposed to temperatures up to 200°C without harm. At temperatures higher than 230°C the zinc crystals anneal through, and there will no longer be any pure zinc layer. This galvannealing takes place over some minutes. At higher temperatures a diffusion process takes place in the coating so that the external zinc layer departs from the iron–zinc layers. The iron–zinc layers, when embedded in concrete, are much more unstable due to a microgalvanic effect. These layers may disintegrate and are less resistant to chloride attack.

2.1.6. Fatigue

German experience indicates that fatigue performance depends on the type of reinforcing steel. The metallurgical composition of the steel might influence the composition of the zinc–iron layers. When subjected to pulsating stresses, these layers crack. Such cracks act as crack initiators at the steel surface. Another possibility is that the temperature of the zinc bath acts as a tempering which may change (improve) the steel quality.

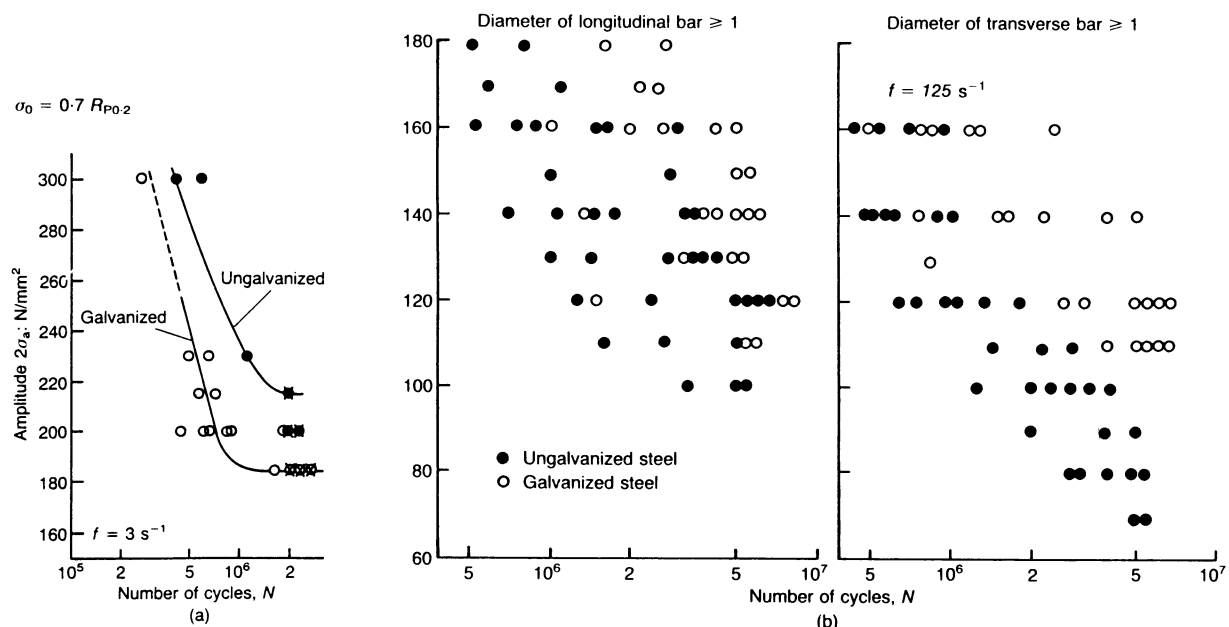
Ribbed hot-rolled bars were tested in Germany as straight bars not embedded in concrete. Galvanized bars showed a decrease in fatigue strength of about 15% compared with ungalvanized bars. For embedded bent bars (dia. of bending mandrel = $15 \times$ dia. of bar) the result was nearly the same. The reductions are most likely due to surface effects. By the pickling process the original roughness of about $25 \mu\text{m}$ of the hot-rolled surface is nearly doubled. Fig. 2.3(a) shows the results for bent and embedded bars.

German tests on welded fabrics also showed that the fatigue performance was better when galvanized (see Fig. 2.3(b)). The reason could be the positive effect of the hot zinc bath in reducing welding stresses.

With galvanized steel the fatigue performance will not differ measurably as long as any zinc coating remains. In general, the reduction that galvanizing can cause in fatigue strength is small compared to the effects of a corrosive attack on non-galvanized steel. Welding usually reduces the fatigue properties. On the other hand, sand blasting improves fatigue behaviour due to compressive stresses introduced in the surface.

Australian research involved testing of both uncoated and hot-dip galvanized bars in concrete beams under dynamic loading both in air, in sea water and in chloride solution. The concrete mix was modified only by the addition of 16 g/m^3 of sodium dichromate to ensure inhibition of hydrogen generation at the bar surface. Comparing the data for galvanized and ungalvanized reinforcement, it was found the galvanized bars improved the fatigue properties of concrete beams in all cases. In the case of galvanized bars in beams tested in air, the existence of long-life regions at higher stress ranges is of importance. But most significant are the improvements in fatigue properties of galvanized bars when beams were tested in sea water.

Fig. 2.3. Fatigue behaviour of ungalvanized and galvanized specimens: (a) bent reinforcing steel BS 420/500 RK (German code), diameter 16 mm, in concrete, (b) welded fabric (Nürnberg)



Bending can take place either before or after galvanizing. If the bars are bent before galvanizing, the coating will give a complete cover. However, in certain cases the bent and cold worked areas may lead to embrittlement of the steel. For practical reasons the bars are usually bent after galvanizing, even if this might cause cracks or, sometimes, flaking of the coating. Therefore, the thickness of galvanizing has to be limited. In Section 2.2.1.4 an upper limit of 200 μm is recommended.

The size of the mandrel used for bending galvanized bars is sometimes prescribed in national standards or codes. Larger bar sizes require larger mandrels to ensure that the coating remains intact. The type of reinforcing steel will also influence mandrel diameters. If the coating is too damaged after bending, it can be patched up with zinc-rich paint.

For welding, all methods used for ungalvanized reinforcement can be applied. The thickness, composition and structure of the zinc coating influence the welding. This could require some alteration in the welding parameters. The welding should proceed at a slower rate than normal, as pores may develop due to evaporated zinc and gases which do not have time to escape before the welded metal solidifies. The problem may be avoided by splitting the parts to be jointed by 1.5 mm when CO_2 -welding, or 2.5 mm when using covered electrodes. As zinc easily alloys with copper, copper electrodes have to be cleaned often.

Loss of the zinc coating locally in the area of the weld should not significantly affect the protection, provided the exposed area is small compared to the surface area of galvanized steel adjacent to the weld. If the damaged area is excessive, additional protection by zinc-rich paint is necessary.

2.1.8. *Handling, repair and safety*

Even if zinc-coating has good abrasion resistance, care should be shown when handling galvanized reinforcement. Some cracking of the coating may occur especially at bends. Small localized cracks up to about 3 mm wide are usually self-healing. Larger damaged areas and cut ends require repair. The application of an organic zinc-rich paint to a cleaned surface is the most rapid and convenient method of repair. The paint should be applied in two coats to provide a total film thickness of 100 μm .

When welding galvanized steel, the fume also includes zinc oxide, giving the fume a white colour. Inhalation of fresh zinc oxide can give 'zinc fever'. The symptoms resemble influenza, and are fever, freezing, headache and nausea with vomiting. In contrast to lead and cadmium, zinc is not stored in the body, but secreted through purgation. Normally zinc fever is over in some hours. There are no known secondary effects. To avoid inhalation of welding fumes, good ventilation is recommended, especially indoors.

2.1.9. *Applicable standards*

Brief extracts from national standards for hot-dip galvanized reinforcement are given below. For more information, refer to the complete standards.

2.1.9.1. *USA standards.* ASTM A 767/A 767M-85⁴ covers hot-dip galvanized reinforcing bars. The standard includes two classes of minimum coating weights, class 1 of 1070 g/m², and class 2 of 610 g/m².

The galvanized coating shall be chromate treated. Either the chromate treatment is performed just after galvanizing by immersing the bars for at least 20 s in a solution at 32°C of minimum 0.2 percentage by weight of sodium dichromate in water, or of minimum 0.2% chromic acid solution. If the bars are at an ambient temperature, the treatment shall be as above except that 0.5 to 1.0% concentration of sulphuric acid shall be added as an activator of the chromate solution. In this case there are no temperature requirements for the solution.

The zinc coating shall have no bare spots. The coating shall be free of

blisters, flux spots or inclusions, dross and acid or black spots. Bars that stick together after galvanizing shall be rejected. The presence of tears or sharp spikes which make the bar hazardous to handle shall be cause for rejection.

If the bars are bent cold prior to galvanizing, minimum bend diameters are given (D from $6d$ to $10d$ as bar size increases). If galvanizing is performed before bending, some cracking and flaking of the coating in the area of the bend shall not be cause for rejection. A note states that the tendency for the coating to crack increases with bar diameter and with the severity and rate of bending.

2.1.9.2. Australian standards. AS 1650-1981⁵ requires minimum average coating thickness of 600 g/m^2 or $84 \mu\text{m}$ for bars 5 mm or greater. This thickness can be increased for particularly severe conditions.

2.1.9.3. German standards. Zulassungsbescheid No. 215 950⁶ regulates the use of hot-dip galvanized reinforcing bars and welded fabric. The maximum temperature of the zinc bath is 460°C . Minimum average mass of the coating shall be 610 g/m^2 ($85 \mu\text{m}$) and in no place shall exceed a thickness of $200 \mu\text{m}$.

The permissible bond stress of galvanized reinforcement compared to ungalvanized is not altered for bars, but for indented wires there is a reduction of about 30% and for ribbed wires about 45%. For galvanized fabric the anchorage lengths are taken as for fabric of plain or indented wires, even if the wires are ribbed.

As for fatigue, for galvanized bars a reduction in stress range of 25% is introduced compared with ungalvanized bars, while for galvanized welded fabric there is no reduction in the rather limited stress range of ungalvanized fabric.

2.2. Performance in concrete environments

2.2.1. In concrete generally

Experience with hot-dip galvanized steel structures exposed to atmospheric environments is that considerably prolonged durability is achieved. The real question is whether zinc coating is stable in the alkaline environment of concrete.

2.2.1.1. Alkaline environment. In fresh concrete the pH-value of the pore water is between 12 and 14. Fig. 2.4(a) shows that the upper limit of the stability of zinc is situated within this range. Tests have established the threshold value at about $\text{pH} = 13.2 \pm 0.1$. Above this value the zinc coating dissolves quickly, with the evolution of hydrogen, until the coating disappears totally. Below the threshold value the corrosion products formed may seal the surface, thus passivating the metal and arresting the evolution of hydrogen (see Fig. 2.4(b)).

Using scanning electron microscopy to observe bar surfaces, it was noted that when the pH-value is 12.6 , the surface is totally covered by crystallized corrosion products during the first one or two days. As the pH-value increases, the size of these crystals also increases. Just above the threshold value of 13.2 ± 0.1 , the corrosion products appear as isolated crystals which cannot seal the surface so perfectly, leaving small zones of the surface without protection. Consequently the metal is not passivated, and the dissolution continues at a high rate.

This is confirmed by tests. Fig. 2.4(c) shows corrosion rates measured 1 h and 33 days, respectively, after immersion of galvanized bars in solutions with pH-values ranging between 12 and 13.6 . After 1 h the corrosion rates agree well with Fig. 2.4(a). After 33 days the solutions which have pH-values below 13.2 ± 0.1 show very low rates as the bars are passivated. Above 13.2 ± 0.1 the rates remain high enough to lead to the total dissolution of the galvanized coating.

If a continuous passivated layer develops during the setting period, tests

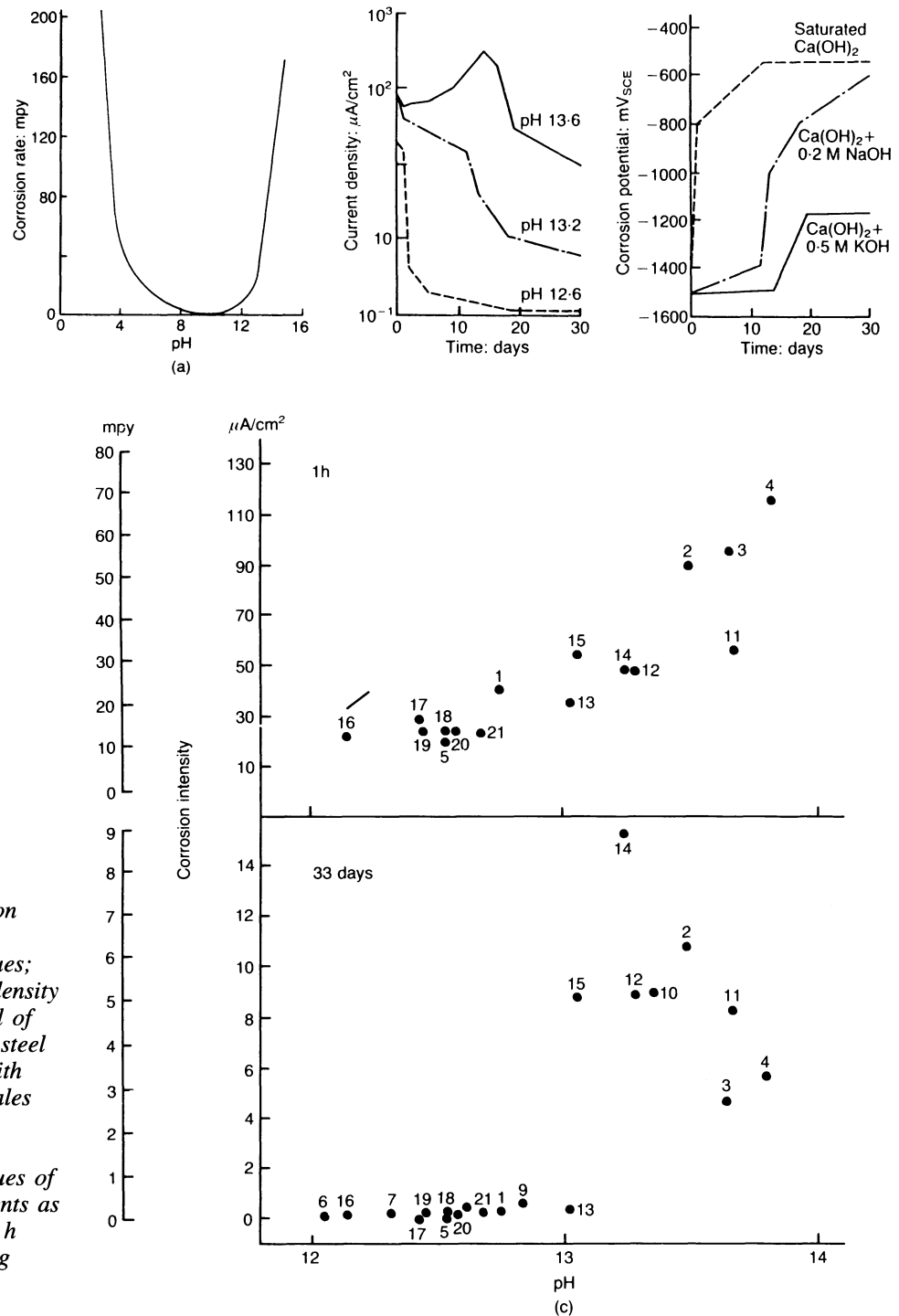


Fig. 2.4. (a) Corrosion rate of pure zinc as a function of the pH values; (b) corrosion current density and corrosion potential of galvanized reinforcing steel in alkaline solutions with pH 12.6–13.6 (Gonzales and Andrade, 1982); (c) variation of the corrosion intensity values of galvanized reinforcements as a function of the pH 1 h and 33 days after being immersed in different solutions

have demonstrated that a further increase of the pH-value does not affect the stability of the passivating layer.

Tests were also made with different cements giving pH-values up to 13.8. The corrosion rates differed by about an order of magnitude. The results indicate that for a cement with the lowest alkali content, a coating of $60\ \mu\text{m}$ would last about 200 years if corroding evenly. With the most alkaline cement the coating would last only 11 years. The corrosion rates seemed to increase approximately with increasing pH-values.

This relation between alkali content and corrosion rate might help to solve the controversy regarding the stability of galvanized reinforcement. It may explain the fact that as most North American cements have low alkali contents, galvanized reinforcement has been mostly successful there.

2.2.1.2. *Humidity of concrete.* The influence of the humidity of concrete on the corrosion of galvanized bars is not clear. However, it seems that dramatic changes in the humidity of concrete affect the passivation process. Curing the concrete at a continuously high relative humidity is recommended.

2.2.1.3. *Corrosion products.* Zinc corrosion products are less expansive and slightly more soluble in the concrete environment than rust, and may therefore diffuse some distance from the metal/concrete interface. In this way the net volume increases and the splitting pressure due to corrosion is reduced. This seems to be confirmed by tests. In general, the amount of cracking is less pronounced with galvanized reinforcement. After 18 months of exposure, one test series resulted in about 10% of the crack area of that of prisms with uncoated bars.

2.2.1.4. *Thickness of coating.* Even if hot-dip galvanized reinforcement is widely used, there are no clear requirements for the thickness of the coating. ASTM requires a coating of 600 g/m² (about 84 μ m) in normal cases, and 1000 g/m² in strongly corrosive environments. The coating thickness should be as large as possible to render good protection, as mentioned in section 2.1.2. But thick zinc layers result in a lesser bond to the base steel when subjected to mechanical stresses. A limit of 200 μ m (roughly 1400 g/m²) is often recommended as an absolute maximum.

2.2.1.5. *Galvanized or ungalvanized.* There is a difference in electric potential of galvanized and ungalvanized steel. In alkaline concrete, steel in contact with zinc tends to accelerate the corrosion rate of zinc. Although, after the protective layer is formed on the zinc surface, this process can be slowed down, the risk of detrimental effects due to the galvanic contact remains. Mixing of galvanized and ungalvanized steel in concrete is not recommended.

2.2.2. *In carbonated concrete*

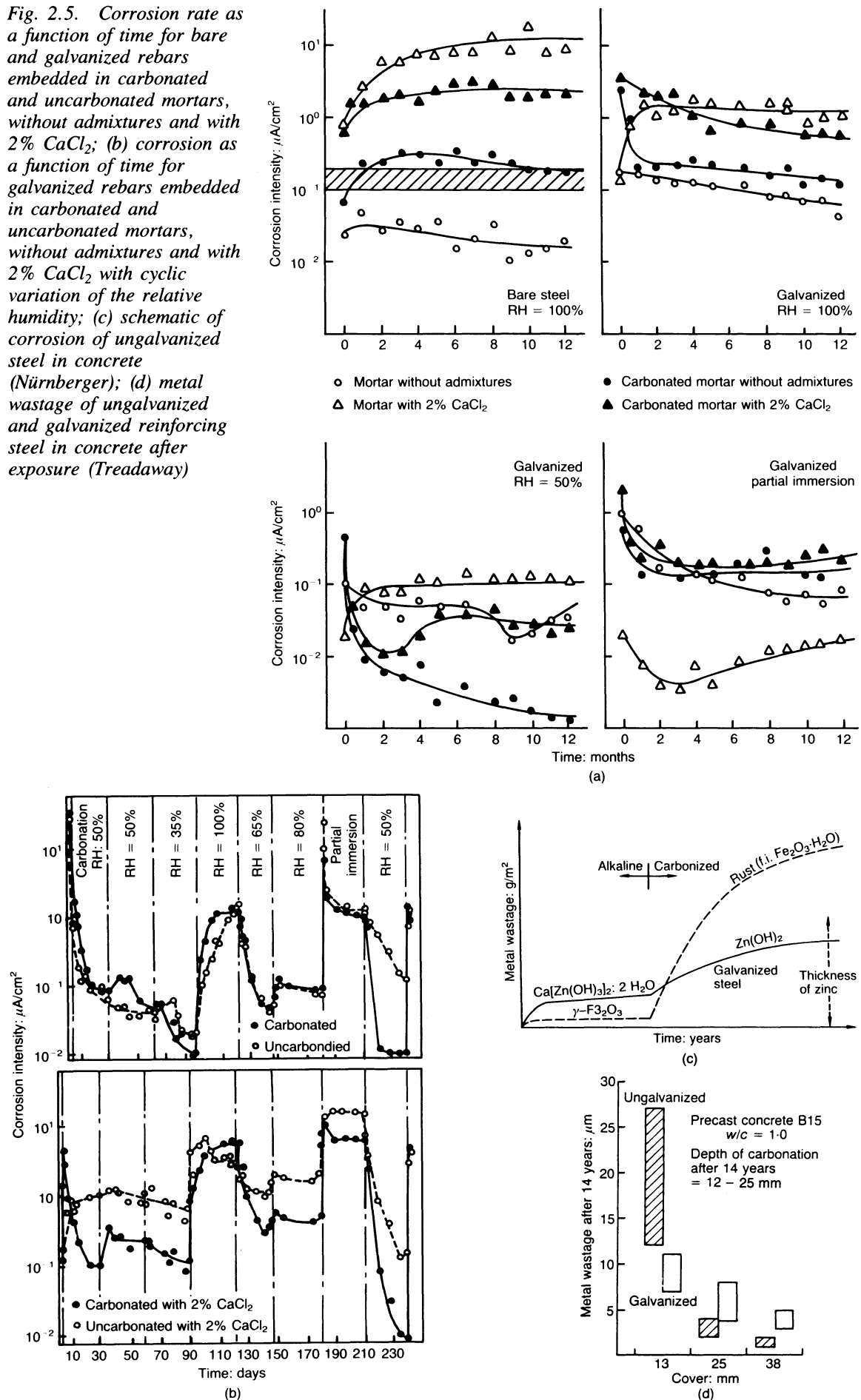
Carbonation of concrete is caused by the reaction of the atmospheric carbon dioxide with the alkaline substances of the pore water and the phases of the hydrated cement (silicates, aluminates and ferrite-aluminates). This results in a reduction of the pH-value of the aqueous solution. When the pH-value drops below the level required for passivation, which is typically about 11.5, and the other parameters necessary for the corrosion process are present (oxygen, water, electric potential), uncoated steel corrodes. The corrosion products lead to the disruption of the concrete cover.

Carbonation may occur throughout the whole mass of concrete. Factors which accelerate carbonation are closely linked to undesirable aspects of mix design and workmanship. An increment of the water/cement ratio from 0.45 to 0.60, will double the rate of carbonation. Bad compaction and curing of the concrete will increase the permeability of the concrete. The presence of cracks and surface blemishes, such as blow holes and honeycombing, will facilitate the access of CO₂ to greater depths.

In good quality concrete of average grade the depth of carbonation, even after 15–20 years exposure, is not likely to exceed 10–15 mm. In dense, high-strength concrete the affected depth could be as little as 2–5 mm. In the absence of chlorides the corrosion will not start until the pH-value has dropped significantly. Consequently there will be a long initiation time, during which the CO₂ not only has to permeate to the reinforcement but also has to consume a large part of the hydroxide reserves in the pore solution.

Figure 2.5(a) and (b) give corrosion rates with time for uncoated and galvanized bars embedded in carbonated and uncarbonated concrete, also in some cases with admixtures of CaCl₂ and with cyclic variations in humidity. Figure 2.5(c) is from German exposure tests of the behaviour of galvanized and ungalvanized steel in carbonated concrete. The conclusions are

Fig. 2.5. Corrosion rate as a function of time for bare and galvanized rebar embedded in carbonated and uncarbonated mortars, without admixtures and with 2% CaCl_2 ; (b) corrosion as a function of time for galvanized rebar embedded in carbonated and uncarbonated mortars, without admixtures and with 2% CaCl_2 with cyclic variation of the relative humidity; (c) schematic of corrosion of ungalvanized steel in concrete (Nürnberg); (d) metal wastage of ungalvanized and galvanized reinforcing steel in concrete after exposure (Treadaway)



- (a) the corrosion rate of galvanized steel is a little higher in carbonated concrete than in alkaline concrete
- (b) the corrosion rate of galvanized steel in carbonated concrete is much lower than that of ungalvanized steel.

Figure 2.5(d) is from British investigations. The carbonation depth after 14 years was 12–25 mm. In the carbonated zone (cover 13 mm) the corrosion loss of galvanized steel was significantly less than that of ungalvanized steel, but higher than in the alkaline zone (cover 38 mm). This corresponds with the results of the German tests (Fig. 2.5(c)).

The conclusion must be that galvanization of reinforcement is beneficial in carbonated concrete both with and without the presence of chlorides. This is due to the range of pH-values over which the zinc coating remains passivated.

2.2.3. *In chloride-contaminated concrete*

The presence of chloride ions (Cl^-) is widely recognized as causing accelerated attack on reinforcing steel. Even at high pH-values, the chlorides depassivate the steel and increase the conductivity of the concrete electrolyte, thereby speeding up the rate of attack.

A distinction should be made between chlorides added in the mix and chlorides which later penetrate from the outside. Those added in the mix are present during the formation of zinc corrosion products. Therefore they may attack the coating before and during the formation of the protective layer. Those penetrating from the outside find this layer already formed and therefore may be less aggressive.

The practical maximum chloride concentration where the corrosion rate really increases, depends both for black and galvanized steel on the properties of the concrete. With a cover of several centimetres, a water/cement ratio less than 0.5 and high humidity, the access of oxygen, and therefore the corrosion intensity is reduced. As a general conclusion it seems to be relevant to set a limit of a content of about 1–1.5% of chlorides by weight of cement for galvanized coating. For ungalvanized steel the values of 0.35–0.40% are usually applied.

2.2.3.1. Chlorides added to the mix. Such chlorides usually originate from contaminated aggregates or mixing water, or from the use of set accelerating admixtures such as calcium chloride. When the concrete contains chlorides from the start, the protective layer of calcium hydroxyzincate on the galvanized coating has to be formed in the presence of the aggressive ions. The zinc is more unstable at higher pH-values. If there is a high pH-value, it is most likely a lower chloride content will be sufficient to initiate the corrosion.

Nevertheless, the most important characteristics of chloride corrosion of galvanized steel are the aspects of the pits and the form of the attack. The pits observed seem to grow on the outside rather than into the zinc coating. A preferential dissolution appears to occur which results in total disaggregation of the gamma-layer at some particular spots. The reason might be that the alloyed layers are less resistant to Cl^- attack than the pure zinc layer. It seems that this eta-layer does not protect the alloyed layers cathodically. In the spots where pitting spreads, the attack may reach the base steel (corrosion rates increase continuously) or may progressively destroy the surrounding coating (alternating periods of relatively high and low corrosion rates).

The most durable zinc coatings are those with a thick external layer of pure zinc. For example, when plain zinc bars are used, very low corrosion rates are measured, although pits may be detected on the surface. In contrast, galvanized coatings are quickly destroyed when they are used in chloride-containing concrete. Therefore, once again, the most resistant part of the zinc coating is the layer of pure zinc, while the alloyed layers form the weakest part.

Regarding the type of cement, the higher the alkali content, the lower the susceptibility to chloride attack, it would be expected. However, calcium aluminates (AC_3) and ferrite-aluminates (AFC_4) react with chlorides. Hence, the corrosion rate will depend on the total amount of AC_3 and AFC_4 , as well as on the alkali content of the cement.

Summing up, galvanized bars are more resistant to chloride attack than bare steel, thus extending the service life of the reinforcement. However, the increase in service life depends on several factors, such as the type of cement, the concrete mix proportions, the quality and thickness of the pure zinc layer, and the humidity of the pores.

2.2.3.2. Chlorides from the outside. Marine environments or the use of de-icing salts are the most common sources of intrusion of chloride from the outside. If the concrete is free from mixed-in chlorides, the intruding chlorides usually meet a continuous protective layer of calcium hydroxyzincate on the surface of the zinc coating.

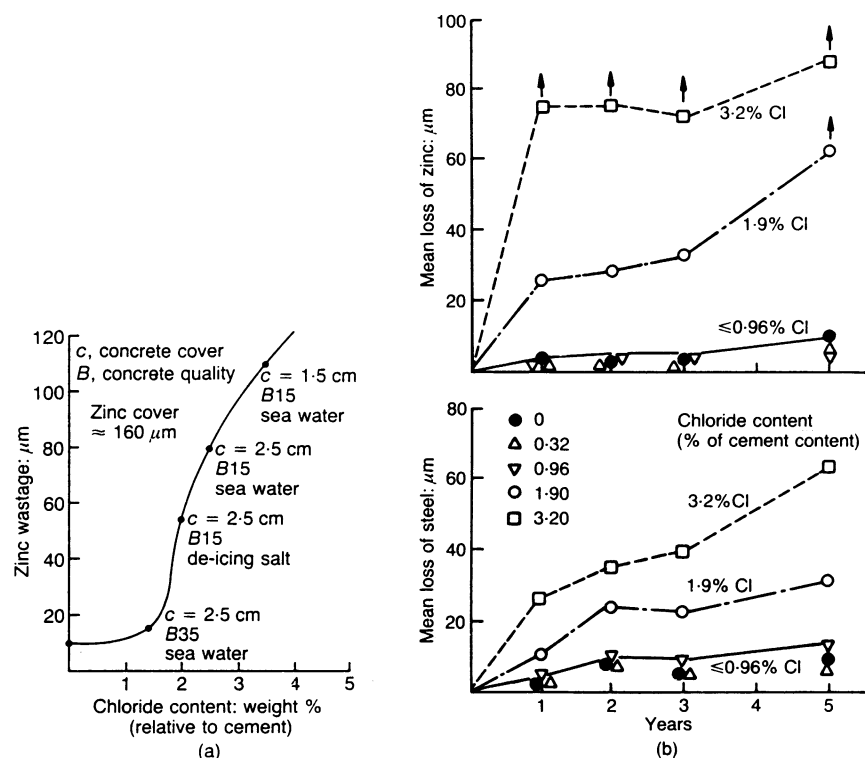
What was mentioned above about mixed-in chloride is mostly valid, but if the zinc coating is to act as a resistant barrier, two factors are important

- (a) the protective layer of calcium hydroxyzincate must be compact and continuous
- (b) the remaining coating must be thick enough to resist pitting if it develops.

Many tests have been performed, and this report can only refer to some of them. The results are contradictory.

Figure 2.6(a) shows loss of zinc in chloride-contaminated concrete in Germany. If the chloride content exceeds 1.5% in the mass of cement, the corrosion rate increases rapidly. Concrete specimens with black or galvanized bars with a cover of 30–90 mm were exposed for 12 years from 0 to 14 m above the lowest sea water level. In areas more constantly wetted by sea water, the corrosion was small. Above the highest water level, where the cover could dry out intermittently, the corrosion was pronounced. Here the intensity of corrosion of the galvanized bars was roughly 75% less than that of the black bars.

Fig. 2.6. (a) Zinc wastage of galvanized reinforcing steels after 2.5 years in chloride-containing concrete as a function of chloride content (Nürnberg); (b) influence of chloride content in concrete on the corrosion of ungalvanized and galvanized reinforcing steel stored in the open ($w/c = 0.6$ and 0.75 ; cover 10 mm and 20 mm; zinc coating about $90 \mu\text{m}$) (Treadaway et al.)



To investigate the effects of the use of de-icing salt on bridge decks, horizontally-placed slabs with black or galvanized bars were subjected for many years to spraying 15 times each winter of 0.5 kg/m^2 of CaCl_2 . With a cover of 1.3 cm , corrosion occurred after two years on the black bars, with scaling after three years. After six years the black bars showed serious local corrosion with a pitting depth of 1.6 mm on the upper part of the bars where the chloride content was 0.4% . The galvanized bars had lost only $5 \mu\text{m}$ of their coating after six years. With a cover of 3.8 cm , about 0.1% chloride was measured after six years, and neither black nor galvanized bars were attacked to any practical degree. Fig. 2.6(b) shows the influence of chloride content on the corrosion.

To compare different tests is not always easy, as researchers use different criteria. Sometimes the results are related to the amount of corroded material, other times to cracking or scaling of the concrete, to electrochemical measurements or to electric potentials. This is especially the case with chloride limits, and the results drawn are rather different.

A general conclusion might be that even in concrete that is highly contaminated with chloride, galvanized steel usually performs well. Should it fail, galvanization has always delayed the onset of the state of active corrosion compared with ungalvanized bars. The use of a low alkali cement (see Fig. 2.4(b)) and a coating thickness more than $100 \mu\text{m}$ is recommended. For concrete structures intermittently exposed to sea water or de-icing salts, galvanizing produces better protection against chloride attacks. Such structures are those which can partly dry out, are of good concrete quality and have a cover of several centimetres. The protection rendered by the galvanization is limited over time.

2.2.4. *At cracks in the concrete or coating*

2.2.4.1. *Cracks in concrete.* Cracks certainly, in principle, contribute to the corrosion process. The sides of cracks will quickly carbonate, and represent 'spearheads' of the carbonation front. Moreover, in chloride-contaminated environments, they represent a quick way for the chlorides to reach the reinforcement. However, the existence of cracks in concrete is no longer regarded as decisive for the occurrence of corrosion, although there is a divergence of opinion concerning the magnitude of crack widths likely to cause corrosion in various environments, mainly in marine ambients.

The indications are that minor cracks which decline to narrow widths at the steel surface are not normally damaging. In a moist environment they tend to heal, providing protection at the base of the crack by alkaline material from the adjacent concrete. Also, transverse flexural cracks do not appear to play any major role in increasing corrosion, except in the case of lean concrete mixes under severe conditions.

The cracks identified as causing the greatest hazard are those with nearly constant width larger than $0.4\text{--}0.5 \text{ mm}$ down through the concrete, or longitudinal cracks running parallel to the reinforcing bars. Fig. 2.7 shows that the corrosion rate of galvanized bars increases at about 0.2 mm crack width at a cover of 2.5 cm , and at 0.3 mm width at 5.0 cm cover.

However, in marine environments the presence of cracks presents a shorter way for the Cl^- ions to access the steel. In the presence of chlorides a crack always represents a corrosion risk. The width of the crack is of minor importance. Whether a particular crack will lead to localized damage or not will depend on other parameters such as concrete quality or cover thickness.

With black steel, crack width is important if the width is larger than 0.3 mm , and the corrosion will increase with increasing crack width. Galvanizing will improve this behaviour, but zinc is not stable. In carbonated cracks zinc will hinder corrosion. In chloride-containing concrete zinc will improve the lifetime.

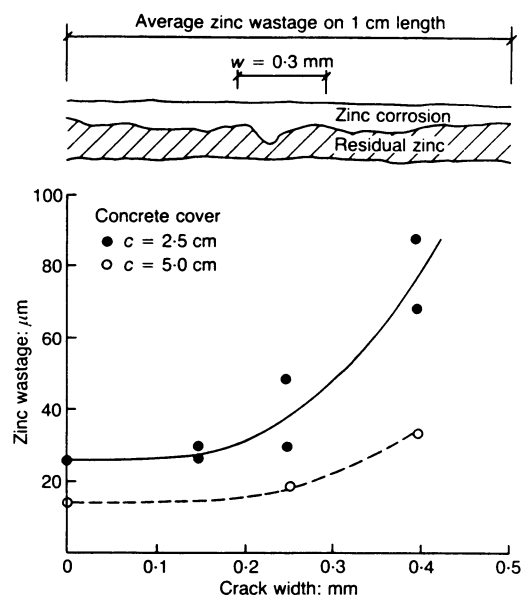


Fig. 2.7. Corrosion of galvanized reinforcing steels in concrete cracks in a seawater splash zone (Nürberger)

2.2.4.2. Cracks in the zinc coating. Regarding the effects of cracks, holes or scaling of the zinc coating on durability, current knowledge does not seem to be sufficient to give a trustworthy answer. At any rate, in chloride-containing concrete no cathodic protection was observed which could protect damaged areas of zinc coating. After local corrosion of zinc, strong pitting corrosion took place.

2.2.5. Long-term performance of the coating

British tests were performed with specimens stored in an industrial environment for 12 and 14 years. They included both normal density and lightweight concrete, with cover between 13 and 51 mm. The reinforcement was either galvanized bars with an average coating thickness of $89 \mu\text{m}$ or black bars. Specimens with black bars were stored for 12 years, and those with galvanized bars for 14 years. Table 2.2 gives the loss of steel or coating in micrometres.

Other tests show that in strongly corrosive environments galvanized bars will delay an early onset of rusting. In carbonated concrete bars with only iron–zinc layers in the coating do not perform as well as bars with a pure zinc layer.

Conclusively, galvanized reinforcement is of advantage compared with black

Table 2.2. Loss of steel or coating

Reinforce- ment	Concrete	w/c ratio	Cement/ aggregate	Depth of carbonation: mm	Losses: μm (at cover: mm)			
					13	25	38	51
Black	Normal	0.6	1:6	0–10	1	1	1	1
	Normal	0.9	1:8	1–8	1	1	1	1
	* Lightweight	1.0	1:4	10–24	12–27	2–4	1	1
Galvanized	Normal	0.5	1:4.5	1–6	1	1	1	1
	Normal	0.6	1:6	1–8	1–2	1	1–2	1–3
	Normal	0.9	1:8	4–6	4–6	1–3	2	2–3
	* Lightweight	1.0	1:4	7–11	7–11	2–10	2–4	2–4

* This lightweight concrete showed longitudinal cracks and scaling. The results indicate that galvanizing is of advantage only in cases where carbonation is fast due to small cover and not so dense concrete.

bars where a quick carbonation of concrete is to be expected. Usually this is the case if the concrete is not dense, as with lightweight concrete, or if the cover is small. Then the zinc coating delays early corrosion, and delays the development of concrete cracks and scaling.

2.3. Practical experiences

2.3.1. *Experiences reported*

Galvanizing of reinforcement was introduced to improve the service life of concrete structures in tropical and marine environments in America.

The first practical use of galvanized reinforcement was reported in 1931 and concerned a dock in Bermuda. The hot and wet marine climate of those islands combined with the use of 'coral' aggregates reduced the durability of concrete structures. As the first experience was successful, the use of galvanized reinforcement was recommended by the Department of Public Works of Bermuda, and consequently was widely used. When the airport was partly demolished after 15 years of service, the galvanized bars revealed good performance.

Due to these encouraging experiences, use of galvanized reinforcement in bridge decks was extended to the USA and Canada. In cold areas in these countries large amounts of de-icing salts are used to improve the safety of roads and motorways. This leads to early corrosion of reinforcement in bridges and reinforced concrete pavements. Even when using a thick, dense concrete cover of low permeability, the bars started to corrode within the first ten years. Such deterioration necessitated premature and very costly repair, even rebuilding of these structures, at a cost of billions of dollars.

Since 1965, nearly 400 laboratory and field studies have been published on corrosion protection by galvanized reinforcement. Only a few can be mentioned here.

In one survey of a number of rather old marine structures, galvanized bars had been exposed to chloride contents as high as 2.2% by weight of cement over a period of 10–20 years with a loss of coating thickness of far less than 10%, and with no record of failure. This should be compared with the knowledge that chloride levels of 0.2–0.3% lead to severe corrosion of black steel in similar circumstances. Often the threshold value of chloride concentrations for depassivation or pitting attack on zinc is 4–5 times the level for attack in concrete on black steel.

Numerous bridges were built in the USA and Canada with galvanized reinforcement. Many were inspected some years after erection to check their performance. Generally it was reported that where black steel was used, there was evidence of corrosion even in the presence of low amounts of chloride. However, when galvanized steel was used, no evidence was found of significant corrosion or concrete distress.

The Portland Cement Association in 1981 began a study of bridge decks in the USA. Five different test procedures were used to obtain data. In short the findings were as follows.

- (a) Long Dick Creek, Iowa, constructed in 1967. In some places only galvanized steel was used, in other places black and galvanized steels were mixed. No active corrosion was found, nor subsurface lamination. Chloride levels approached, in one instance exceeded, the threshold value for black steel. After 14 winters there was no sign of distress, nor evidence of 'coupling' between black and galvanized steels.
- (b) Montpelier, Vermont, 1971. No evidence of corrosion in a high-chloride environment.
- (c) Northbound and Southbound, Pennsylvania, 1973. No evidence of corrosion.
- (d) Bradford County, Pennsylvania, 1973. Galvanized steel throughout the deck. Some areas had cover far below the requirements, and the

concrete here showed unusually high water/cement ratio, and this extremely permeable concrete had very high chloride levels. The combination of these parameters apparently led to a separation of the concrete from the rebars. The Pennsylvania Department of Transportation also examined and tested this bridge as part of its evaluation of 237 decks constructed with galvanized reinforcement. This deck was the only one of the 237 in which any distress was found.

- (e) Columbia, Pennsylvania, 1974. No active corrosion.
- (f) Tioga County, Pennsylvania, 1974. Transverse cracking of the deck led to a variation in zinc coating thickness. No associated distress.
- (g) Dauphin County, Pennsylvania, 1975. Mild corrosion of galvanized coating which might have developed before, or soon after, concrete placement. No corrosive environments were found to exist on this deck.

Together, these eight bridge deck studies and the 237 bridge deck evaluations conducted by the Pennsylvania Department of Transportation uncovered only one area of deck distress (Bradford County). This single incident may be characterized as resulting from defective concrete, permitting extraordinarily high levels of chloride concentration. No failure of the galvanized reinforcing bar performance was revealed in decks constructed in accordance with standard specifications.

In spite of these encouraging results of the performance of galvanized reinforcement, the US Federal Highway Administration did not recommend galvanizing for general application in bridge decks. This was due to some poor laboratory results showing early failure. But after nearly four years of further laboratory testing in the USA and Canada, the poor results have yet to be confirmed. This has led to some controversy.

Among bridges in oceanic environments, three bridges extremely exposed to sea water spray were inspected after 3, 8 and 21 years of service respectively. Ribbed bars with average zinc coating thicknesses of 130 to 250 μm were used. The concrete had a w/c -ratio of about 0.44. With proper compaction such a concrete is usually regarded as sufficiently resistant against intrusion of chlorides. The chloride content in percentage by weight of cement at the reinforcement and the amount of lost coating were measured, and the results are shown in Table 2.3.

In another bridge in much the same environment, the 28.5 cm thick concrete deck had a chloride content of 0.4–0.7% at the reinforcement after three years. In areas with black reinforcement there was a marked corrosion with cracks in the 3.2 to 7.6 cm cover. In areas with zinc-coated reinforcement, the 190 μm coating showed nearly no attack.

Inspection of four structures exposed for several years in the tidal zone gave the results shown in Table 2.4. The chloride content was larger than for structures intermittently wetted. In spite of all the chloride, very little zinc was lost compared to the original coating thickness of 100–200 μm . Only in the case of poor quality concrete (holes and pores) in structure 1, with originally a coating thickness of 100–160 μm , was a complete loss of zinc found after seven years exposure.

In another case galvanized bars with cover of 30–90 mm were mostly intact after 12 years exposure in the tidal zone. There are other examples of structures with galvanized reinforcement in North America, Europe and Australia. In

Table 2.3. Oceanic bridge tests

Bridge	Age: years	Cover: cm	Chlorides: %	Lost zinc: μm
1	3	3.8	0.56	5
2	8	5.7	0.25	5
3	21	5.7	0.45	50–100

Table 2.4. Tidal zone tests

Structure	Age: years	Cover: cm	Chloride: %	Loss of zinc: μm
1	7	13.2	1.4	2
1	10	6.2	2.2	5
1	12	8.0	2.7	13
2	8	7.4	1.6	<2
3	10	5.4	0.8	5
3	10	6.8	1.4	4
4	23	10.5	1.9	5

France, at a metallurgical works at Dunkirk, a coke-quenching tower made of concrete reinforced with black steel failed after two years. After rebuilding with galvanized reinforcement it is reported still to be satisfactorily performing after more than ten years. Due to the poor condition of the black reinforcement, the Onomichi Pier in Japan was reconstructed in 1971 using galvanized reinforcement. No further problems have been reported.

Other examples where galvanized reinforcement is reported to have been successful in use are

- (a) Crocker Building, San Francisco: galvanized rebar in structural elements
- (b) ANDOC North Sea oil rig: 2000 tons of galvanized rebar in sea-bed storage
- (c) Collegiate buildings, University College, London: galvanized rebar and fabric with 19 mm cover
- (d) High Court, National Gallery and Parliament House, Canberra: galvanized rebars in all critical areas to ensure very long life
- (e) Takehara refinery, Japan: 300 tons of galvanized rebars in chimney structure.

To avoid cracking and rust staining, galvanized reinforcements were used in the following thin walled structures and cladding panels

- (a) Staten Island Community College, New York: white precast panels
- (b) Bank of Hawaii, Waikiki: thin decorative precast arches
- (c) National Theatre, London: 1000 tons of galvanized reinforcement in exposed areas such as parapet walls
- (d) Eastbourne Congress Theatre, UK: cladding panels and window mullions
- (e) New Hall, Cambridge: galvanized fabric in 22 mm thick triangular roof segments
- (f) University sports hall, Birmingham: 37 mm thick panels
- (g) Sydney Opera House: 35 mm thick chevron panels for cladding of roof 'sails', also sea wall units
- (h) Hydro-Electricity Commission HQ, Hobart: precast cladding panels
- (i) Telecom Exhibition Exchange, Melbourne: precast cladding panels
- (j) Intercontinental Hotel, Sydney: 1549 precast windows and fascia units with 25 mm cover
- (k) Library Tower, Sydney: external columns and cladding panels
- (l) Parliament House, Wellington: precast cladding panels.

Summing up, galvanized reinforcement has mainly been used in bridge decks. Other structures are cooling towers, chimneys, docks and offshore structures, dams, lamp posts, paper mills, water and sewage treatment works, chemical processing plants and a host of buildings of all types. The reports give the impression of successful results.

However, it is mentioned below: 'There are other structures in North America where galvanized reinforcement has not performed well and required

costly maintenance programmes.' It is a pity no further details are reported. Could the reason be that the works were not carried out according to specifications? Anyhow, it leaves an open question as to the effect of galvanized reinforcement.

As to the benefits from the practical use of galvanized reinforcement, the reports list the 17 points below

- (a) proper galvanizing procedures have no significant effect on the mechanical properties of the steel reinforcement
- (b) for best performance, galvanized reinforcement should be passivated by chromate treatment
- (c) zinc coating furnishes local cathodic protection to the steel, as long as the coating has not been consumed
- (d) galvanized reinforcement provides protection to the steel during storage and construction prior to placing the concrete
- (e) corrosion of galvanized steel in concrete is less intense and less extensive for a substantial period of time than that of black steel
- (f) galvanized steel in concrete tolerates higher chloride concentration than black steel before corrosion starts
- (g) galvanized reinforcement delays the onset of cracking, and spalling of concrete is less likely to occur or is delayed
- (h) the concrete can be used in more aggressive environments. Thus a standard design of concrete components can be retained for various exposure conditions by the use of galvanized steel in the most aggressive cases
- (i) lightweight and porous concretes can be used with the same cover as for normal concretes
- (j) greater compatibility is obtained with low alkali cement
- (k) poor workmanship resulting in variable concrete quality (poor compaction, high water/cement ratio), can easily be tolerated
- (l) accidentally reduced cover is less dangerous than with black steel
- (m) unexpected continuous contact between concrete and trapped water can be tolerated
- (n) repair of damaged structures can be delayed longer than with black steel
- (o) galvanized hardware is acceptable at the surface of the concrete, as it is for the joints between precast panels
- (p) the use of galvanized reinforcement ensures a clean appearance of the finished concrete with no trouble arising at cracks either from spalling or rust staining
- (q) the galvanized reinforcement is cleaner and easier to work with, and makes it possible to consider the use of thinner wires as welded fabrics.

It is important to remember that even if these benefits are achieved, the use of galvanized reinforcement should not be considered as an alternative to the provisions of adequate cover or dense, impermeable concrete, unless special design criteria have to be met. Galvanization of reinforcement is a complementary measure of corrosion protection — a kind of insurance against the inability of the concrete to isolate and protect the steel.

2.3.2. *Extent of use*

Galvanized reinforcement has been used in all countries which consume significant quantities of reinforcement. It seems to be mostly used in Bermuda, the USA, Australia, the UK, the Netherlands and Italy.

In the USA about 2 % of ordinary reinforcement is galvanized. In Europe the estimate is 1 %. In the UK up to about 3000 tons of reinforcement are currently galvanized each year, mostly for specific and sporadic jobs. The yearly consumption of galvanized reinforcement as a percentage of total consumption of ordinary reinforcement is reported for some countries in Table 2.5.

Table 2.5. Consumption of galvanized reinforcement by country

USA	2.1 %
Netherlands	(5 % mentioned as an economical target)
Sweden	0.3 %
Finland	0
Norway	0.1 %
Australia	0.3 % in 1985
	0.7 % in 1986
	2.5 % in 1987
	5.0 % (20 000 tons) target for 1989

In the UK the question is often asked why consumption of galvanized reinforcement is not greater when there are so many examples from the last 8 to 12 years with no problems. The answer might be that there is a reluctance on the part of stockists to handle yet another material with no particular economic advantage, and similarly for the contractor who has to arrange for galvanizing. The specifier is reluctant to increase costs to take care of the effects of bad workmanship or to allow for contingencies which it is hoped never arise. Perhaps the main reason is that steel production and coating are carried out by different organizations, with no properly coordinated plan for approval and supply.

2.3.3. Economy

The cost of galvanizing depends to a large extent on the price of zinc. This cost is now at its highest ever level. In the UK the cost of zinc was £1150 per ton in June 1989.

In the UK, hot-dip galvanizing including chromate treatment of 20 mm ribbed bars in 12 m lengths will most likely cost £566 per ton. The price varies with bar diameter due to the difference in surface area. The trend is illustrated in Table 2.6.

Escalating zinc prices in Norway have also been reported. Converted into US-dollars the prices for galvanizing and chromate treatment per ton of straight bars in 10 m length in February 1989 were as given in Table 2.7. The ratios are very much the same as in the UK. From the USA it was reported that the additional cost for the equivalent treatment was 5 cents/lb, while the steel price was 12 cents/lb. This gives a ratio of 1.4. In total this should increase the cost of a bridge superstructure by about 1 %, but the cost of resurfacing the deck would be about 10 times that of galvanizing all the reinforcement in the bridge deck.

Table 2.6. Cost ratios for hot-dip galvanizing treatment of 20 mm ribbed bars in 12 m lengths

Cost ratio galvanizing treatment/ordinary steel	Bar diameter: mm		
	10	20	40
For straight bars	1.9	1.8	1.6
For bent bars	1.7	1.7	1.6

Table 2.7. Prices of galvanizing and chromate treatment per ton of straight bars of 10 m length

Bar diameter: mm	8	16	32
Ordinary steel	\$459	\$432	\$432
Galvanized steel	\$776	\$388	\$196
Combined treatment	\$1235	\$820	\$628
Ratio combined treatment/ordinary steel	2.7	1.9	1.5

The costs of galvanized reinforcement readily fixed in the formwork will depend not only on the galvanizers' basic price, but in addition the fabricators' mark-up, the length of the bars relative to the galvanizers' kettle, the size of the bar, if the bars are straight or bent, and on other factors as well. To put the costs of galvanized reinforcement into perspective relative to typical building costs, it must be remembered that its use will normally be restricted to severely exposed and to high-risk parts of a structure. Other points to note are as follows.

- (a) In a high-rise building the structural concrete framework typically comprises about 30% of ultimate total costs including finishes, services, etc.
- (b) The reinforcement in slabs, columns and beams is roughly 180 kg/m^3 of concrete. With the price of galvanizing \$300 per ton, this represents a \$54 surcharge per cubic metre of concrete. Setting the price of concrete including formwork, fixing of reinforcement and pouring, as \$500 per cubic metre, the cost of galvanizing increases concrete costs by approximately 11%.
- (c) The influence of galvanizing the reinforcement on the total building costs is no more than 3.3%.
- (d) If galvanized reinforcement is used only in selected parts of the structure, the percentage drops to between 0.5% and 2.5% of total building costs. When related to total project costs or to final selling costs, the percentage is even lower.
- (e) If the numbers above are related to the instantaneous depreciation that occurs when corrosion of reinforcement and spalling starts in a structure, the extra costs of galvanizing become insignificant. This also applies to the costs of repairs, which can be as high as 10% of building costs. Besides, repairs may only eliminate visible damage and rarely represent a long-term cure.

3. Epoxy-coating

3.1. Manufacture: influences on materials and application

The general principle of the protection provided by epoxy-coating is to furnish the steel with an insulating and isolating layer to separate it from aggressive ions and inhibit corrosion. The insulating nature of the coating ensures that any corrosion occurring is reduced (compared with uncoated steel) because of the elimination of available cathodic sites.

3.1.1. *Manufacture of the coating*

The application of an epoxy-coating can be performed in different ways.

- (a) Liquid epoxy is brushed or sprayed on to the steel, or the steel is immersed in the liquid epoxy. Liquid epoxy contains solvents that evaporate after application, resulting in pore formation. In addition, liquid coating shrinks, and prior to hardening the coating material flows from the top of the ribs, resulting in a thin coating at the top of the ribs while the coating layer may be quite thick between them. The Task Group is not aware of any supplier who coats rebar commercially in this way, but it is used for repair works.
- (b) Preheated steel passes a fluidized bed with epoxy powder which melts to the steel surface. Either air is passed through the bed of coating powder so that the powder behaves like a fluid, or the steel passes just over electrically-charged powder which is attracted to the steel. Fluidized-bed dipping of reinforcement is still relatively new. The first commercial plant to coat reinforcement by fluidized-bed dipping is now operational for cages for the Storebaelt Eastern Tunnel in Denmark.
- (c) Electrostatic spraying of epoxy powder to the steel. Electrostatic or tribostatic spraying of straight lengths of rebar currently accounts for the majority of coated rebar. The coating is applied in a stepwise procedure. There should be controls on the type of steel to be coated, especially its rib profile.
 - (i) The steel has to be cleaned of oil and fat, and free from humidity. This is done by abrasive blast cleaning, using steel grit, to a near-white finish SA 2·5. Surface roughness of the steel after blast cleaning is normally 0·05–0·07 mm, giving a better physical anchorage for the coating.
 - (ii) The steel is heated to 235–240°C, for example by passing through an electric induction furnace.
 - (iii) In a chamber, electrostatic nozzles spray epoxy powder on to the steel. The electrically-charged particles are attracted to the earthed steel.
 - (iv) The epoxy powder melts on reaching the hot steel, flows out and gels within 5 s. About 25 s later, the coating has solidified. The time taken for the coating to solidify is normally referred to as the 'gel' time.
 - (v) The coating is cured, usually in air, for a minimum of 28 s followed by a separate water quench to cool the powder below its glass transition temperature, typically around 95°C, before handling the bar.
 - (vi) The coating is inspected for thickness, adhesion of coating, holes, voids, cracks and damaged areas. The existence of small holes is checked using electric detectors. Damaged areas larger than prescribed are patched-up with liquid epoxy. The steel is then bundled and made ready for delivery.

Usually there are strict requirements on the epoxy powder used and on the liquid patching epoxy.

3.1.2. *Properties of the coating*

The purpose of the coating is to isolate and insulate the steel from the corrosive environment. The protective coating works where the problem occurs, at the steel/concrete interface. The coating should protect against external and internal corrosives, should require no maintenance and require little, if any, changes in structural design. To be able to do this, the coating must

- (a) provide easy and economical application
- (b) coat the mantle, the ribs and lugs of the reinforcing bar uniformly
- (c) be tough and adhere well to resist transportation and construction site handling
- (d) be flexible enough to allow easy fabrication of coated bars
- (e) be mechanically stable to sustain normal rebar loading
- (f) resist the actual corrosion environment for the designed lifetime.

To achieve these properties, certain requirements have to be set for the epoxy powder used. In the USA only prequalified powders are allowed for use by the majority of state highway authorities. Prequalification consists of submitting samples of commercially-coated rebar to the 'Valley Forge Testing Laboratories' which tests in accordance with ASTM A775 annex A1.⁷ Properties required are

- (a) the powder shall be of organic composition, except for the pigment which may be inorganic (a special green colour which contrasts with rust and makes it easy to spot attacked areas is preferred)
- (b) the powder shall form a film free from holes, voids, cracks, contamination, damaged areas and holidays. The reinforcing bars shall be uniformly coated over ridges and valleys with small deviations in coating thickness
- (c) chemical resistance when immersed in four different solutions. The coating must not then blister, soften, lose bond, nor develop holidays
- (d) resistance to applied voltage
- (e) sufficient chloride impermeability
- (f) flexibility of coating through bend testing of coated bars. Coating shall not crack on the exterior side of the bent bar
- (g) bond strength to concrete through pull-out testing
- (h) abrasion, impact and hardness testing of coating.

Coating thickness is important. For optimal corrosion protection a thick coating is desirable, but considering the bond between coating and steel and the bond between coated bars and concrete, large thicknesses are unfavourable (see section 3.1.4). A uniformly thick coating without pores and cracks is best. Experience shows that fusion-bonded epoxy coatings render even thicknesses, even across the ribs on ribbed bars. The range of coating thicknesses usually aimed for is between 130 and 300 μm : experience indicates that the optimum balance between coating flexibility and corrosion protection is reached at around 200 μm .

The protective action of epoxy-coating is due in part to its resistance to the passage of aggressive ions and in part to its insulating qualities. The protection provided is also related to the integrity of the coating. The fewer the holes in the coating, the better it will perform.

The epoxy resins have good chemical durability and are very impermeable to chloridic ions. Epoxy-coatings are regarded as providing better protection against high chloride concentrations than hot-dip galvanizing.

The protection against intrusion of corrosive agents depends on a good bond to the steel surface. Correct cleaning and pretreatment is essential, particularly to ensure no contamination with chlorides or other aggressive ions. Mechanically, epoxy-coatings show good resistance against abrasion, and good flexibility if the thickness is not too large. Epoxy-coatings exposed

to sunlight will 'chalk', that is, lose the glossy appearance of freshly manufactured material. This may also be accompanied by slight fading. There is, however, no evidence to indicate any marked deterioration in the properties of the coating.

3.1.3. *Influences on the steel*

No special influences on the properties of the steel have been reported or are known. The reinforcing steel used as raw material for epoxy-coating can have two significant influences on the finished coating product:

- (a) Bars with longitudinal as well as transverse ribs may well prove difficult to blast clean thoroughly and coat at the base of the longitudinal ribs. Experience has shown that longitudinal ribs frequently result in higher levels of pinholes in the coating.
- (b) The bond to concrete of epoxy-coated bars can be sensitive to the rib profile of the reinforcement.

3.1.4. *Bond to concrete*

Bond to concrete may be defined either as the ultimate stress at pull-out, or as a certain slip between bar and concrete at lower stress.

Regarding the ultimate bond stress, it seems the epoxy-coating reduces strongly the adhesive bond to concrete. Plain and poorly ribbed bars have shown large drops in bond when epoxy-coated. It seems that the mechanical bond caused by the ribs has to be relied upon.

The effective rib height of coated bars seems to be the original rib height minus the coating thickness on the core of the bar. Due to the smaller rib heights on the thinner bars, the percentage reduction in rib area will be larger than for greater bar sizes. Consequently, at a constant coating thickness a more pronounced reduction in bond should be expected at smaller bar sizes.

Coating thicknesses larger than 500 μm decrease the bond distinctly. If the thickness is 300 μm or below, the bond values are not far from those of uncoated bars. Reductions up to 20% are reported in more normal cases. In spliced regions without any confining reinforcement, stirrups or ties, splitting failure resulted in 35% reduction.

Regarding slip of epoxy-coated bars, some tests demonstrate a larger slip at lower loads, but the total slip at ultimate load seems to be roughly the same as with uncoated bars.

Other beam tests exhibited about 50% increase in concrete crack width and distance when the bars were epoxy-coated, but the beam stiffness measured by deflections was not affected.

From these tests the reported conclusions are

- (a) epoxy-coating significantly reduces the bond strength; the amount of reduction depends on the mode of failure, pull-out or splitting
- (b) at pull-out failure the bond was approximately 85% of uncoated bars; at splitting failure the bond was about 65% of uncoated bars
- (c) the effect of bond strength can be quite sensitive to bar diameter depending on the type of bar and of the test
- (d) the reduction in bond was insensitive to variations in coating thicknesses from 125 to 300 μm
- (e) the width and spacing of concrete cracks was increased by epoxy-coating; for 19 mm bars, the average width of cracks was nearly twice the width in specimens with uncoated bars
- (f) cracking load and deflections were not significantly affected by epoxy-coating.

Based on this, in the USA rules for development length of epoxy-coated bars have been discussed. Compared with uncoated bars, the requirements are likely to be

- (a) a 50% increase in tension development length when the concrete cover is less than three bar diameters or when clear spacing between bars is less than six bar diameters; the intent is to control potential splitting failures through the cover or splitting between bars in one layer.
- (b) a 20% increase for all other conditions of cover and bar spacing
- (c) the combined factor (increase) for top bar effect and epoxy-coating does not have to exceed 1.7.

More research is being undertaken in the USA to assess the influence of confinement of epoxy-coated transverse reinforcement such as stirrups and ties on the bond strength of coated main bars. However, no information is available from these tests. The reason might be the tests have run into problems as the different rib patterns are not equivalent to bond, and this first has to be sorted out.

The British Cement Association (BCA) reported in December 1990 a series of bond tests comparing uncoated and epoxy-coated bars with diameters of 10 to 40 mm. It studied splitting failure in bond of both bottom and top cast bars in prismatic specimens. As beam tests give a more realistic indication of the bond performance in a structural situation, 4 m long beams with links and lap splices at the maximum moment were also included. To assess the performance of coated reinforcement in resisting shear, beams with either coated or uncoated links were shear tested.

The overall conclusions of these tests when comparing fusion-bonded epoxy-coated reinforcement with uncoated reinforcement were

- (a) there is a 20% reduction in the bond capacity for bottom cast epoxy-coated bars (the reduction would appear to become smaller as the bar diameter is increased)
- (b) for top cast bars there is no significant difference in the bond capacity
- (c) lap lengths of epoxy-coated bars in tension should be increased by 25% in the bottom cast position (it is likely that tension laps in the top cast position need not be altered)
- (d) there is no significant difference in the shear capacity.

As the coated bars showed an increase in bond capacity in the top cast position, this could indicate that the coating improves the compaction around the bar and reduces problems due to plastic settlement.

3.1.5. *Extreme temperatures*

Epoxy-coatings lose their stability at temperatures of about 200°C. The question is how such bars behave when embedded in concrete. Two similar two-way concrete slabs reinforced with ribbed bars of grade 60 (about 400 MPa) were fire tested. One slab had uncoated bars, the other epoxy-coated bars. Fire endurance was about 4.5 h. The slab with epoxy-coated bars behaved well—comparable to that with uncoated bars.

Pull-out tests were performed at temperatures from 110°C to 370°C. Both uncoated and coated 19 mm ribbed bars were used. The uncoated bars withstood 20% higher pull-out loads. However, the test showed that epoxy-coated bars heated to these temperatures developed stresses greater than their nominal yield stress (which is temperature dependent) without pulling out.

In this test, displacements were measured at maximum load both at the free end and at the loaded end. The total displacement reported includes both those caused by slip and those by elastic and inelastic elongation of the bar. For uncoated bars displacement at the free end was generally zero; for epoxy-coated bars it was more significant. Up to 150°C, displacements at the free and loaded ends were of almost the same magnitude. This indicates pull-out of the epoxy-coated bars which may be caused by softening or melting of the coating. At temperatures of 260–370°C, displacements of epoxy-coated

bars at the free end were about half that of the loaded end. This does not necessarily indicate pull-out, but it does indicate some bar slip.

3.1.6. *Fatigue*

The bond to concrete under repeated loading within the service load range has been tested. The test specimens were beam ends where the tension load was applied to the protruding bar and the compression load to the concrete compression zone. The slip of the loaded end corresponding to the greatest and smallest bond stress measured was registered as a function of loading cycles. The test comprised uncoated bars covered with mill scale, epoxy-coated bars, and uncoated sand-blasted bars. Conclusions were:

- (a) under bond fatigue loading in a working stress range, the slip of the three types of bars is essentially the same
- (b) after the first load cycles, epoxy-coated bars showed larger slips at the loaded end than mill scale bars, the difference in loaded-end slip usually diminished somewhat as the number of cycles increased
- (c) the bond strength of epoxy-coated bars, compared with mill scale bars, is not adversely affected by up to 1·4 million cycles of loading in the working stress range. The relative differences based on both flexural bond pull-out strength and the slip criterion was actually slightly lower after the cyclic loadings.

The BCA investigation referred to in section 3.1.4 also included fatigue testing. The 2·5 m long, 125 mm wide and 250 mm deep beams were reinforced with a single 25 mm bar in the tension zone and had 10 mm links at 150 mm centres throughout. The specimens were cast in pairs, i.e. one with coated bars and one without. The beams were tested on a span of 1·7 m with a central load. The two pairs were subjected to different loadings as follows (at about 50 cycles/min):

- beams 1 and 2: 150 000 cycles from 16 to 40 kN
- beams 3 and 4: 150 000 cycles from 26 to 50 kN

The deflection of the beams with coated steel was in the region of 5% to 10% more than with uncoated steel. The crack widths increased with successive cycles, and for the coated bars for the first pair of beams was about 1·6 times, for the second pair of beams about 1·2 times, that for the uncoated bars. The overall conclusion was that there is little difference in the fatigue behaviour, although crack widths and deflections may be slightly higher for coated steel.

3.1.7. *Cutting, bending and welding*

Coating plants are usually designed to coat several straight bars simultaneously. Some plants can coat bent bars, but it slows the operation down as bundles have to be opened and rebundled while identification tags are preserved. For overall economy, the preferred procedure is to coat straight bars and then to fabricate them. When coated bars are cut, the uncoated ends, and sometimes the flaked-off areas close to the cut, have to be repaired with patching compound. Bending of coated bars is best at an operating temperature of 12–15°C. At lower temperatures the coating becomes more brittle. Artificial heating to higher temperatures does not seem to produce significant benefit. To protect the coating, the use of coated bending rollers and mandrels is essential, as are timber faces to tables, bending machines, shear pockets, etc.

Reworking of epoxy-coated bars on site often results in problems and thus is not a recommended practice.

Welding of coated bars may generate noxious fumes from the resin. Therefore, measures should be taken to ensure the safety of the working environment. In gas-pressure welded joints, if the end surface and periphery

of the weld are coated with epoxy-resin, the surface of the weld will not be completely compressed, being left as a flat fracture. Therefore, the coating on the end surface to be welded must be completely removed prior to welding. The static strength of an enclosed arc-welded joint can meet the requirements even when the coating on the end surface is not removed, but remember the noxious fumes!

3.1.8. *Handling, repair and safety*

All coated reinforcing steel should be handled with care. Avoid direct contact between metal and coating. Do not drop or drag coated bars. When unloading coated bars, minimize scraping of the bundles or bar-to-bar abrasion from sags in the bundles. Use lifting devices as spreader beams with multiple pick-up points. Use nylon slings or padded wire rope. Storing should be on timbers with adequate blocking placed between layers of bundles to prevent sagging. Long-term storage (from one season to the next) is not recommended.

Placing of epoxy-coated bars is as for uncoated bars. The use of PVC-coated tying wire is essential. There is also an emerging range of plastic clips and ties which are suitable for use in conjunction with epoxy-coated bars. Walking on placed reinforcement should be minimized. Avoid dropping heavy utensils on the bars. Take care that equipment for transport or casting of the concrete does not damage the epoxy-coating.

Experience has shown that with normal commonsense care, repair of damaged coating or field touch-up is seldom necessary. When using couplers it is necessary to choose a system which does not introduce damage into the coating as part of the process, such as, for example, the grips of a threading machine. Couplers may be coated with fusion-bonded epoxy and bare areas of the finished coupled joint should be painted with repair material. The bars should be inspected for damage to the coating both after delivery to the site and when finally installed.

What should be repaired depends on the specification. ASTM requires that the sum of damaged areas on a bar should not exceed 2 % of the total surface. In the UK a limit of 1 % is favoured. ASTM requires that a single damaged area larger than 60 mm² shall be repaired. Bent bars are inspected for more than hairline thick cracks and flaked-off coating. In the UK bent bars have to be inspected using special detectors. Cleaning of damaged areas with complete removal of rust is usually done with a wire brush and emery paper. Do not make the bare areas larger than necessary, as the corrosion resistance of repaired spots is less than that of the coating. The patching material should be applied strictly in accordance with the manufacturer's instructions.

The patching compound is usually of the two-component type. The two components must be mixed thoroughly in the correct volume relation. The mixed compound has a certain pot life. It is applied with brush or roller, and should have time to dry before the bars are handled.

When using a patching compound, bear these precautions in mind: keep away from heat, sparks and open flames, as the components are flammable; avoid contact with skin and eyes; wearing of safety goggles and impervious gloves is recommended; avoid prolonged breathing of vapours; use only in well-ventilated areas with sufficient air movement to maintain airborne concentration levels at recognized health and safety levels; keep containers closed when not in use; do not take internally. Suggested first-aid is

- (a) eye contact: immediately flush eyes with plenty of water for at least ten minutes; call a physician
- (b) skin contact: immediately wash thoroughly with soap and water
- (c) inhalation: remove to fresh air; immediately call a physician
- (d) accidental ingestion: *do not* induce vomiting; call a physician.

Also, when epoxy-coated bars are heated, for example by welding, toxic gases develop. Such operations should take place in well-ventilated areas only.

3.1.9. Applicable standards

3.1.9.1. *The USA.* ASTM A 775M-90⁷ is the latest edition of the first standard for this type of coating. In short, it sets requirements for the steel and epoxy powder used, the steel surface preparation, application of the coating, and requirements for coated bars.

The finished coating thickness shall be from 130 to 300 μm in 90% of checked cases. There shall be not more than six coating discontinuities (pinholes not discernible to the unaided eye) per metre of bar. The adhesion of the coating to the steel is tested by bending coated bars 120° around a mandrel with diameter eight to ten times that of the bar without cracking or disbonding appearing in the coating on the outside of the bend. The maximum damaged areas shall not exceed 1% of the surface of each bar. All coating damage due to fabrication or handling shall be repaired.

3.1.9.2. *The UK.* British Standard BS 7295⁸ requires that in 95% of cases the coating thickness shall be 150 to 250 μm , with 130 μm and 300 μm as absolute limits. The bend test is 180° around a mandrel the diameter of which is six times the bar diameter. The bars shall have no more than five discontinuities in their coating per metre. Damaged areas larger than 10 mm require repair. Not more than four such defects are allowed per coated bar length. Quality assurance should be consistent with BS 5750 (ISO 9002). BS 7295 is compared with ASTM A 775M-90 in Table 3.1.

3.1.9.3. *Germany.* A code for the use of epoxy, resin-coated steel reinforcement is in preparation. A version dated January 1990 restricts use to bars of 30 mm maximum diameter. The coating thickness should be between 200 and 300 μm with a maximum of six pinholes per metre. Faults larger than 25 mm shall be repaired. Bars with damaged areas which exceed 0.5% of the total bar surface after manufacture, or 1% after installation, shall not be used. Anchorage lengths shall be enlarged by 25%.

3.1.9.4. *Japan.* The Japanese Society of Civil Engineers has prepared JSCE EP10.⁹

3.1.9.5. *Norway.* A standard for epoxy-coated reinforcing bars is in preparation.

3.2. Performance in concrete environments

As coatings from electrostatically-sprayed fusion-bonded epoxy powder outperform coatings from liquid epoxy in all tests, in the following only coatings manufactured by the former process are treated.

3.2.1. In concrete generally

As mentioned previously, to protect the reinforcement against corrosion in concrete, the coating must be able to give

- (a) resistance against the alkaline environment of concrete, against carbonized concrete and water
- (b) good adhesion to the steel surface (because of danger of under-rusting)
- (c) resistance against low (freeze) and elevated temperatures (hydration heat)
- (d) high diffusion resistance against water vapour and oxygen and diffusion density against chlorides.

Table 3.1. Main differences between ASTM A 775M-90 and BS 7295

	ASTM A 775M-90	BS 7295
Coating thickness	90%: 130–300 μm	95%: 150–250 μm 100%: 130–300 μm
Bend test:		
Angle	120°	180°
Mandrel diameter	8D–10D	6D
Quality assurance	None	BS 5750

From experience to date it can be taken as granted that epoxy-coating mostly fulfils these conditions.

The protective action of the coating depends on its durability in the alkaline medium. Epoxy-coating is characterized by its resistance to alkalines, with no softening or peeling off.

Nearly all reported laboratory tests or practical experiences include the effects of chlorides. Extremely little data has been found from behaviour in uncontaminated concrete. However, tests involving measuring electrical current have shown that bars with undamaged epoxy-coating cannot serve as electrodes, due to electrical insulation. Damaged areas in the coating can act as cathodes if they are situated close to anodic areas. Then the relation between cathodic and anodic areas will govern the situation. As this ratio will usually be small, the corrosion will also be minimal.

The use of coated bars only in the top mat of reinforcement was standard practice in the USA and Canada throughout most of the 1980s. Today, mostly, both layers are coated. However, the question arises whether there is danger of macro-cells developing between coated and uncoated steel. If there is a conductive connection between the coated and uncoated reinforcement, there is a danger of cell formation. The following conditions must exist

- (a) active (anodic) and passive (cathodic) surface areas
- (b) electrical connection between anode and cathode
- (c) oxygen in the cathodic area.

Conditions are especially unfavourable if the coated steels are embedded in humid (not aerated), chloride-containing concrete and the uncoated steel is in drier (well aerated), still passivating concrete. Then sufficiently high potential differences arise. High current densities may only develop if the coated reinforcement has local faults, that is, when small anodes are opposed to extensive cathodes. Extensive field tests were reported from the USA comparing three different cases:

- (a) both reinforcing layers uncoated
- (b) both layers coated and with damage
- (c) upper layer coated and with damage, lower layer uncoated.

Regarding the currents measured to the existing anodic surface, (c) gave ten times higher corrosion current densities than (a), because in (c) 1% of the surface acts as an anode. It follows that by using coated reinforcement the formation of macro-cells should be considered.

When discussing the relative merits of using coated, coated/uncoated or uncoated reinforcement, a review by the Washington State Transport Centre concluded that: The rate of corrosion in a chloride-contaminated bridge deck can be reduced by 46 times if fusion-bonded epoxy-coated reinforcement is used throughout, and by 12 times if epoxy-coating is used in part, that is, in the vulnerable areas.

The bond of epoxy-coating to the steel is normally too good for large areas to be undermined. Should undermining occur, the corrosion between coating and steel often results in formation of flat cavities where there is little oxygen, and the corrosion process is therefore slow. Another process involving the transformation of water into hydrogen is then more probable. This hydrogen might penetrate into the steel and start hydrogen-induced crack corrosion. This process is only possible in damaged coating on high-strength steels used for prestressing.

3.2.2. *In carbonated concrete*

Nothing is found in the literature concerning problems of epoxy-coatings in carbonated concrete. This may be because no problems have been met in practice, or that epoxy coatings have not been used in such a situation.

3.2.3. In chloride-contaminated concrete

The use of epoxy-coated reinforcement began in bridge decks exposed to de-icing salts. Many tests therefore relate to bridges. The general result reported from the USA is that a noticeable reduction in corrosion damage to reinforcement has taken place over the period in which epoxy-coating has been in use. The only known negative result, most likely due to bad workmanship, is reported in section 3.2.5.

An evaluation by the Pennsylvania Department of Transport of 'Bridge Deck Protection Measures' included half-cell and electric-resistance field determinations, chloride and permeability measurements, and visual inspections of reinforcing bars in cored samples. Although there were some mixed results in the comparative evaluations, the overall consensus from the work was:

- (a) coated reinforcing bars should prove to be more effective in corrosion protection than low-slump concrete, latex-modified concrete, membrane or overlay systems
- (b) under short-term exposure (less than 35 years), galvanizing and epoxy-coating should be equally effective in corrosion protection
- (c) zinc coating is sacrificed in the short term, so epoxy-coating should be more effective if longer-term protection is required
- (d) lack of adhesion between the concrete and the epoxy-coated reinforcement emphasizes the importance of mechanical interlock in the bond strength of epoxy-coated reinforcing bars
- (e) some quality-control problems are still encountered. Specification revisions were recommended that required 100% touch-up, more extreme bend test requirements (1800), and tighter coating thickness tolerances.

After laboratory work in the UK, the Building Research Establishment (BRE) concluded that epoxy-coated reinforcement offers significant, effective protection in chloride-contaminated environments and at a chloride concentration of 3.2% by weight of cement. The BRE said: 'Epoxy-coated reinforcement offers enhanced reinforcement durability relative to galvanized or unprotected carbon steels. However, damage to the coating at bends or cut ends may reduce the durability of the material.'

At an American Concrete Institute (ACI) symposium it was reported that in cyclic wet-and-dry salt water exposure tests epoxy-coated reinforcing bars did not corrode, even in the presence of a high chloride concentration.

An accelerated corrosion test in Finland involved both uncoated bars and bars with epoxy-coating thicknesses of 100, 200 and 300 μm . The concrete cover was 20, 40 or 70 mm. Figure 3.1 presents the percentage of corroded

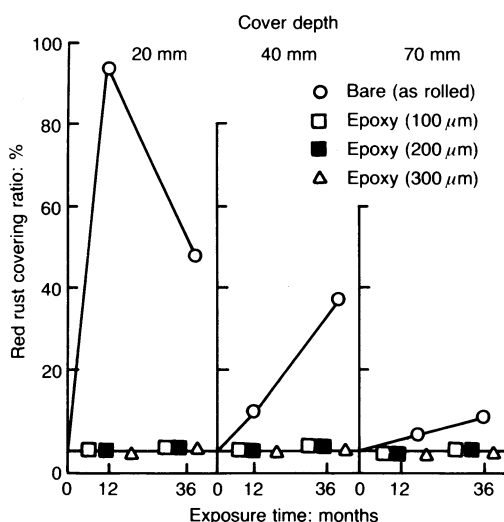


Fig. 3.1. Rusting of steel bars cured in the tidal zone

area of the total steel surface inside specimens cured in the tidal zone. All epoxy-coated bars remained totally free from corrosion regardless of thicknesses of cover or of coating and the length of curing time. The coating also retained its initial properties such as hardness, glossiness, adhesion to steel surface and continuity.

Accelerated corrosion laboratory tests under increasingly aggressive conditions were performed on epoxy-coated bars embedded in concrete. Either the specimens were immersed in a NaCl-solution, or chlorides were added to the concrete. Other specimens were exposed to industrial atmosphere, in a splash water region or to alternate freezing and thawing. The conclusion drawn was that under extreme corrosive conditions the behaviour of epoxy-coated bars is more favourable than that of uncoated bars. However, it was recognized that damage in the coating tends to result in local under-rusting.

Finnish tests of more than 300 cylinders with embedded bars investigated how different curing conditions influence the durability of coated bars. The conclusions were

- (a) sound epoxy-coating gave good protection against corrosion even in aggressive environments. Coated bars were in good condition after 24 and 36 months of sea water curing inside cracked concrete cylinders containing CaCl_2
- (b) damage of coating considerably weakened the corrosion prevention
- (c) addition of CaCl increased the corrosion of bars with damaged coating
- (d) cracks in the concrete did not increase corrosion
- (e) sea water curing increased steel corrosion remarkably.

All coating materials have a certain permeability in relation to their thickness. The Finnish report mentioned that epoxy-coatings absorb water to some extent. Therefore, thin coatings (50–250 μm) are not totally impermeable to water. Chloride permeability is considerably lower than that of clean water. Test results indicate the chloride-ion permeability of powder epoxy-coatings to be zero, or very much less than that of liquid epoxy-coatings.

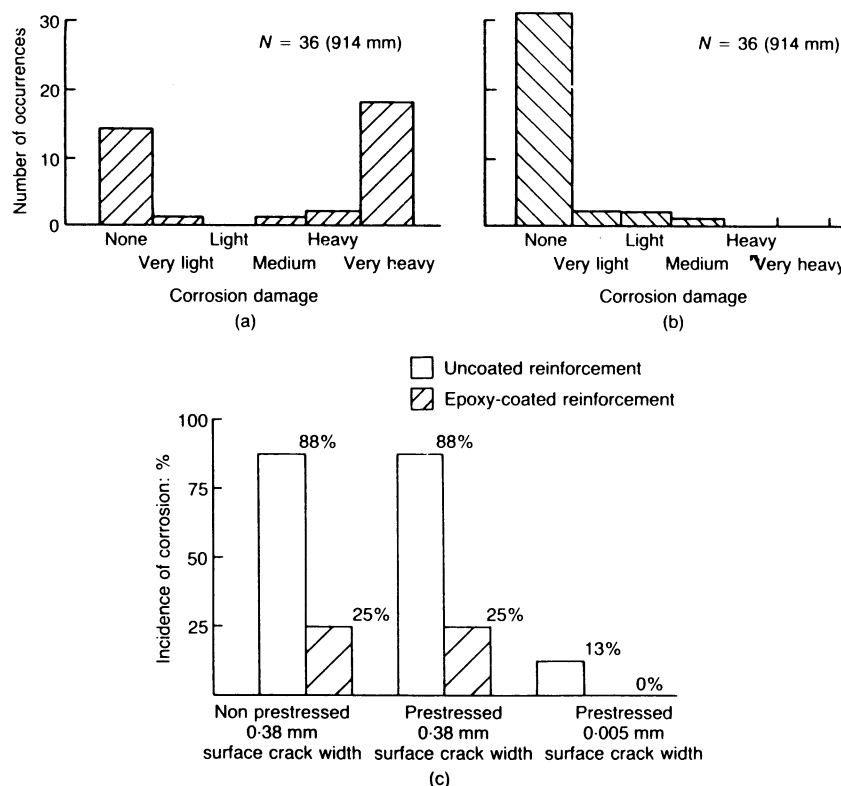
3.2.4. *At cracks in the concrete or coating*

3.2.4.1. *Cracks in the concrete.* Section 3.2.3 reported tests which showed that cracks in the concrete did not increase corrosion of epoxy-coated bars with an undamaged coating. Another test used 200 mm thick specimens simulating bridge decks. They were reinforced by uncoated or epoxy-coated ribbed bars, and had combinations of unstressed or stressed tendons. A 3.5% salt solution was bonded on the top surface every 14th day. The next day the specimens were subjected to loading cycles to produce top-surface crack widths of 0.05 mm for prestressed and 0.38 mm for non-prestressed conditions. The cycles were repeated 8 to 14 times. Figure 3.2(a) shows that on uncoated bars heavy corrosion (severe pitting) was observed on a majority of the specimens. The corrosion often extended along the bars for six to eight bar diameters. According to Fig. 3.2(b), epoxy-coated bars show very little corrosion. Fig. 3.2(c) indicates that epoxy-coated reinforcement is extremely beneficial for crack widths of 0.38 mm. At a width of 0.05 mm there was still some corrosion on uncoated bars, while the epoxy-coating was completely effective. It was concluded that

- (a) epoxy-coated reinforcement greatly reduces the incidence and extent of corrosion in cracked structural members
- (b) repeated loading of epoxy-coated reinforcement in concrete specimens can result in minor chipping and flaking of the coating on the ribs of the bars. This can provide the path for minor corrosion.

The ACI has observed that new bridge decks with epoxy-coated reinforcement developed an excessive number of deep cracks during the early

Fig. 3.2. (a) Histogram of corrosion of uncoated reinforcement; (b) histogram of corrosion of epoxy-coated reinforcement; (c) comparison of the incidence of corrosion of reinforcement in non-prestressed and prestressed and prestressed specimens



stages of curing. This may be due to several causes

- (a) greater concrete shrinkage (with higher cement contents and certain admixtures, along with higher concrete strength)
- (b) greater cover (steel is not as effective in restraining surface cracks)
- (c) lower 'in-and-out' bond strength of epoxy-coated bars (transfer of tensile thrust into the bars at cracks and away from cracks).

Australian researchers pointed out the problems that can result from the fact that, with time and frequent loading, the coating will flow away from high-stress locations. An example is the base of a transverse lug on a ribbed bar. A higher concentration of chloride ions will occur where the epoxy is squeezed out. The need for more precise control of coating thickness is thus apparent.

3.2.4.2. Cracks in the coating. Through defects in the coating, aggressive agents can penetrate to the steel surface. Corrosion is concentrated in these areas, resulting in reduction locally in the cross-sectional area of the bars. Defects which are smaller than those required to be repaired in relevant standards are not regarded as causing any corrosion risk. Only larger defects are required to be patched.

3.2.5. Long-term performance of the coating

Since the use of epoxy-coated bars in bridge decks started only 16 years ago, comparatively few data are published on the long-term performance. A field study in Pennsylvania covered 22 bridge decks after about 10 years service. None of the 11 decks reinforced with epoxy-coated bars showed any visual signs of deterioration. Four of the 11 decks with uncoated bars showed deterioration caused by corrosion of the reinforcement.

The ACI reports that, generally, there has been a lack of serious problems on well-managed jobs. Good housekeeping and common sense should take care of most problems. But touch-up techniques have been somewhat haphazard, and tighter enforcement on requirements of touch-up is needed.

One failure in performance of epoxy-coated bars has recently been the focus of much attention. Five bridges built in the late 1970s and early 1980s linking the Florida Keys, in less than ten years showed severe corrosion above the high-tide mark. High temperatures and humidity as well as extremely high chloride content of the surrounding sea water create one of the harshest environments encountered in reinforced concrete applications. Some reports indicate the bridges were built to an experimental design to save money, and they started cracking immediately. Early crack widths were more than twice the size tolerable in a corrosive environment. Four of the bridges are now showing severe signs of deterioration.

Regarding the epoxy-coated bars used in these bridges, factors stressed in these reports were as follows.

- (a) The coating firms have ceased to exist, and no records are available. Inspection of coated bars reveals bad coating workmanship. Testing showed serious coating fracture and disbondment when subjected to either standard bend test or fabrication bending.
- (b) Corrosion was limited to the splash zone and was initiated in fabricated bars at bends and then progressed to straight parts.
- (c) Chloride concentration in concrete was five to seven times that normally associated with the onset of corrosion.
- (d) Cover to the bars was less than 25 mm, and in all cases below specification.

A report from the University of South Florida said that the information available to date is not sufficient to draw a conclusion on the specific mechanisms leading to the deterioration observed in the field. Weathering at the construction yard is expected to have created a certain amount of disbondment (of the coating) around the imperfections. This disbondment would be worse where the rebar has been fabricated. This report is summarized in six points.

- (a) Severe corrosion of epoxy-coated reinforcing steel has been documented at major construction projects in a subtropical marine environment.
- (b) The damage takes the form of extended metal loss with additional localized pitting. Low pH water accumulates at the resulting metal-coating crevices. Coating disbondment extends beyond the areas of high metal loss. Spalling of the concrete cover was observed within ten years of service.
- (c) The deterioration could not be traced to any particular deviation from construction design or materials specification (this conclusion contradicts other reports saying the steel used in the bridges was seriously damaged and precorroded for around six months before being put into the work).
- (d) Disbondment at the metal-coating interface prior to concrete casting can result from fabrication bending or from exposure to a sodium chloride electrolyte.
- (e) The corrosion morphology can be reproduced in anodically polarized epoxy-coated rebar exposed to an alkaline, chloride-rich electrolyte.
- (f) It is proposed that the corrosion resulted from the combined effects of several factors, including mechanical coating damage and weathering exposure prior to concrete casting, and macro cell action during service in a severely corrosive environment.

A later report investigated the effect of precorroding damage to coated steel in a chloridic environment and concluded this will lead to substantially increased corrosion over that which would be expected from similar damage which was not precorroded.

This is the first known incidence of significant corrosion of epoxy-coated

reinforcing bars in the USA. However, the result is not surprising, if some reports are correct, when adding up the factors involved in this failure.

- (a) Epoxy-coating of bars is not meant to, and will not of itself, protect against the use of poor concrete or below-specification cover.
- (b) The epoxy-coating itself is suspect. Some of the bars exposed and examined showed cracking at the bends. The specification allowed no cracking or disbonding. Where was the inspection?
- (c) While cure of the coating was found to be satisfactory, there are severe doubts as to the cleanliness and application of the coating.

3.3. Practical experiences

The first major field application of epoxy-coated reinforcing bars was in a Pennsylvania bridge deck in 1973. Fusion-bonded epoxy-coated reinforcement reached the commercial market in 1976, and fusion-bonded epoxy-coated welded fabric in 1984. ASTM first issued a standard specification for epoxy-coated reinforcing steel bars in 1981. A similar ASTM specification for epoxy-coated welded fabric is being developed.

In contrast, a recent trend in Germany is to use epoxy-coating for the protection of bars, but to use polyvinylchloride (PVC) coating for welded wire fabric, because of the need for a more ductile (though weaker) material to coat the fabric.

3.3.1. Experiences reported

Experiences from practical applications were presented in section 3.2.

An ACI symposium consented that more information is needed, through testing and/or documented information, on the following items related to epoxy coating

- (a) bond fatigue strength
- (b) fatigue strength of lap splices
- (c) bond strength of bars confined by coated stirrups
- (d) effect on bond strength of
 - (i) the interactive effects of concrete cracking, repeated loading and corrosion, as typically experienced in ocean structures
 - (ii) these effects plus thermal effects in thin walls of cooling towers and similar structures
- (e) development and/or establishment of epoxy-coatings prequalified on the tight bends used for stirrups and ties
- (f) in-bond and anchorage tests of coated welded fabric, and the effect on the integrity of the epoxy-coating at the concentrated bearings on the cross wires near the welded intersections with the longitudinal wires
- (g) integrity of coating after long-term direct exposure to weather and ultraviolet radiation
- (h) corrosion/bond testing with large pre-cracks in the specimens (because corrosion concentrates at cracks)
- (i) the optimum combination of parameters to minimize cracking in bridge decks
- (j) long-term in situ (cast in concrete) durability to provide information on factors controlling long-term life
- (k) further verification that the allowance of 2% damaged coating is satisfactory (ASTM A 775M-90 has now reduced this to 1% damage).

3.3.2. Extent of use

A short historical survey follows.

1970: US Federal Highway Administration identifies problem in bridges. Research by National Bureau of Standards identifies epoxy-coating as the most viable and cost-effective solution.

- 1973: Epoxy-coated reinforcement first used experimentally in a bridge.
- 1975: Canadian research into major problem of reinforcement corrosion identifies epoxy-coating as the most promising answer.
- 1977: Epoxy-coated bars being used as standard procedure in bridge decks in 17 US states. Further nine states carry out field trials.
- 1978: Ontario Highway Authority decides to use epoxy-coated bars in top-deck slabs and parapet walls.
- 1979: First Canadian bar coating plant commissioned.
- 1981: Positive corrosion protection utilizing coated bars employed throughout bridge structures in Ontario. ASTM standard for specification of epoxy-coated reinforcing steel.
- 1985: Middle-East commissions first coating plant in the region.
- 1987: Coated bars specified as standard in bridges in 48 US states. Use of coated bars as positive corrosion protection extends to multi-storey car parks, marine structures, power plants, cooling towers, water treatment plants. First European purpose-built rebar coating line commissioned.
- 1988: Opening of coating plants in Norway and the UK.
- 1990: British Standard BS 7295 issued for specification of epoxy-coated reinforcement.

The consumption of epoxy-coated reinforcement in the USA has increased gradually from about 1000 tons annually in the 1970s to around 180 000 tons in 1987, of from 5% to 7% of the total cut and bent rebar market. An annual consumption of 250 000 tons is soon expected.

In Canada, consumption is reported to have developed rapidly, without any total tonnage mentioned. In Vancouver the world's largest cable-stayed bridge used 4000 tons of epoxy-coated reinforcement in pier and deck. The state of Ontario has a mandatory specification to use epoxy-coated bars in the top mat of all bridges.

In the Middle-East, epoxy-coated reinforcement has been used to combat corrosion arising from the often unavoidable use of contaminated aggregates. The Middle-East market is currently greater than 30 000 tons annually.

In the UK, applications in the early 1980s were confined to a few small trials, such as the Colwick Loop Road Bridge in Nottingham and a test section of concrete pavement on the M18 motorway. In contrast to the extensive use in bridges in the USA, only two of sixteen projects in the UK were bridges. This radical difference is partly due to an obvious difference in climate and social pressure to maintain an open roads policy which has led to salt consumption in USA on a much greater scale than in the UK. Additionally, many states in the USA have not embraced the use of waterproof membranes in the construction of bridge decks, a standard feature of UK road bridge construction.

Recent UK applications have included parapets and soffits in bridges, in sea defences and harbour works, in silos at chemical plants, pipelines and water-retaining structures. Overall the tonnage used in the UK remains relatively small, but the number of applications is growing rapidly.

In the rest of Europe the construction industry has been even slower than in the UK to embrace epoxy-coated reinforcement. Both in the Netherlands and in Scandinavia some trial structures have been built. By far the largest project to specify epoxy-coated reinforcement is the Storebaelt Crossing in Denmark. The Eastern Bridge could use 20 000–26 000 tons, while the Eastern Tunnel already uses epoxy-coated reinforcement.

3.3.3. *Economy*

A responsible engineer will carry out a cost-benefit analysis and risk assessment. Experience shows that 'non-rusting reinforcement' costing twice

Table 3.2. Cost comparisons, UK supply of epoxy-coated reinforcement

Bar diameter: mm	Costs as a factor of black bars	
	Straight bars	Complex bent bars
10	2.8	3.3
20	1.4	2.9
40	1.2	1.5

Table 3.3. Norwegian cost ratios

Bar diameter	8	10	12	16	20	25	32
Factor	3.6	2.3	2.2	2.1	2.0	2.0	2.0

as much as normal reinforcement is still 'good value' when subsequent protective measures, or indeed refurbishment, to cope with corrosion are taken into account. In special cases, even higher reinforcement costs may still offer the cheaper and better solution.

In the USA commercial competition and economics of scale have combined to provide epoxy-coated reinforcement in straight lengths at between 1.4 and 1.7 times the price of uncoated bars. Similarly in the Middle-East, ratios of this level are the norm. In Europe as a whole, the use of epoxy-coated reinforcement is so embryonic that no clear cost indications have yet emerged.

In the UK the cost is on average 2.1 times the cost of conventional reinforcement delivered on site. However, it must be remembered that the cost of epoxy-coating will be very sensitive to the exact bar diameter and the complexity of bending required. Table 3.2 illustrates UK costs. At the introduction of epoxy-coated bars in Norway, supposing 20 tons delivery of grade 500 ribbed bars to the Oslo area, the cost ratios were as shown in Table 3.3.

From the USA the case of a parking deck at Parkdale Plaza at Minneapolis has been reported. The cost increase for epoxy-coated reinforcement in the deck was calculated as 40 cents per square foot, approximately 3.6% of the total costs. In contrast, conservative estimates of replacement costs range from \$13 to \$15 per square foot, which is 32 to 37 times the initial costs of using epoxy-coated bars.

Recent UK projects showed the extra costs of using epoxy-coated reinforcement to be (the percentage increase is a fraction of total costs)

- (a) road/rail bridge with top-mat only coated: +1.5%
- (b) chemical plant with epoxy-coated bars only: +3.0%.

An ACI workshop concluded that epoxy-coatings for reinforcement should find significantly greater use because their cost-effectiveness has been proven. Epoxy-coating has gained widespread acceptance for reinforcing bars, and is about to be used extensively for welded wire fabric. With the anticipation that epoxy-coated reinforcement will result in significant increase in the service-life of bridge decks, parking-garage floors and other structures, it has become generally apparent that epoxy-coating is very cost-effective.

4. PVC-coating

The general principle of PVC-coating is that it furnishes the steel with an impermeable barrier preventing gases, water and active chemicals from reaching the steel, and also electrically insulates the steel from the concrete, thereby stopping any corrosion process.

4.1. Manufacture: influences on materials and application

4.1.1. *Manufacture of the coating*

The process is similar to the electrostatic spraying of epoxy powder described in section 3.1. After cleaning and pretreatment of the steel, a specially modified PVC-powder is sprayed electrostatically on to the steel surface. The PVC is converted into a closed and firmly adhering layer by sintering or gelation at temperatures between 180°C and 240°C. With the electrostatic process it is possible to obtain a uniform coating on a strongly profiled surface and to maintain relatively close tolerances in the coating thickness. The desired thicknesses are between 130 and 300 μm , preferably around 240 μm .

4.1.2. *Properties of the coating*

PVC-coating can be applied free from pores or defects in adequate thicknesses to protect the steel against corrosive actions. There is practically no difference between epoxy- and PVC-coatings as regards the protection they provide against corrosion. PVC-coatings are almost impermeable to water vapour, oxygen and chlorides.

Resistance against chemicals is high, but PVC contains plasticizers which, among others, react sensitively to alkalis. Due to the molecular constitution of PVC-coatings, they are softer than epoxy coatings. On the one hand this softness causes a greater reduction in the reinforcement-to-concrete bond, but on the other hand it makes the coating more resistant to mechanical actions, including bar bending operations.

4.1.3. *Influences on the steel*

No influences on the properties of the steel have been reported or are known.

4.1.4. *Bond to concrete*

PVC-coated bars show a greater reduction in bond to concrete than epoxy-coated bars. Bond strengths of PVC-coated bars of only 14–60% of uncoated bars have been reported. For plain or vaguely ribbed bars the bond reduction could be even larger. However, the coating was found to be intact even after relatively large displacements of more than 0.5 mm.

Pull-out tests were conducted to compare the bond strength of epoxy-coated, PVC-coated and uncoated bars and welded fabric. The PVC-coating was again found to be more ductile, but less strong than epoxy coating. Compared with uncoated bars, the bond strength of epoxy-coated bars was somewhat lower, and that of PVC-coated bars was significantly lower (see Fig. 4.1). However, none of the coatings had any effect on the bond of anchorage strength of fabric, due to the presence of the transverse welded-on wires.

The comparatively low bond stiffness of ribbed bars coated with PVC leads to the recommendation that this type of coating should, for the time being, be used only on welded fabric. The bond to concrete can be controlled by the welded-on transverse wires, which should be checked in the design.

4.1.5. *Extreme temperatures*

PVC-coatings lose their rigidity at temperatures around 200°C, that is, they become ineffective in achieving reinforcement to the concrete bond. There is the further problem that PVC produces hydrochloric acid when decomposing.

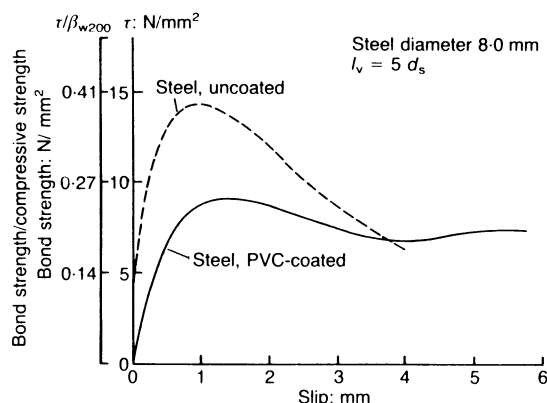


Fig. 4.1. Bond and slip of PVC-coated and uncoated steel in concrete (Rehm et al.)

Extensive fire tests on non-loaded concrete specimens reinforced with PVC-coated bars revealed that, although the chloride concentrations due to decomposition were relatively low and were confined merely to a few centimetres around the reinforcement, the corrosion protection was lost. In a case where the effect of the PVC-coating has to be preserved even after a fire, it will therefore be necessary to apply special protective measures to the structural member concerned in order to keep the temperature of the PVC-coated reinforcement safely below 200°C.

4.1.6. Fatigue

Fatigue bend tests on a slab reinforced with PVC-coated welded fabric showed good results even after 100 000 load cycles. The ratio of minimum to maximum stress was 0.6. The maximum stress resulted in crack widths of the order of 0.35 mm. No damage or spalling of the coating was observed. The test showed that the reduced bond of the PVC-coated fabric had a rather small influence on the crack distance, while its influence on crack widths was of no practical importance.

4.1.7. Cutting, bending and welding

What was said of epoxy-coated steel in section 3.1.7 should apply. Experience has shown that bending of PVC-coated fabrics presents no problem. Ordinary bending rollers may be used, and the bending radii stipulated in the normal codes of practice adopted. When bending around formers of diameter four times the bar diameter, no cracking of the coating appeared. On the compression side of the bend, flattening and crushing appeared, but there was no destruction of the coating. All the same, it is an advantage to use rollers covered or faced with a suitable padding material: this precaution reduces the risk of local bruising or other damage to the coating.

4.1.8. Handling, repair and safety

Again, refer to what was said about epoxy-coated bars in section 3.1.8. However, the elastic, but yet plastic, PVC-coating displays its particular advantage when fabrics are handled roughly. Even very large local deformations at the points of suspension do not cause spalling. At worst, the coating is plastically deformed or slightly worn at its surface. Such results were obtained when the fabrics were deliberately maltreated by pulling them over steel supports, by driving a forklift over them, by dumping a whole load of fabrics upon them, and by other forms of rough handling. Hence it can be concluded that PVC-coating is a rather tough form of protection, the effectiveness of which is substantially preserved even when the reinforcement is handled in a rough way.

With properly selected PVC-coatings, the risk of local spalling is indeed low. All the same, any steel surface that might be exposed should always

be patched with the appropriate patching material made available by the manufacturer of the coated reinforcement.

Areas where the coating has been subjected to bruising, surface abrasion or local deformations, should not constitute reasons for complaints against the supplier, as long as the steel surface has not been laid bare. The repair systems for patching any defective spots can *not* simply be bought from paint merchants. They should be recommended and supplied by the manufacturer of the coating material.

Assembly of coated reinforcement can take more time than uncoated, as coated or non-metallic tying wire may tend to slip. Another aspect is the slippery nature of coated bars when wet. Particular care should be taken when walking on coated reinforcement in damp weather.

4.1.9. *Applicable standards*

To date no valid standards have been developed. In Germany a 'Zulassungsbescheid' (permit)¹⁰ was given in 1989 for PVC-coated welded fabrics of ribbed wires with diameters from 4 to 12 mm. The permit contained requirements for

- (a) the pretreatment of the steel surface
- (b) the composition of the coating, with specifications of the materials
- (c) the application of the coating.

The coating thickness should be $240 \pm 40 \mu\text{m}$ in the unribbed areas. Mechanical damage of the coating must be repaired. Coated and uncoated reinforcement may be used in combination. For concrete under the influence of chlorides, there shall be no electrically-conducting connection between coated and uncoated reinforcement.

If the PVC-coating shows migration of plasticizer and embrittlement due to saponification in the alkaline concrete environment, it is possible that the properties of the coating may change detrimentally concerning adhesion to the steel surface and its general qualities. This behaviour cannot be stated for new coatings as they may not be derived from the chemical composition. Detachment of the coating and saponification of the material favours the formation of rust underneath the coating.

Regarding fire, it is stated that at temperatures above 150°C the coating dissolves, the bond to the steel surface is lost, and chlorides are separated which penetrate into the concrete. Welding on PVC-coated fabrics is not allowed.

Until now it has not been stated whether PVC-coatings separate chlorides or tend to detach or embrittle in concrete; it is not clear even if methods to determine such changes exist. For example, by embedding coated steel with a cover of 1.5 cm in concrete specimens, detachment, embrittlement and rust underneath the coating due to application of unsuitable PVC modifications may be found by inspection after 6 to 12 months. If there are free chlorides underneath the coating, they may be determined by means of X-ray analysis of the corrosion products.

4.2. Performance in concrete environments

The purpose of the PVC-coating just as epoxy-coating, is to prevent aggressive agents from getting in contact with the steel.

4.2.1. *In concrete generally*

In tests the use of PVC-coating turned out to be less advantageous than epoxy-coating regarding protection against corrosion of reinforcement in concrete. Independently of the kind of storage (industrial atmosphere, splash water area), the following phenomena were found for PVC-coating:

- (a) saponification of the plasticizers and, thus, detaching and embrittlement of the coating

- (b) in concrete the under-rusting of an intact coating was stronger than in solution, independent of carbonation and chloride content of the concrete. In concrete, oxygen can penetrate through the concrete cover and coating to the steel. Oxygen cannot reach the steel surface in a solution
- (c) corrosion intensity increased with decreasing coating thickness, concrete quality and concrete cover.

Specimens of PVC-coated reinforcement were stored for up to 140 days in alkaline solutions ($\text{Ca}(\text{OH})_2$), and also in cement paste with and without added sodium chloride and concrete admixtures. Some of the specimens were embedded in cement mortar. The PVC-coatings behaved favourably. Only under extreme conditions (cement paste at 350°C and 500°C) was some slight, but in any case negligible, migration of the plasticizers found. This did not involve any change in the properties essential to the protective effect of the coating.

The increased softness causes a reduction in the reinforcement-to-concrete bond. On the other hand, the coating is more resistant to mechanical actions including bar-bending operations. It might be advisable to use PVC-coating for welded fabric that is to be bent sharply.

As to mixing of PVC-coated and uncoated reinforcement, what was mentioned in section 3.2.1 applies. Experience shows that although the theoretically anticipated processes of macrocells do in fact occur, they are so retarded that the corrosive loss of metal remains within acceptable limits, except at major defects.

Due to its plastic and elastic properties, PVC-coating has been found to display a particular advantage compared to epoxy-coating: PVC-coating is a comparatively tough form of protection, the effectiveness of which is substantially preserved even when the reinforcement is handled in the normal way. The risk of major defects is therefore rather small. An objective assessment of the facts therefore indicates that the use of PVC-coated reinforcement in conjunction with uncoated reinforcement does not involve any 'unpredictable' risk.

4.2.2. *In carbonated concrete*

No special information is available.

4.2.3. *In chloride-contaminated concrete*

In Germany, tests were performed with PVC-coated reinforcement in alkaline solutions and in cement mortars. In each test cycle the specimens were sprayed with a 3.5% solution of sodium chloride (NaCl). The specimens were first stored at 35°C and 100% relative humidity for 8 h. Next they were placed at 25°C and 75% relative humidity for 16 h. After more than 75 cycles, the surfaces were still intact and had not undergone any reduction in their protective effects. Results obtained with specimens embedded in mortar or in concrete are in good agreement with those for specimens immersed in solutions.

4.2.4. *At cracks in the concrete or coating*

The tests referred to in section 4.2.3 also included specimens with intentionally-made holes in the coating.

For the specimens in alkaline solutions, relatively little under-rusting of the coating took place close to the defect. For specimens embedded in mortar or concrete, local corrosion under the action of chloride solutions took place at the holes in the coating.

The same results were achieved in specimens with an intentionally-made hole in the coating located in a predetermined crack in the mortar. Under the same conditions, uncoated bars exhibited considerable rates of corrosion.

4.2.5. Long-term performance of the coating

No data is available on this issue.

4.3. Practical experiences

4.3.1. Experiences reported

To the knowledge of the Task Group, PVC-coated reinforcement is not yet in practical use.

4.3.2. Extent of use

No use has been reported. Assessments in Germany for medium-term demand for PVC-coated fabric is set as 3 % to 5 % of the total consumption of fabrics. Ribbed wires only will be used.

The principal fields of application include cladding panels for buildings, shell- and slab-type structures, structures in aggressive environments such as bridge decks, decks in car parks, etc. Slender members in exposed locations are also a potential field of application, e.g. parapets, cornices, stairs, mullions and transoms on façades. This is especially the case when the reinforcement is formed into cages.

4.3.3. Economy

No data is available so far.

5. Choice of protection system

5.1. When is a protection system necessary?

There are many examples of old, durable reinforced concrete structures which show no, or very few, signs of corrosion damage. Inspections usually reveal that the workmanship has been good. This implies the use of a dense and well proportioned concrete which is sensibly compacted and cured, and that specified values for the concrete cover to the reinforcement are fulfilled. Neither has the environment been too aggressive.

Nowadays the construction process has speeded up for economic reasons. Sometimes the impression given is that the contractor mostly thinks of easy casting of the concrete when proportioning it. In some places accelerating admixtures containing chlorides are still used. In many places proper sand for concrete is scarce, and sand from the sea bed is used instead. If this is not washed, it will introduce chlorides to the fresh concrete. As proper curing of the concrete through its first two or three weeks is regarded as too costly, the speed of the later carbonation of concrete is considerably increased. Then there is the problem of many environments being far more aggressive than previously. In total, factors like these increase the danger of reinforcement corrosion. Another factor of great importance to durability is the size of the concrete cover to the reinforcement. Extensive new investigations show that the cover is usually less than specified, and often marks the start of corrosion attacks. More concern about correct cover should result in better durability. However, where modern types of high-strength concrete are used, the danger of corrosion could be somewhat reduced.

It is difficult to give specific advice about when protection of the reinforcement is necessary. It should be left to the evaluation of the engineer responsible for a particular structure. In general, when parts of a structure or a complete structure will be exposed to an environment evaluated as too aggressive for the expected quality of the concrete as produced, or there are doubts if the specified values of concrete cover will be kept throughout, the application of reinforcement protection should be evaluated.

A good rule of the thumb is the 'Law of fives' referred to in section 1.3. Based on practical experience, this rule implies that protective measures taken early, especially during design, strongly reduce the overall lifetime costs of a structure where there is a risk of corrosion of the reinforcement. As environments seem to be increasingly aggressive, and with escalating costs of repair work, in the future there is expected to be more use of protective systems for reinforcement.

Some rough calculations of the 'initiation period' for development of corrosion can be made by means of the 'square root law': the depth of carbonation or the penetration of chlorides is proportional to the square root of time.

5.2. Which protection system seems to be best technically?

Section 1.4 referred to several methods of reducing the risk of corrosion on reinforcement. This report has dealt with only three methods for protective coating reinforcement: galvanizing, epoxy-coating and PVC-coating.

PVC-coating is apparently in its infancy in Germany only, where it is intended for use on a single reinforcement product as welded fabrics, and no practical experience has been reported. Those interested in this method of coating should use chapter 4 of this report. As coating methods in practical use, this section therefore concentrates on galvanizing and epoxy-coating. Based on information from chapters 2 and 3, a brief comparison of the two methods yields the following.

Influences on the steel. Galvanizing of normal reinforcing steels has no practical influence. Epoxy-coating has no influence.

Extreme temperatures. Galvanized bars can take 200–230°C without harm. Epoxy-coated bars can tolerate about 200°C, but some slip of the bars in concrete may occur when the temperature exceeds 100°C.

Bond to concrete. Galvanized bars develop bond strength more slowly than uncoated bars in the early stages, but the ultimate value is equal to that of uncoated bars. Epoxy-coated bars reduce the ultimate bond strength by about 15% compared with uncoated bars. The slip starts at lower loads. An increase in anchorage and lap lengths of about 20% is proposed. New tests with confining reinforcement may result in reductions in proposed lengths. Under corrosive conditions both coatings maintain their bond strength for a longer period than uncoated bars, due to the delay of rusting.

Fatigue. Although the test conditions were not similar, the following comparisons can be made. Some galvanized bars have shown 15% reduction, some even better results than uncoated bars. Epoxy-coated bars were not adversely affected until 1.4 million cycles. The slip after the first cycles is larger, but is not significantly different later. Generally, the reduction in fatigue properties is small compared with the effects of a corrosive attack on uncoated bars.

In carbonated concrete. Galvanizing is beneficial; no problems have been reported for epoxy-coated bars.

In chloride-contaminated concrete. Galvanized bars exposed to chlorides added to the fresh concrete may develop pitting; further, the service life of the coating may be shortened. Regarding chlorides penetrating from the exterior, an upper limit of 1–1.5% chlorides by weight of cement seems to be relevant, if the concrete can dry out intermittently. Epoxy-coated bars with undamaged coating seem to withstand chlorides well, but excessive damage to the coating considerably weakens the protection.

At cracks in concrete. Galvanized bars show increased corrosion rates as crack widths increase above 0.3 mm, depending on the depth of cover. Epoxy-coated bars with undamaged coating are reported to reduce corrosion greatly in cracked concrete.

Long-term performance. Galvanized bars seem to perform well in carbonated concrete. Exposed to chlorides, it should be remembered that the zinc-coating is a sacrificing protection which will disappear over time. Epoxy-coated bars have been in use for less than 20 years, generally with good performance.

To conclude, it seems epoxy-coating is the most reliable method, especially when chlorides are present, but it must be ensured that damage to the coating is minimized. The bond to concrete is reduced compared with uncoated or galvanized bars, and for the time being longer anchorage and splice lengths have to be used.

5.3. Economic considerations

The evaluation will depend on the economic situation in each country. The factor of increased purchase price of coated straight bars compared to uncoated bars, reported in sections 2.3.3 and 3.3.3, are shown for some countries in Table 5.1. The costs are roughly the same. It is interesting to note that the

Table 5.1. Coating cost comparisons by country

Coating	Country	Bar diameter: mm						
		8	10	12	16	20	32	40
Galvanised	UK	—	1.9	—	—	1.8	—	1.6
	Norway	2.7	—	—	1.9	—	1.6	—
Epoxy-coated	USA	1.4 to 1.7						
	UK	2.8	—	—	—	1.4	—	1.2
	Norway	3.6	2.3	2.2	2.1	2.0	2.0	2.0

factor for epoxy-coating is lower in the USA than in Europe. Most likely this is due to commercial competition between several coating plants and to larger volume of coated bars. In time the same will probably happen in Europe. Estimates referred to in sections 2.3.3 and 3.3.3 indicate that the use of coated bars does not increase the total costs of a structure by much. In cases where corrosion attacks are expected, the use of coated reinforcement therefore seems to be profitable. At any rate, the costs of eventual repair work will far exceed the costs of using coated reinforcement.

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