

**GUIDELINES
FOR
CORROSION PREVENTION, MONITORING
AND
REMEDIAL MEASURES FOR CONCRETE
BRIDGE STRUCTURES**



**INDIAN ROADS CONGRESS
2008**

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GUIDELINES FOR CORROSION PREVENTION, MONITORING AND REMEDIAL MEASURES FOR CONCRETE BRIDGE STRUCTURES

1. INTRODUCTION

1.1. The Guidelines for Corrosion Prevention, Monitoring and Remedial Measures for Concrete Structures has been under the consideration of the Maintenance and Rehabilitation Committee since July, 2004. The draft as prepared by Maintenance and Rehabilitation Committee (B-8) was discussed in number of meetings.

1.2. The Maintenance and Rehabilitation Committee (B-8) (personnel given below) in its meeting on 23.2.2008 has finalized the "Guidelines for Corrosion Prevention, Monitoring and Remedial Measures for Concrete Bridge Structures" and recommended for its placement before through Bridges Specifications Standards Committee and Council.

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1.3. The draft document was approved by the Bridges Specifications and Standards Committee meeting held on 29.3.2008, and the Executive Committee in its meeting held on 11.4.2008 and authorized Secretary General, IRC to place the same before Council. The document was approved by the Council in its 185th meeting held on 11.4.2008, at Aizawl (Mizoram) for printing subject to incorporation some comments offered by the Council members.

CIVIL & STRUCTURAL ENGINEERING Civil and Structural engineers have been confronted with the problem of corrosion. The problem is aggravated due to use of composite materials such as reinforced concrete, pre-stressed concrete combined with extensive use of steel.

The process of corrosion is being studied scientifically by the specialists over many years. Engineers handling civil structures have to deal with this problem based on experiences gained in as much as repairs to structures and prevention of corrosion are concerned. It has now become imperative that this phenomenon of corrosion is well understood by the engineers who should also be equipped with measures for prevention and remedial actions so as to choose the most appropriate and effective measure suitable for a particular situation. With this objective, these guidelines have been prepared.

All concrete with embedded steel and steel structures are affected by corrosion. In this guideline, however, only bridge structures constructed with composite, reinforced and pre-stressed concrete are considered. Depending upon the type of material, the structural system and environment of the bridge, different types of corrosion takes place. Various factors and situations which promote and propagate this phenomenon are brought out.

Measures to prevent and control corrosion with a view to ensure that durability and service life of the bridges are not adversely affected are highlighted. Bridges are already in existence, many are under construction and several are going to be constructed in times to come. All these situations together with different types of materials and structural system and under varied exposing conditions have been dealt with.

Bridges need to be protected from the onslaught of corrosion. Protective measures are therefore necessary to be adopted.

During the course of service life of the bridge, it becomes necessary to know the state of its health with regard to corrosion and as such, monitoring methods are also described. Inspite of all these controls and preventive strategies, if any bridge is affected by corrosion, the techniques that need to be adopted are elucidated in this guideline.

2. SCOPE

This document deals with the guidelines on corrosion prevention, monitoring and remedial measures for concrete road bridges. Relevant IRC documents may be referred for assessing of strength, retrofitting etc. of structure.

The provisions of these guidelines are meant to serve as a guide to the design and construction engineers, but mere compliance with the provisions stipulated herein will not relieve them in any way of their responsibility for the safety and functional performance of the structures. These guidelines limit themselves to the areas of direct relevance to the corrections needed in the basic approach.

Various guidelines and specifications given are indicative. Actual requirement of proprietary items, wherever used, depends upon the patented design and specifications of manufacturers.

In brief, the scope of this guideline shall cover the following aspects of corrosion:

- Corrosion of Steel and deterioration of Concrete

- Aggressive Environment
- Corrosion Control Methods
- Protective Measures
- Investigation and Assessment
- Remedial Measures and Repairs
- Problem and Remedies, monitoring and Instrumentation.

3. CORROSION OF STEEL

3.1 Definition

Corrosion is defined as the "destruction or deterioration of materials in environments to which they are exposed". Metals and their composites (alloys) corrode at different rates, or not corrode at all depending on the chemical properties of materials with respect to their environment.

In overall sense under normal conditions, various metals and their naturally found compounds have stable states. The naturally occurring pure metals like gold and platinum are stable on their own and are noble metals. Iron (Fe) on the other hand, is not naturally stable and achieves stability in the form of oxides (FeO , Fe_2O_3 , Fe_3O_4 , etc.). This is explained by the 'energy level' of these materials. The oxides have higher energy level and to separate and obtain iron and steel, energy is supplied to them in the manufacturing process. These metals so extracted tend to slip back to lower energy levels, i.e. to a stable state. This is the fundamental reason behind corrosion.

This is a "thermo dynamical" point of view, and is very useful to understand qualitatively where and how corrosion will take place. This is conceptually presented in "Pourbaix" diagram, named thus after the scientist who studied and developed the same.

3.2 Electro-Chemical Mechanism of Corrosion

Corrosion is an electro-chemical process, in which corrosion-cells are set-up locally between the metal and surrounding material. Fig. 3.1 shows schematically the corrosion cell with anode and cathode, on the reinforcement bar, the bar itself providing electrical continuity for flow of electrons, and the charged ions flowing through electrically conductive surrounding (i.e. electrolyte). The explanation given below regarding currents and potentials is self-explanatory and simple to follow.

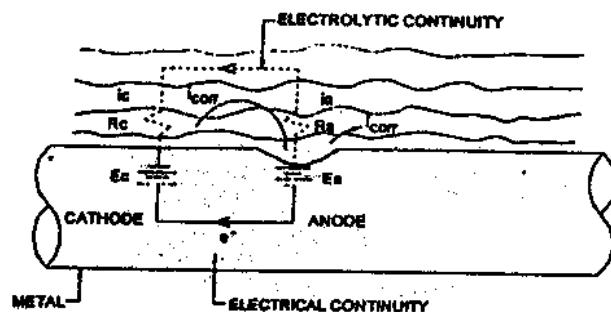
A question automatically arises as to how and why both anodic and cathodic sites exist on the same reinforcement bar. The answer lies in both the metallurgical history of the steel/iron, and in the surrounding electrolyte. In either case, the properties (i.e. potential) of electrolyte change due to local change at micro level in the composition of concrete, concentration of oxygen, acidity/alkalinity levels (pH), presence of chlorides etc. Different micro-environments can change potential of steel with reference to the electrolyte.

A typical example is seen in Fig. 3.2 where at a construction joint in concrete of a water retaining wall exchange in quality of concrete takes place due to leaching and corrosion. Anode is set-up causing

corrosion and steel eaten away. [Note: This figure also explains the observed high level of corrosion at construction joints with loss of section for steel].

CORROSION OF STEEL IN CONCRETE

CORROSION CELL



FOUR ESSENTIAL COMPONENTS

- (1) AN ANODE
- (2) A CATHODE
- (3) ELECTRICAL CONTINUITY BETWEEN "ANODES" AND "CATHODES"
- (4) ELECTROLYTIC CONTINUITY BETWEEN "ANODES" AND "CATHODES"

AT, E_{corr} CORROSION POTENTIAL.

$$i_{corr} = i_a = i_c$$

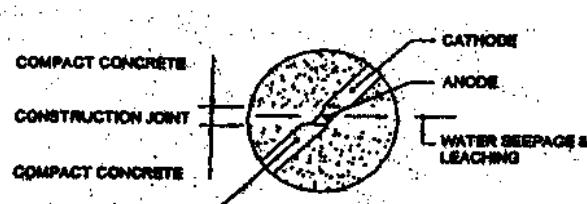
WHERE i_{corr} = CORROSION CURRENT DENSITY
 i_a = ANODIC CURRENT DENSITY
 i_c = CATHODIC CURRENT DENSITY

AND ALSO

$$\frac{1}{i_{corr}} = \frac{E_a - E_c}{R_a + R_c}$$

WHERE E_a = OPEN CIRCUIT POTENTIAL OF ANODE
 E_c = OPEN CIRCUIT POTENTIAL OF CATHODE
 $R_a + R_c$ = RESISTANCES OF ANODIC AND CATHODIC CURRENT PATH

Fig. 3.1 : Corrosion Cell



(a) CONST. JOINT / SEEPAGE OF WATER

COLOUR OF OXIDE

GRAY

IRON

BLACK

Fe_3O_4 FERROFERRIC OXIDES

WHITE

$Fe(OH)_2$

BROWN

$Fe(OH)_3$

BROWN / YELLOW

$Fe_2O_3 \cdot 3H_2O$

MAY GET ABSORBED IN CONCRETE

LARGE VOLUMES SPLIT COVER

REGIONS OF CONCRETE. EASIER

ACCESS TO HARMFUL ELEMENTS

0 1 2 3 4 5 6

(b) RELATIVE VOLUMES OF PRODUCTS OF CORROSION

Fig. 3.2 : Showing Corrosion

The metallurgical phenomena are more complex. In simple terms, it can be stated that different metallurgical history along the same bar (different levels of cold working or changes in types of local impurities in material and such factory), introduce heterogeneity in materials, which can and does cause different local potentials on the same bar. This phenomenon is called a 'local cell theory'.

More complex theories assuming randomly shifting sites of cathode and anode on steel surfaces are used to explain corrosion of ultra-pure homogenous materials and more or less uniformly observed pitting corrosion. However, these details are not of much interest to structural engineers, who mostly worry about the phenomena of local cell theory.

The calculation of rate of corrosion is based on the calculation of corrosion current/corr. This is done in terms of current density per surface area of steel iron (Amp/cm^2). The civil/structural engineers are conversant with the alternative method of expressing cumulative loss of metal (i.e. $S \text{ Fe}^+$) passing in the metal in given period of time, i.e. mills./year, etc. $\mu\text{m}/\text{year}$ or mdd i.e. $\text{mgm}/\text{cm}^2/\text{day}$.

Fig.3.2(b) shows the relative volumes occupied by iron and its oxides (corrosion products). Which oxide will form depends upon the surrounding and the amount of available oxygen. The extensive pitting in concrete, peeling off of cover regions in slabs results from the splitting pressure exerted by the volume of corrosion product on surrounding concrete.

3. Other Types of Corrosion

Although all corruptions are basically electro-chemical in nature and follow the mechanism described in practice, various names are given to the corrosion phenomena depending upon the circumstances in which it takes place or depending on the immediately apparent causative situation, or type and place. Some types, which are of interest to structural engineers, are briefly described in this section.

3.1. Uniform corrosion and localized corrosion

These terms indicate more or less uniform overall corrosion like atmospheric corrosion of exposed structures and localized corrosion caused by pitting corrosion at one location compared to the rest of steel is more heavy. Localized corrosion is of greater concern to structural engineers, since it will lead to local loss of strength and local fracture of steel reinforcement, which may lead to substantial loss of strength, local failure and eventual failure of the structure.

3.2. Galvanic or bi-metal corrosion

When two different metals are in physical contact, the metal less noble in electro-chemical series undergoes accelerated corrosion, e.g. in steel-aluminium contact, aluminium corrodes. Here, steel is cathodic and aluminium is anodic.

3.3. Crevice corrosion

This is noticed in steel structures at lap joints, bolts, rivets etc., where in the crevices formed, local accumulation of dirt/dust, salts, water etc., takes place. This forms highly conductive and sometimes aggressive environment leading to localized corrosion and failure of connection.

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3.3.4. Pitting corrosion

This is formation of highly localized corrosion leading to eventual loss of section/hole formation in otherwise un-attacked or mildly attacked surface. The reasons for pit formation are the same as those of crevice corrosion. Physical 'pit' formation (i.e. depression) is noticed earliest only by localized bugle or by falling away of corroded grains and physical roughness (depression) created by the same. In this process, the corrosion proceeds within the pit due to properties of surface steel on the pit being different from inner material. Also, corrosion between pits (inter-granular corrosion) proceeds along the grain boundaries, separating the grains due to localized stress concentration.

3.3.5. Ex-foliation

In this type, the corrosion process starting at the surface penetrates locally inside the body of the metal and then spreads laterally. The swelling pressure by high volume corrosion products builds up thus separating the sound metallic layers. External manifestations are the flaky or blistered surface. On edges, separation layers can be observed.

3.3.6. Stress corrosion cracking

Simultaneous action of high level of stress and corrosive micro-environment lead to faster rate of corrosion than that of unstressed or lightly stressed material in similar environment. Cracks formed this way often exhibit tendency to branch. This behaviour can be ascribed to crack propagation mechanics arising from local concentration of stress at the tip of the existing cracks (a fracture mechanism due to stress raising effect).

3.3.7. Hydrogen embrittlement

Under certain circumstances charged hydrogen ion in electrolyte is picked up by metal (i.e. hydrogen atom dissolves in metal crystalline structure of metal in localized defects), where atomic hydrogen combines to form hydrogen gas (H_2), which occupies a larger volume. If accumulated to sufficiently larger extent, it causes internal pressure and leads to local bursting failure. This was a serious matter few decades ago for pre-stressing steels; but is no longer the case for those steels deploying modern methods of production.

3.4. Factors Initiating and Progressing Corrosion

3.4.1. Behaviour of steel in 'Ideal' concrete

For an 'ideal' concrete, affording the best – almost non-corroding, - micro-environment for embedding steel need to have the following characteristics. The significance of these will be obvious, if its role as electrolyte in the mechanism of electro-chemical cell together with the behaviour of steel as understood from Pourbaix diagram is kept in view.

- (a) Maintain high alkalinity ($pH > 12$). This will retain the passivation film on steel.
- (b) High electrical resistance, which will directly reduce corrosion current /corr (i.e. corrosion at anode).
- (c) Dry concrete/low moisture contents, which will increase resistivity, change characteristics of electrolyte for better.

- (d) High density, low porosity, which will make penetration of harmful elements from outside difficult.
- (e) Almost crack free internal structure, which will have the same effect as (d) above. However, this requirement is fundamentally in contradiction with the structural behaviour of RCC. This may be achievable for pre-stressed concrete to a larger extent. However, the crack widths not wider than 0.2 mm width are known to have acceptable performance in controlling corrosion.

3.4.2. Effect of presence of oxygen

Oxygen plays an important role in setting up corrosion cells. Differential concentration of oxygen is caused by change in permeability, cracks, difference in depth of cover etc.. Areas with higher level of oxygen become 'cathodic' and those with relatively lower levels become 'anodic'.

3.4.3. Effect of water/moisture

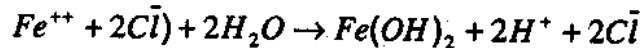
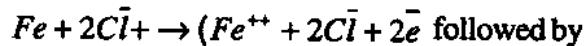
This is the most important single factor. Water takes part in the chemical reactions. The conductivity of electrolyte is also mostly due to water and water-salt mixtures. The quality of original mixing water itself is substantially reduced in hardening process and only small amount is left, which is well-held in pores. However, the externally penetrating moisture and water circulates freely, taking with it the harmful elements and setting up corrosion cells.

3.4.4. Effect of chlorides

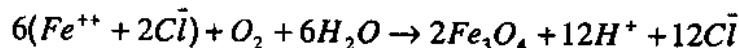
This is the best known and most feared contaminant. The fear is well founded arising out of number of collapses, serious reduction in life, damages observed in structures and costly repairs of structures situated in marine environment. Ports, bridges over creeks, on-shore structures within few hundred meters of sea are thus affected.

The reason for such widespread and incessant attack will be obvious, if one looks at the chemical reactions involved, shown in (i) and (ii) below:

- (i) Without oxygen at Anode:



- (ii) In presence of Oxygen:



Note the most important product on right-hand side of the reactions (i) and (ii). The 'Cl' has been released back in the micro-environment. It has acted as a catalyst in attacking and corroding iron, and free to attack again and again. Chlorine has other effects too:

- (a) Chlorine ions break down passivating film in Zone-2 of Pourbaix diagram.
- (b) External penetration causes differential concentrations and set up micro-cells.

Presence of salts (sodium and magnesium chlorides) in water increases its electrical conductivity.

How much chlorine can be tolerated? The experts differ. But, general observations indicate that:

- (1) Original chloride contents in concrete mix are evenly distributed and contribute less to corrosion than externally penetrating Cl. This is partially due to the fact that some of the chlorine is chemically bound and does not release ions of chlorine easily.
- (2) In presence of oxygen, critical chlorides as low as 700 ppm (0.07%) cause corrosion. In absence of oxygen, the tolerance is raised to about four times of this value (0.3%) depending upon the potential of steel.

3.4.5. Sulphates

Sulphates on their own do not cause corrosion. Their presence in electrolyte makes it more acidic (lowered pH) and more conductive. However, SH ion has effect on pre-stressing steel (ref. Section 4.1.3)

3.4.6. Other chemical elements and environments

Various other chemicals have their own effects on steel and steel-concrete micro-environments. These are, however, comparatively rarely met in special environments and reference is made to the specialist literature for understanding their effects and protective treatment. The Reynolds Handbook and the Handbook on Corrosion by the Electro Chemical Society of India can be the starting points of referencing.

The industries connected with Petrochemical, Fertilizers or any of the acidic products are the typical examples. Some of the environments created by presence of micro-biological life are known to lead to corrosive micro-environment.

3.5. Corrosion of Pre-Stressing Steel

3.5.1. General

The discussions presented in earlier sections remain valid for pre-stressing steel also. However, pre-stressing steel is more sensitive to corrosion than normal reinforcing steel for several reasons. These are:

- (a) Pre-stressing steel wires and strands made of wires are of smaller size varying between 3 mm to 8 mm dia. Any film of corrosion reduces the radius and the area of sound core of steel, very severely and therefore, leads to rapid loss of strength.
- (b) Additional mechanism (than those listed in Section 3), which is less severe for un-tensioned steel, becomes operative.

These include:

- (i) Inter-crystalline stress corrosion.
- (ii) Hydrogen embrittlement.
- (iii) Comparatively severe effects of sulphur.

- (iv) Metallurgical factors depending upon direction of crystalline arrangements.
- (v) Susceptibility to corrosion in all four zones of Pourbaix diagram.

These factors and their level of severity are discussed below.

3.5.2. Pitting corrosion

This is pitting corrosion as in case of un-tensional steel. The loss of area due to depth of pits and stress concentrations at edges of pits both have more severe consequence due to the originally small diameter and high level of average normal stress (local stress concentration effect).

3.5.3. Stress corrosion (leading to brittle failure)

The simultaneous effect of corrosive environment and high level of stress in the steel leads to faster and more severe corrosion. This combined attack is called as stress-corrosion. The risk of stress corrosion increases at stress level above 60 – 65% of UTS of presently manufactured steel.

The corrosion attack once initiated extends along the grain boundaries resulting in higher mechanical stress concentration and further crack propagation.

Some steels produced by hot tempering and quenching are having Bainites and Sorbites, which have an orientation that the propagation of cracks inside the steel is perpendicular to surface and hence more lagging, giving rise to brittle failure. The cold worked steels are better placed where pearlite steel have less and therefore less severe crack directions.

3.5.4. Hydrogen embrittlement

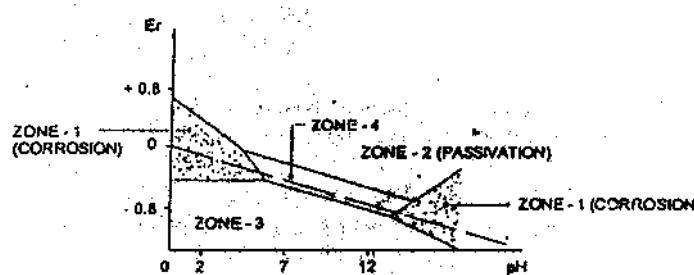
The modern cold work steels are not susceptible for this type of failure. It is however, interesting to know the mechanism.

Hydrogen is released near the steel surface by cathodic reaction. Also, if zinc is present (galvanized elements like steel ducts) it reacts with $\text{Ca}(\text{OH})_2$ and liberates hydrogen. Also sulphur in the form of SH prevents recombination of generated cathodic hydrogen and it then penetrates into steel along grain boundaries or cavities.

Inside the cavities 'H' combines to form H_2 (Hydrogen gas) which needs larger volume. This builds-up pressure inside cavities leading to stresses and crack progression, eventually causing brittle failure (hydrogen embrittlement).

3.5.5. Pourbaix diagram and pre-stressing steel

Referring to Fig. 3.3 Zone - 3 is shown as the immunity zone for normal steel. Zone-2 is relatively stable due to passivation of steel. Zone-4 is protective or susceptible depending upon the mechanical condition of the passivation film. For pre-stressing steels, in Zone-2, the stable film of Fe_2O_3 may get broken by chloride, leading to pitting corrosion. In Zone-3, the cathodic reaction may liberate atomic hydrogen which may lead to hydrogen embrittlement. Zone-4 may lead to formation of Fe_3O_4 , but pre-stressing steel is more sensitive to loss of area due to small diameters of steel. Hence, in all zones of Pourbaix diagram pre-stressing steel has its susceptibilities.

**POURBAIX DIAGRAM**

ZONE - 1 : CORROSION IN ACIDIC OR BASIC ENVIRONMENTS (STEEL DISSOLVES)

**ZONE - 2 : PASSIVATION ZONE : Fe_2O_3 STABLE FILM
MAY GET BROKEN UNDER ATTACK OF CHLORIDES
LEADING TO PITTING CORROSION.**

**ZONE - 3 : IMMUNITY ZONE : CATHODIC REACTION MAY REDUCE HYDROGEN FROM
WATER & ATOMIC H PENETRATE STEEL. HENCE ZONE OF
HYDROGEN EMBRITTLEMENT.**

ZONE - 4 : FORMATION Fe_3O_4 / COMBINATION OF ZONE 2 & 3 BOTH ACTIONS PROCEEDING

Fig. 3.3 : Pourbaix Diagram – Corrosion of Pre-stressing Steel

3.6. Structural Steel Members (without Concrete Surround)

3.6.1. General

The process of corrosion as is applicable to iron/steel is the same as that described in earlier Sections viz. the thermo-dynamical and the electrochemical concepts are valid. However, traditionally, steel structures exposed to environments are protected by coatings of various types and it is difficult to visualize the formation of electrochemical cell in the absence of obviously seen electrolyte. It is useful to remember that –

- Formation of cathodes/anodes on the same steel member is caused in similar way as those mentioned for embedded steel.
- Presence of dust, vapours, wet surfaces, liquids with low acidity levels ($\text{CO}_2 + \text{Water}$), and salt spray, sulphates in water etc., provide the conducting film of electrolyte needed for completion of cell formation, and hence to corrosion.
- Break-down of protective paint/other passivating membranes happens due to mechanical damage and chemical attack.
- The structural members do have unprotected areas such as rivet/bolt holes although in case of hot driven rivets, the holes are filled up more tightly as compared to bolt holes.
- Welded members are not always ‘sealed’ by seat-welding all around the contact faces of two members. The contact surfaces can hold moisture/water and salts carried by water.
- Phenomena of stress dependent corrosion exist locally due to high level of locked-in stresses in welded areas/members and bearing areas of rivets/bolts. It is well to remember that the average or ‘permitted’ design stress levels are only notional and steel has many areas of

lock-up internal stresses arising from various sources which when combined with externally imposed stress leads to local areas of high level of stresses.

- (g) Basic in-built resistance to corrosion is different for different types of steel. Also in iron/steel structures, the main external causes of corrosion have different hierarchy than the reinforcing/pre-stressing steel.

4. DETERIORATION OF CONCRETE

4.1. Phenomenon of Deterioration of Concrete

During its service life, concrete is exposed to several aggressive elements which contribute to the deterioration process. Some such important elements are given below:

- Aggressive Chemicals
- Alkali Aggregate Reaction
- Sulphate Attack
- Cavitation
- Abrasion
- Chloride Penetration
- Carbonation

Due to the attack of these elements, the process of deterioration in concrete sets in and it results into following distresses:

- Cracking and spalling
- Dissolution of concrete constituents
- Changes in mechanical properties
- Expansive volume change causing splitting of concrete

4.1.1. Exposure to aggressive chemicals

Certain chemicals in solution will attack various constituents of concrete and can be categorized as under:

- Inorganic Acids
- Organic Acids
- Alkaline Solutions
- Salt Solutions
- Miscellaneous

Acid attack in concrete is the reaction between the acid and the calcium hydroxide of the hydrated Portland cement. The reaction produces water soluble calcium compounds which are leached away. When dolomite or limestone aggregates are used, the acid may dissolve them.

4.1.2. Alkali Aggregate Reactions

Alkali-aggregate reactions (AAR) may create expansion and severe cracking of concrete structures. The mechanisms that cause alkali aggregate reactions are not fully understood. What is known about this type of reaction is that certain aggregates such as reactive forms of silica react with potassium, sodium and calcium hydroxide from the cement and form a gel around the reacting aggregates. When the gel around the aggregate is exposed to moisture, it expands, creating forces that cause tension cracks to form around the aggregate.

Concrete undergoing AAR exhibits tell-tale signs of surface map cracking on exposed surfaces. Once the cracking has formed, more moisture penetrates the concrete, accelerating the AAR. Typical cracks are shown in Fig. 4.1 below.

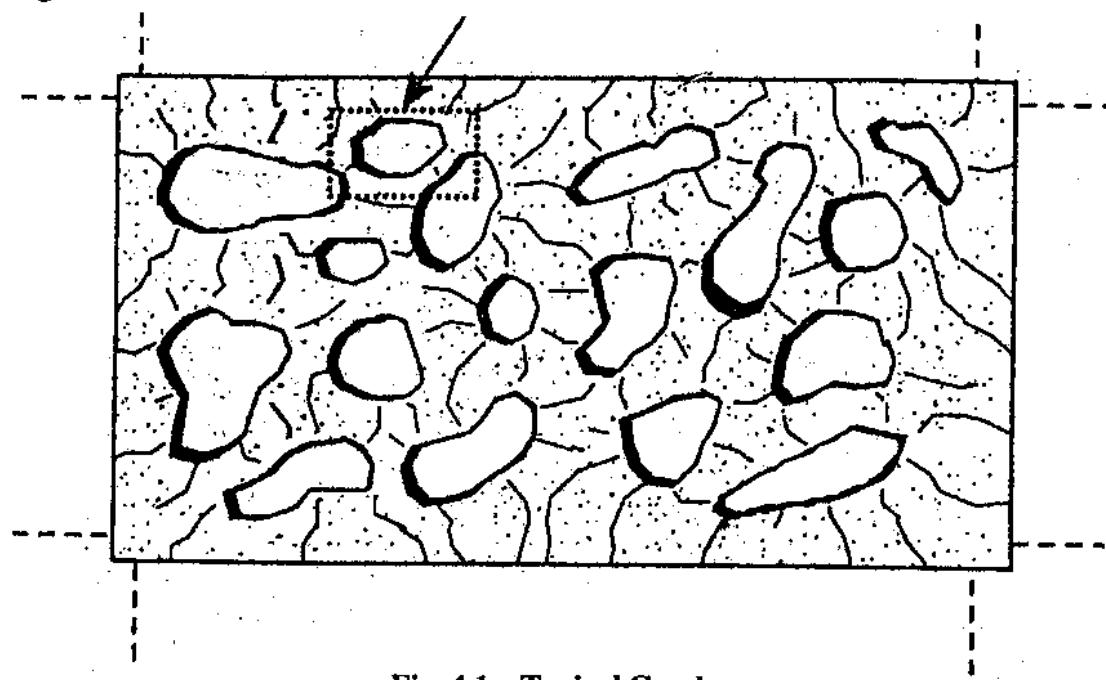


Fig. 4.1 : Typical Crack

The AAR may go unrecognized for some period of time, possibly years before associated severe distress will develop. Usually testing for the presence of AAR is conducted by petrographic examination of concrete and/or concrete bar expansion test. Recently, a new method is evolved in USA in which uranyl acetate fluorescence technique is used. This method would be fast and economical when fully developed.

4.1.3. Sulphate attack

The presence of soluble sulphates (principally those of sodium, calcium and magnesium) is common in areas of mining operations, chemical and paper mill industries. Sodium and calcium are the most common sulphates in soils, water and industrial processes. Magnetic sulphates are less common but more destructive. Soils or water containing these sulphates are often called "alkali" soils or water.

All sulphates are potentially harmful to concrete. They react chemically with cement pastes hydrated lime and hydrated calcium aluminates. As a result surface scaling and disintegration set in followed by mass deterioration.

Surface resistance of the concrete is improved by a reduction in water cement ratio and adequate cement factor with a low tri-calcium aluminate and with proper air entertainment. With proper proportioning, silica fume (micro-silica) fly-ash and ground slag generally improve the resistance of concrete to sulphate attack, primarily by reducing the amount of reactive elements (such as calcium) needed for sulphate reactions.

The effect of sulphate on concrete is diagrammatically shown in **Fig. 4.2** given below:

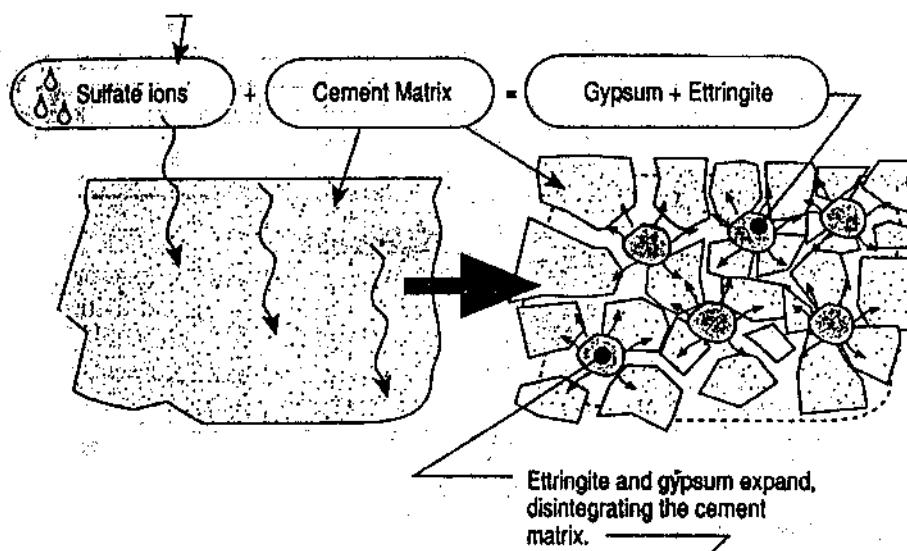


Fig. 4.2 : Effect of Sulfphate on Concrete

4.1.4. Cavitation

Cavitation causes erosion of concrete surfaces resulting from the collapse of vapour bubbles formed by pressure changes within a high velocity water flow. When vapour bubbles form, they flow downstream with the water. When they enter a region of higher pressure, they collapse (implode) with great impact. The formation of vapour bubbles and their subsequent collapse is called cavitation. The energy released upon their collapse causes 'cavitation damage'. Cavities are formed near curves and offsets at the centre of vortices. Cavitation damage results in the erosion of cement matrix leaving harder aggregates in place. At higher velocities, the forces of cavitation may be great enough to wear away large quantities of concrete.

4.1.5. Abrasion

Abrasion is the wearing away of the surface by rubbing and friction. Generally the surface is uniformly worn away including the cement matrix and aggregates. Factors affecting abrasion resistance include – compression strength, aggregate properties, finishing methods and curing.

4.1.6. Chloride penetration

Chloride gets introduced into concrete by contact with environment containing chlorides such as sea water. Penetration of chlorides starts at the surface and then moves inwards. Penetration is a slow process

and depends on permeability of concrete, moisture present and amount of chlorides in contact. Eventually, the concentration of chlorides in contact with rebars will cause corrosion, when moisture and oxygen are present. As the corrosion product builds up, tensile forces generated by expansion of the oxide cause the concrete to crack. Concentration of chlorides necessary to promote corrosion, among other factors is greatly affected by concrete pH.

4.1.7. Carbonation

Carbonation of concrete is a reaction between acidic gases in the atmosphere and the products of cement hydration. Normal air contains carbon-dioxide (CO_2) in relatively low concentrations (0.03%). The level of carbon-dioxide penetrates into the pores of concrete by diffusion and reacts with the calcium hydroxide dissolved in the pre water. As a result of this reaction, the alkalinity of the concrete is reduced a pH value of about 10 and consequently concrete protection of the reinforcing steel is lost. The passivity of the protective layer on steel is destroyed. When steel is de-passivated and the environment is acidic or mildly alkaline, corrosion begins if moisture and oxygen gain access into the concrete.

5. CORROSION CONTROL METHODS

Corrosion in bridge structures can be prevented by controlling various aspects of design, detailing and construction methods. It is also necessary to draw suitable specifications of materials including testing with acceptance/rejection criteria, which will have inherent features to take care of corrosion problems. At construction stage, methodology for construction, quality assurance for materials and products will have bearing on this process and will need to be controlled.

5.1. Design Stage

Following points shall be considered at this stage:

- **Depth of the cover** over the reinforcement should be minimum as given in **Table 5.1**. Wherever possible, it may be increased suitably.
- **Crack width** should be controlled and kept to the minimum. Provisions given in **Table 5.1** as per IRC:21, strive to achieve it by controlling the bar diameters and spacing. Alternatively, crack widths as calculated by formulae given in Appendix-I and IRC: 21 are recommended for achieving the same. It will be generally applicable to RC members and non pre-stressed members. For elements where crack width cannot be calculated, it should be controlled by using well distributed steel not exceeding spacing of 200 mm dia. In pre-stressed concrete members also, this early age shrinkage and thermal cracks should be controlled by minimum level of reinforcement similarly placed at not greater than 200 mm spacing.
- **Minimum Expansion Joints:** Joints are sources of leakage and as such these should be as minimum as possible. Continuous decking would be preferable wherever it is feasible. Traditionally most common form of construction for concrete bridges entails simply supported span where a joint is used at each end. These can be reduced by using deck continuity while designing and detailing. Continuous decks reduce number of joints.
- **Pre-cast Construction:** Generally control on quality of construction is better when members are

pre-cast as compared to cast-in-situ construction. Wherever feasible, pre-cast construction would be preferable in this context.

- Minimum number of bearings.
- **Integral structure :** This has inherent advantages of having jointless structure with reduced or no ingress of water.

5.2. Detailing of Bridge Components

There are certain locations as mentioned below, which are vulnerable to corrosion and need to be designed and detailed out properly.

- **End diaphragms under leaking expansion joints:** In new constructions, leak-proof expansion joints should be provided. In case of old bridges, the joints should be replaced or modified to take care of leakage.
- Pier and pier cap tops should be sloped gently to prevent ponding of water.
- **Bridge decks:** Waterproofing membrane should be provided below the wearing coat.
- Portions of pier and abutment in close proximity of the road surface.
- Splash zones of substructure where alternate dry and wet conditions prevail.
- Cable tendons, ducts and anchorages should be protected by grouting. Detailing to be such as to permit easy flow of concrete. Use of HDPE corrugated sheathing is preferable for protection of HTS from external penetration of chlorides and sulphates.
- Bearings and Appurtenances should be provided in such a way that it is possible to inspect and replace them as required. Drainage spouts shall be such as to avoid spilling of water on any component of the bridge.
- **Cables (Pre-stressing):** The cables should not be terminated at the deck level. Bunching of cables should be avoided.

5.3. Materials

5.3.1. Concrete

It is the best known material to the bridge engineers and should serve as a first line of defense against corrosion. Good concrete practices should be adhered to. In order to avoid corrosion of embedded steel, it is necessary to have proper thickness of concrete cover that is well compacted and impermeable. The durability of concrete depends on its composition, strength, properties of its constituents and how it is cast, compacted and cured. High resistance of the concrete cover to the penetration of elements through the mechanism of permeation, diffusion, absorption and capillary action will help in corrosion.

5.3.2. Cement and mineral admixtures

Ordinary Portland cement in sufficient quantity (kg/m^3) and low water-cement ratio provide adequate 'pH' value and dense concrete mix is needed to achieve corrosion resistance in normal conditions. However, addition of mineral admixtures to the OPC enhances durability parameters. Such additions can be by way of fly ash, silica fumes and ground granulated blast furnace slag (GGBS). It mostly decreases the permeability of the concrete. Such addition also results in less cement paste and as such less shrinkage and cracking. Addition of silica would increase shrinkage; cracks, further ice curing or other comparable methods may be made mandatory. Replacing suitable percentage of OPC with GGBS reduces the diffusion of chloride ions by a factor of 10. Partial/full reduction of cement if not felt necessary may be avoided. It is also shown that proper curing practice alone can improve performance in terms of rebar corrosion by a factor of 12.2.

5.3.3. Coarse and fine aggregates

The provisions specified for these materials in IS:383-1995 and IS:456-2000 are adequate and should be respected. The possibility of alkali-aggregate reaction should be investigated and if needed, suitable precautions should be taken.

5.3.4. Steel

For reinforcement, steel bars normally used are as follows:

- Mild Steel & High Yield Strength Deformed (HYSD) bars conforming to IS:432 & 1786. Methods of thermo-mechanical treatment and cold drawing, rolling or twisting (alone or combined) are permitted processes under IS:1786.
- Stainless Steel
- Pre-stressed steel

The rebars can be coated by suitable coating system as an additional safety against corrosion. Use of galvanized steel will also give good corrosion resistance. The details of such coating systems are given in Chapter 6.

5.3.5. Water

Water used shall not contain oil, acids, alkalis, sugar and organic materials which may be deleterious to concrete and steel and shall generally conform to provisions contained in IRC:21-2000 IS:3025 and IS:456.

5.3.6. Chemical admixtures

Chemical admixtures used to control w/c ratio and achieve desired workability should be conforming to relevant IS Codes. Compatibility of these admixtures with cement being used in the work needs to be established on each change of source of cement. The control dosage is very important and suitable quality control checks for ensuring consistently uniform properties of approved admixture are needed.

Concrete Penetrating corrosion inhibitors are one of the patented items for which generic specifications are not available. However, these are being used in concrete in new construction and claim that they help in combating corrosion of steel in concrete. The Employer may use such a system after satisfying oneself about

its adequacy, criteria for acceptance for both material and process control and final products. In this regard reference could be made to the tests prescribed in ASTM-G-109. However, all such products should necessarily be used with a performance guarantee of at least 5 years.

5.4. Construction Specifications

It should cover various aspects as mentioned below:

Concrete: A dense mix of concrete should be used with specified minimum quantity of cement. High grades of concrete should be used with admixtures such as super-plasticizers and mineral admixtures such as fly ash, silica fumes and slags as generally described in **Table 5.2**.

Water Cement Ratio: It shall be just adequate to ensure desired workability and placing conditions. Use of chemical admixtures for reducing w/c ratio is recommended.

Permeability: Concrete should not be permeable to allow ingress of water, oxygen, carbon dioxide, chloride, sulphates and other potentially deleterious substances. Required impermeability can be achieved by optimum cement content, sufficiently low water cement ratio, mineral admixture and ensuring good compaction and curing.

Cover: Protection to the rebars in concrete against corrosion is provided by the adequate thickness of good quality concrete. The cover should be as generally given in **Table 5.1**.

Chloride and Sulphates content in concrete: It is necessary to check chlorides and sulphates in constituents of concrete. Concrete may be contaminated by these elements coming from the environment. To minimize the deterioration of concrete from harmful chemical salts, the levels of each element coming from the constituent materials of concrete as well as by diffusion from the environment should be limited to the values given in **Table 5.1**.

5.5 Summary of Corrosion Control Methods

Table 5.1: Requirements to prevent/control corrosion

Sr. No	Parameters	Requirement	Reference	
A. DESIGN STAGE				
1.	Crack width	A) Average strain value calculated as per specified formula should be negative. B) Crack width under sustained loads shall not exceed 0.2 mm for severe conditions of exposure and 0.3 mm for moderate conditions.	IRC: 21-2000 Clause 303.4.2 Appendix -1	
2.	Clear Cover for Reinforcement	Exposure Moderate Severe Alternate Wetting & Drying zone } 75	Nominal Cover (in mm) 40 50 } 75	IRC: 21-2000 Clause 303.4.3

Sl. No.	Parameters	Requirement	Reference
1.		Note: The above cover may be reduced by 5 mm for factory made precast products with higher level of quality assurance.	
3.	Cover for pre-stressing steel and spacing	<ul style="list-style-type: none"> Wherever pre-stressing cable is nearest to concrete surface, the minimum clear cover measured from outside of sheathing shall be 75 mm. A minimum clear distance of 50 mm or diameter of the duct whichever is greater shall be maintained between individual cables. 	IRC: 18-2000 Clause 16.1, 16.3
4.	Grouping of cables	<p>Grouping of cables shall be avoided to the extent possible. If unavoidable, only vertical grouping of cables upto 2 cables shall be permitted.</p> <p>In case of severe condition of exposure, grouping of cables shall be altogether avoided. This may be achieved by use of high capacity strands.</p>	IRC:18 -2000 Clause 16.4
5.	Proper detailing and spacing of steel	Attention should be given to detailing to ensure proper concreatability, use of vibrators etc.	
B. MATERIALS			
6.	<ul style="list-style-type: none"> Coarse Aggregates Fine Aggregates 	<p>The preferred nominal size of aggregate is 20 mm for pre-stressed and reinforced concrete. It should consist of natural sand crushed stone or gravel. It should not contain dust, lumps, soft or flaky particles, mica and other deleterious materials.</p> <ul style="list-style-type: none"> Concrete should be kept wet at least for 14 days. 	IS:383 & IRC:21-2000
7.	Water	<p>Should not contain injurious amounts of oil acids, alkalis, sugar and organic materials, which may be deleterious to concrete and steel.</p> <p>Permissible limits of solids shall be as under:</p> <p>Organic - 200 mg/l Inorganic - 3000 mg/l Sulphates - 400 mg/l Chlorides - 500 mg/l (RCC) Suspended matter - 2000 mg/l • pH value - 6 • Use of sea water is not allowed.</p>	IRC:21-2000 & IS:3025
8.	Reinforcement	<p>a) Following grades of reinforcement steel shall be used:</p> <ul style="list-style-type: none"> Grade S-240 Mild Steel 	IS:432 – Part I. IS:1786

Sr. No	Parameters	Requirement	Reference																				
		<ul style="list-style-type: none"> Grade S- 415 – Deformed bars. Grade S-500 – Deformed bars <p>b) Coated bar</p> <ul style="list-style-type: none"> - Galvanised, zinc aluminium coating - Fusion bonded epoxy coating. 	As per relevant standards detailed under chapter-6																				
9.	Concrete	<ul style="list-style-type: none"> Cement content shall not exceed 450 Kg/m³ of concrete. <p>Minimum cement content for various structural members and different exposure conditions shall be as under:</p> <table border="1"> <thead> <tr> <th>Structural Member</th><th>Moderate Exposure</th><th>Severe Exposure</th><th>Min. Cement Content</th></tr> </thead> <tbody> <tr> <td>PCC Member</td><td>M-25</td><td>M-30</td><td>360 kg/m³</td></tr> <tr> <td>RCC Members</td><td>M-30</td><td>M-35</td><td>380 kg/m³</td></tr> <tr> <td>PSC Members</td><td>M-35</td><td>M-40</td><td>400 kg/m³</td></tr> <tr> <td>HPC Members</td><td>--</td><td>M-40</td><td>380 kg/m³ with Admixtures 450 kg/m³ without Admixtures</td></tr> </tbody> </table> <p>Sulphate Content (SO₃) shall not exceed 4% by mass of cement used in the mix of concrete. In severe condition, this should be limited to 2%.</p> <p>Chloride Content in concrete shall not exceed following values by mass of cement :</p> <p>PSC - 0.1%</p> <p>RCC - Severe exposure 0.2%</p> <p>RCC - (Moderate) 0.3%</p> <p>The above values shall be respected in case of HPC also.</p>	Structural Member	Moderate Exposure	Severe Exposure	Min. Cement Content	PCC Member	M-25	M-30	360 kg/m ³	RCC Members	M-30	M-35	380 kg/m ³	PSC Members	M-35	M-40	400 kg/m ³	HPC Members	--	M-40	380 kg/m ³ with Admixtures 450 kg/m ³ without Admixtures	IRC:21-2000. Clause 302.6.5
Structural Member	Moderate Exposure	Severe Exposure	Min. Cement Content																				
PCC Member	M-25	M-30	360 kg/m ³																				
RCC Members	M-30	M-35	380 kg/m ³																				
PSC Members	M-35	M-40	400 kg/m ³																				
HPC Members	--	M-40	380 kg/m ³ with Admixtures 450 kg/m ³ without Admixtures																				
	Water Cement Ratio	Maximum water cement ratio should be maintained as under:	IRC:SP:70-2005 IRC: 21-2000 Table-5																				

Sr. No	Parameters	Requirement			Reference
		Member	Normal Exposure	Severe Exposure	
		PCC	0.45	0.45	
		RCC	0.45	0.40	
		PSC/HPC	0.40	0.40	
10.	Sheathing	<ul style="list-style-type: none"> • Use Corrugated HDPE Duct. They are especially recommended to serve as barrier to externally penetrating chlorides/sulphates. • Whenever feasible, metallic ducts may be manufactured at site to eliminate long storage period and thereby corrosion problem. • to prevent corrosion on outer side surface should be applied with washable water soluble oil/VPI powder injection. 			

C. CONSTRUCTION STAGE

1.	Curing	Concrete shall be kept atleast for 14 days. Steam or other improved method of curing wherever necessary should be used.	IRC:21-2000
2.	Storage and Handling of Pre-stressing Material	<p>All pre-stressing elements such as strands/wires, anchorages, ducts, couplers, should be protected. These materials should be stored in such a way that there is no deterioration in quality and usage in the work. These should be protected from rain, damp ground, ambient temperature by covering them properly. These should be coated with soluble oil, silica gel or vapour phase inhibiting materials.</p> <p>The HTS strands should be wrapped in HDPE wrappers. The steel should be stored in the godown wherein humidity should be controlled upto 60% with the help of de-humidifiers, electric heaters or equivalent means. Hygrometers should be installed in such godown to check humidity.</p> <p>Just enough quantity of strand/HT wire sufficient to last for 4 weeks or so should be ordered.</p>	IRC:18-2000 Appendix-3

Sr. No	Parameters	Requirement	Reference
3.	Grouting of Post-Tensioned Pre-stressing cables.	<p>The grout protects the pre-stressing steel and ensures encasement of steel in an alkaline environment for corrosion protection and by filling the space, it prevents water collection and freezing.</p> <ul style="list-style-type: none"> • For effective protection, recommended practice stipulated in IRC:18:2000 - Appendix - 5, in respect of materials, grout vents, equipment, properties of the grout, mixing and grouting operations etc, shall be followed. • Grouting shall be carried out as early as possible but not later than 2 weeks of stressing a tendon. Whenever this stipulation cannot be complied with for unavoidable reasons, adequate temporary protection of the steel against corrosion by methods or products which will not impair the ultimate adherence of the injected grout should be ensured till grouting. The sealing of the anchorage ends after concreting is considered to be a good practice to prevent ingress of water. For structures in aggressive environment, sealing of the anchorage ends is mandatory. • No admixture containing chloride of any sort shall be used in grout. • Water cement ratio should not be more than 0.40 • Temperature of grout should not be more than 25°C. Use ice to maintain the temperature. • Power driven equipment should be used for grouting the cables. • Grout cap should be used to provide protection to the anchorage elements. <p>Should be supervised by Senior engineers. There is tendency to leave it to junior staff since it is a low value, last item.</p>	IRC:18-2000

D. OTHER DETAILS

1.	Deck slab	Water proofing membrane of mastic of minimum thickness of 12 mm should be provided under the wearing coat to prevent	MOSRT&H specification Clause
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S.R. NO.	PARAMETERS	REQUIREMENT	REFERENCE
		ingress of water in the deck slab and corrosion of cables and rebars.	2702.1.1.
2.	Railing	Provide mild steel post and pipe railing and paint them periodically.	IRC:5-1998
3.	Crash Barrier	Crash Barrier of desired height with or without pipe	IRC:5-1998

Use of Admixtures

Addition of cementitious minerals or admixtures is done to improve workability and general practice followed is as under:

Table 5.2

S. No.	Minerals	Recommended Method	Codal reference
1.	Fly Ash	It should conform to Grade I of IS:3812 as part replacement of Ordinary Portland Cement. Blend uniformly with cement while mixing.	IS:456:2000
2.	Ground Granulated Blast Furnace Slag (GGBS)	<ul style="list-style-type: none"> • It should conform to IS - 12089 • The blending must be uniform 	IS:456:2000
3.	Silica Fume	It should be used as per standards approved by the authority. Usual proportion is 5 to 10% of cement content of a mix. Uniform mixing must be ensured.	IS:456:2000 IRC:SP:70-2005
4.	Admixtures	<p>It should comply with IS:9103. Compressive strength, workability and slump with and without admixture should be established by trial mixes before actual use.</p> <p>The admixture selected should be independently tested for chlorides.</p> <p>The admixture should be such that it should not affect durability of concrete.</p>	IS:456:2000

5.6. Aggressive Environment

5.6.1. Micro environment, aggressive elements and general atmospheric conditions

From the discussion in Chapter No.3, it is clear that the type and extent of corrosion is directly and solely influenced by the micro environment around the steel. The factors influencing the corrosion are also discussed there. However, the current codes of practices have generally divided the conditions of exposure of the structural elements based on the general atmospheric environment into two classes: Severe and Moderate. The moderate includes all conditions except what is described as 'Severe' (Ref. IRC:21).

5.6.2. Precautions for different situations

Various preventive measures for various situations are briefly given in the Table 5.3 below:

Table 5.3

S. No.	Situation/Materials	Preventive Measures	Reference
1.	Concrete in Severe Environment	<ul style="list-style-type: none"> • Selection of appropriate materials for production of concrete. • Use suitable additive/admixture. • Follow good concrete practices. • Concrete placed must be dense and impermeable which gives protection against salts in sea water. • Avoid discontinuity keep joints minimum and achieve proper construction joints. • Adequate cover 	<ul style="list-style-type: none"> • ACI:318-89 • Rilem Technical 32 • RCA Committee • IS:456-2000 • IRC:21-2000 • fib-
2.	Alkali aggregate reactions likely	Low C ₃ A cement	Special literatures
3.	Concrete subject to sulphate attack	<ul style="list-style-type: none"> • Sodium sulphate in ground water attacks concrete. Use sulphate resistant cement or super sulphated cement. When both chlorides and sulphates are present, sulphate resistant cements should not be used. 	IS:456
4.	Concrete subject to aggressive chemicals (sulphate chlorides etc.)	<ul style="list-style-type: none"> • Provide surface barrier or coating with chlorinated rubber, epoxy or other speciality formulations. • OPC cannot resist attack of even weak acids. Therefore, use sulphate resistant cement and micro-silica. 	Standard textbook on Concrete e.g. Properties of concrete by Neville
5.	Reinforcement	<ul style="list-style-type: none"> • Use of galvanized steel as reinforcement is effective. Powder coating with epoxy can also be used. 	Relevant IS Codes for Specifications & Methods.
6.	Pre-stressing steel	<ul style="list-style-type: none"> • Provide anti-corrosive treatment to anchorage components. • Tendons should be grouted completely with dense and solid grout (zero bleeding grout). 	IS:456

S. No.	Situation/Materials	Preventive Measures	Reference
7.	Atmospheric pollutants	<ul style="list-style-type: none"> Epoxy coated or galvanized structural steel. In general, appearance of fair face concrete diminishes with time in polluted atmospheres. Integral water-proofing admixtures or coating of concrete surface may be adopted for preserving the appearance. 	
8.	Chloride Penetration	<ul style="list-style-type: none"> Use of corrosion inhibiting admixtures in concrete. Coating of rebars Special additives (microsilica) low water cement ratio will reduce permeability 	
9.	Carbonation	<ul style="list-style-type: none"> Protective coating on concrete surface Realkalization of concrete Low permeability 	

6. PROTECTIVE MEASURES

In order to prevent corrosion, in addition to control of the design mix proportion of concrete, measures can be taken to directly protect various other materials elements in the bridges. These elements broadly are rebars, pre-stressing steel, concrete surfaces, anchorages, sheathing, and structural steel. Presently, some of the protecting methods are covered by IS standards. Other patented/general systems are covered by specifications, processes, etc. as developed by specialist manufacturers.

6.1. Reinforcement Coating

Protective anti-corrosive treatment to steel reinforcement before it is embedded in concrete can guard the steel against corrosion during construction as well during service life. Different types of treatments used in practice are as under:

- System covered by Standard Specifications
 - Galvanising
 - Zinc-Aluminium Coating
 - Fusion bonded Epoxy coating
 - Cement polymer composite coating
- Patented/General System
 - Epoxy Phenolic Rebar coating system
 - Epoxy based coating

metallic Coatings

6.1.1. (a) Galvanizing (zinc coating)

Although coatings can be provided by number of metals such as nickel, copper, lead, tin etc., coating provided by zinc is more suitable and economical. This process is known as "Hot Dip Galvanizing" and involves steps such as picking, rinsing, flux solution dipping, drying and coating.

Recommended mass of zinc coating to be provided shall be as given the following **Table 6.1**.

Table 6.1

S.No.	Environment	Mass of zinc coating- minimum gm/m ² of surface
1.	Aggressive surrounding such as marine areas, chemical plants	915 (125 micron)
2.	Normal	610 (85 micron)

Zinc coating covers up any scratches/holidays that may occur in the coating due to electro-chemical property of zinc. Galvanized bars can be bent without cracking or peeling up of coating due to ductility of zinc. Bond characteristics and weld-ability of zinc coated bars remain practically unaffected.

Its use should be avoided where there is possibility of chloride contamination.

Testing of Galvanized Bars:

Following tests given in **Table 6.2** shall be carried out to determine suitability of galvanized bars and couplers before using them in concrete:

Table 6.2

S.No.	Test	Criteria for Acceptance	Reference	Remarks
1.	Hammer Test (Hammer Wt.215 gm)	Should not peel off	IS :2629	
2.	Knife Test (Sharp Edge)	No scratches and no peel off	IS :2629	
3.	Thickness Test	Zinc coating should conform to specified thickness	IS :3203	
4.	Preece Test (copper sulphate test for coating uniformity)	No copper deposits	IS:2633	
5.	Hydrogen evolution test for purity of zinc coating wt.	Coating mass to be as specified	ASTM-A-123	
6.	Stripping test 1.5% HCL solution	Coating as specified	IS:6745 IS:4759	

(b) Zinc + Aluminium Coating

The system of coating with zinc containing about 5 per cent aluminium is being practiced since eighties. The composition corresponds to the eutectic in the zinc aluminium phase diagram. It has a very fine microstructure, which gives good ductility and allows it to be cold drawn after coating wire. The corrosion resistance in the salt spray test of the zinc aluminium coating is estimated to be 2 to 3 times higher than the pure zinc coating.

Under corroding conditions, the zinc in the coating reacts sacrificial and generates cathodic protection. It is dissolved first and thus the aluminium content in the layer gets higher. Oxidation of the aluminium takes place and aluminium oxide thus formed provides a good barrier against further corrosion.

The production is similar to that of hot dip galvanizing with zinc. In case of wire/strand dipped in molten bath, there is a need for very careful surface preparation. In the first step, the steel passes through a classic hot dip zinc bath. In the second step, it passes through a zinc bath with controlled aluminum content.

6.1.2. Fusion bonded epoxy coating

• The System

In early seventies, this system was originally developed in USA and around 1988, it started in India.

Fusion bonded epoxy is basically 100 per cent solid finely ground fused powder particles, which when heated, melt to form a continuous adherent film. It involves coating of epoxy and bonding over re-bars under factory conditions and as such, with appropriate QA/QC systems, reliable quality can be ensured.

• Surface Preparation

The surface should be cleaned by abrasive blast cleaning to near white metal. The surface profile shall be free from mill scale, rust and foreign matter when viewed under well lit conditions. After cleaning, the coating shall be applied as soon as possible. Any formation of rust blooms on the cleaned bars shall be removed by blast cleaning before application of coating, which shall be done within 8 hours after cleaning.

• Application

The coating shall be applied as an electro-statically charged dry powder sprayed on to the grounded steel bar using an electro-static spray gun. The powder is applied to either hot or cold bar. The coated bar shall be given a thermal treatment specified by the manufacturer of epoxy resin, which will provide a full cured finished coating. Temperature shall be controlled as recommended by the manufacturer of the coating to ensure a workman like job without blistering or other defects.

- **Tests on Coated Bars**

Following tests given in Table 6.3 are carried out to check the efficacy of FBEC coating system:

Table 6.3

S.No	Parameter	Requirement	Reference
1.	Chemical Resistance	<p>Specimen without holidays and with 6 mm ϕ intentional holes drilled through coating shall be immersed in each of the following:</p> <ul style="list-style-type: none"> • Distilled water • Aqueous solution of CaCl_2 • Aqueous solution of NaOH • Solution saturated with $\text{Ca}(\text{OH})_2$ • The coating shall neither blister, soften, loose bond nor develop holidays. • The intentional hole should not exhibit under cutting. 	IS:19620-1993
2.	Resistance to Applied Voltage	<p>Effect of Electrical and Electro-Chemical stresses on the bond and integrity of coating is assessed.</p> <p>The specimen is suspended in the Electrolyte which is aqueous solution of 7% NaCl and a potential of 2V is applied. Coated bars are Cathode and Anode.</p> <p>No film failure should take place at Cathode end and no deposits of corrosion products at Anode end should be seen.</p>	
3.	Adhesion	<p>3 coated bars are bent @120° around a 150 mm ϕ mandrel.</p> <p>No visible cracks or disbonding in the coating on outer radius shall be permitted.</p>	IS:13620-1993 Clause 13.3
4.	Abrasion	<p>Resistance to coating shall be checked by Table Abrasor by using CS-10 wheels.</p> <p>Weight loss shall not exceed 100 mg/1000 cycles.</p>	
5.	Impact Test	<p>A mass of 1.8 kg. is dropped on to the coated bar from a height of 0.6 m. to 1.25 m.</p> <p>No cracking or loss of bond should occur at the impact area within 9 impacts.</p>	
6.	Hardness Test	<p>A knoop hardness tester is used for checking hardness.</p> <p>The hardness shall not be less than 16 knoop hardness number</p>	IS:6885-1973

S.No	Parameter	Requirement	Reference
7.	Thickness of coating	<ul style="list-style-type: none"> Should be between 0.1 mm to 0.3 mm. Minimum 15 readings shall be taken at uniform spacing. Minimum 90% readings should be within specified limits. 	IS:13620
8.	Continuity of coating	<ul style="list-style-type: none"> To be inspected visually. Coating should be free from holes, voids, cracks and damaged areas. Holidays when checked with detector should not exceed 2 for a length of 300 cm. 	IS:13620 IS:13620 Clause B.2
9.	Bond Strength	The mean bond strength at measured slip of 0.025 mm and 0.25 mm for coated bars shall not be less than 0.80 of corresponding mean bond strength of uncoated.	IS : 13620 A-5.5

- Handling and Identification**

The coated bars should be carefully handled. The bundles of coated bars shall not be dragged or dropped during handling. Wooden packing batons shall be used while conveying, stacking, placing or stacking so as to avoid rubbing of coated bars against each other or hard surface, which may damage the coating during handling. Bundles of coated bars shall be tied with PVC binding material.

The coated bars shall be stacked leaving gap between ground and bars with wooden batons between rows of bars or bundles of such tied bars. As bundles of coated bars shall be lifted with a strong back, spread or bar, multiple supports or a platform bridge to prevent bars – to -bar abrasion from sags in the bundles.

The identification of all temporary bars shall be maintained through coating process of the point of shipment by way of taking from the reinforcement bars delivered to the point until delivery point of the client.

- Pros and Cons of the System**

Though this system is being used in India for more than a decade and in USA for more than 3 decades, there are opinions expressed in favour and against this system, based on the experience of its performance in previously built structures. Briefly, some points are highlighted here for the benefit of decision makers.

Points in favour of the system

- Research carried out by The National Bureau of Standards (NBS) for Federal Highway Administration, USA concluded that epoxy coated reinforcement by fusion bonding process in excess of 102 microns in thickness would be capable of protecting steel from corrosion. Similarly, research done in other countries (Canada, Japan, UK) has shown that FBEC bars performed better as compared to uncoated bars.

- In India, Central Road Research Institute conducted the test on coated bars in the laboratory and in field for 3 years and evaluated that the performance of FBECR bars is satisfactory.
- It is possible to repair cut ends and damages to the fusion bonded epoxy coated bard by touch-up methods. Within few hours, it can be used.
- By and large, in aggressive environments, this method is recommended.

Points against the system

- There is no passivating primer film provided in case of FBEC rebars.
- Adhesive chemical bond between steel and concrete is broken and mechanical bond is adversely affected by smoothening effect over the base of the projecting ribs in HYSD. Reduction of bond of the order of 25% is reported. The coating may peel off if not properly done.
- Investigation carried out on 40 bridges in Florida Key in USA has revealed that dis-bondment can occur easily in the FBEC rebars which lacked passivation layer of Fe_2O_3 and is a precursor to corrosion.
- Higher co-efficient of Thermal Expansion of fusion bonded epoxies impose large thermal stresses in epoxy coating leading to its early failure.
- Lot of precautions and care is required in transportation, handling and placement of these coated bars failing which there will be damages, cuts, peeling off of coat and abrasions which may lead to corrosion.

In Indian conditions this is a major factor needing attention.

Performance Guarantee

This type of protective coating should be executed only through authorized technical applicators of standard manufacturers who have requisite work experience for having carried out similar type of coating works. However, the Employer/department may take performance guarantee from the agency responsible for the execution of the work for a period of 5 years or so, which is the minimum time for defects due to corrosion of embedded steel to start becoming apparent.

6.1.3. Cement polymer composite coating system

- **The System**

Rebars embedded in concrete are surrounded by an alkaline medium and as such cement based coating is more compatible. Basically two coats are applied – Primer coat and Sealer coat. The primer and sealer products have thermo-plastic acrylic resin as basic raw material. The sealer product is formulated with resin mixed with cement as a pigment. The principle of the system is that the base metal contains 'p' electrons which get released in corrosive environment leading to formation of Iron Oxide (Fe_2O_3) popularly known as rust. To prevent oxidation, a surface coating capable of nullifying the released electrons is provided. The sealer coat is compatible with primer and alkaline environment.

• The Surface Preparation

Prior to blast cleaning, visible deposits of oil or grease etc, are removed by suitable cleaning method.

Clean and dry compressed air is used for nozzle blasting, which shall be carried out by silica sand free from contaminants.

Dust and residues are removed from prepared surface by brushing and dry air vacuum cleaning.

• The Application

The primer coat shall be applied to the cleaned surface after cleaning within 4 hours either by brushing or dipping. Application of sealer coat can be started after 30 minutes of application of primer coat. The sealer coat shall be applied either by brushing or by dipping and thickness shall be 150 microns for severe conditions higher thickness of 200 macrons or so may be adopted.

Air curing for minimum 6 hour is essential before the coated bars are handled or put to use.

• Inspection and Tests

Following checks shall be made on coated bars:

- Dry film thickness
 - Uniformity of thickness
 - Cracks
 - Peeling
 - Bulging
 - Uncoated areas (holidays)
- } Elcometer should be used for this.
} By visual inspection

Tests to be conducted for the following:

- Adhesion
- Bond strength
- Abrasion resistance
- Chemical resistance

The tests shall be done as per MOSRT&H/Appendix 1000.1 Specifications.

System at a Glance is given in Table 6.4

Table 6.4

S.No.	Parameter	Requirement
1.	Pre-treatment (surface preparation)	Sand blasting to the near white metal
2.	Primer coat	To be given within 4 hour of sand blasting
3.	Sealer coat	Within 30 minutes of primer coat, this should be given. Thickness 150 microns \pm 25 microns
4.	Air curing	Six hours before use in the work.
5.	Continuity of coating	No defects such as cracking, bulging, peeling, no rust mark. Inspect visually.
6.	Adhesion of coating – Test	Coated bars are bent @ 120° around a mandrel. No peeling or cracking should be observed on outer radius.
7.	Stacking	Stack bars on buffer material
8.	Cutting, bending, welding	Coated bars can be cut and bent. Cut ends and weld portion should be treated with same formulation.
9.	Bond Strength	Shall not be less than 85% of Bond Strength of uncoated bar.

Performance Guarantee

This type of protective coating should be executed only through authorized technical applicators of standard manufacturers who have requisite work experience for having carried out similar type coating works. However, the Employer/department may take performance guarantee from the agency responsible for the execution of the work for a period of 5 years or so, which is the minimum time for defects due to corrosion of embedded steel to start becoming apparent.

6.1.4. Patented/general systems

There exist patented/other systems for protective anti-corrosive treatment to steel reinforcement, viz. Epoxy- Phenolic rebar coating system, epoxy based coatings, polyurethane coatings etc. developed by specialists as a patented product. These systems are not amenable to generic specifications as also the processes acceptance tests, etc, do not conform to prescribed published standards.

The experiences on performance of such systems are varied and not well documented. Clients performance certificates are also not forthcoming. As such use of such systems can be accepted by an Employer on case to case basis after satisfying oneself about its adequacy, criteria for acceptance of both the material specification, process control and final product. These products have to be durable, carbonation and UV resistance, etc. for which specific norms for acceptance need to be obtained from the Manufacturer/Patentee and considered appropriately. The Employers are advised to consider the above aspects while accepting such products and also insist on a performance guarantee of at least 5 years to cover the risk of malfunctioning of the product.

6.2. Pre-stressing Steel

The pre-stressing steel used in pre-tensioned and post-tensioned bridge structures is susceptible to pitting corrosion, crevice corrosion, stress corrosion; cracking and hydrogen embrittlement. It can also suffer severe corrosion under aggressive environment like chlorides, sulphates etc. Failure and distresses noticed in pre-stressed bridge structures are traced to premature corrosion of prestressing steel. The pre-stressing steel therefore requires protection against corrosion at various stages right from manufacturer's end till it is encased in cement grout.

6.2.1. Manufacturing stage

This involves surface modification of pre-stressing steel by controlled shot peening. It is observed that introduction of surface compressive stress can effectively increase the resistance to stress corrosion cracking of many engineering materials. In the case of pre-stressing steel, surface rolling is reported to be beneficial. The effect of controlled shot peening on the stress corrosion behaviour of pre-stressing steel has been studied by CECRI in 20% ammonium thiocyanate solution. (As per fib, this solution is the most convenient and reliable one for the study of hydrogen induced stress corrosion cracking of pre-stressing steel. Pitting potential of shot peened steel has been studied in 0.04 N NaOH containing 500 ppm of chloride.) The results have shown that controlled shot peening can increase its resistance to hydrogen induced stress corrosion cracking.

6.2.2. During transportation stage

It is necessary that the transport vehicles carrying the steel from manufacturer's factory to the site/godown must be clean and free from contaminated materials. (It has been reported that the steel transported in trucks carrying sulphur got adversely affected which lead to premature corrosion at the early stage.) The steel should be duly wrapped by gunny or plastic cover to avoid contact with any contaminants. The transport should be properly covered so that water/moisture does not come in contact with the steel.

6.2.3. Temporary corrosion protection (storage etc.)

Depending upon manufacturing process, the surface of the steel is covered with intentionally applied or production dependent remainders which provide some temporary corrosion protection.

Cold-drawn wires/strands receive a preparatory layer consisting of phosphate lime borax or similar as a lubricant which consists mainly of stearate or drawing soap. Under normal, non-aggressive environmental conditions, this layer can protect the steel from corrosion for several days upto some months.

In addition, soluble oil may be sprayed on. The effectiveness of such oil is for a short time till the layer does not get evaporated. However, the shelf life under normal environmental conditions can be considerably increased and a rust film is avoided. Tendons in non-injected sheathings can be protected for longer periods (at least six months), if there is no circulation of humid air. It is advisable to place some reference steel which can be taken out for inspection, and re-flushing the sheathing with additional oil. Such steel would help in monitoring the situation in the environment of pre-stressing steel.

By use of this technique, steel has been protected for over five years without corrosion setting in.

Water-soluble oil may get wiped off during handling or during insertion into the cable duct because the water-soluble oil has poor abrasion resistance. The oil is to be removed from the steel surface before cement grouting since the presence of oil will adversely affect the bonding between cement grout and the steel.

Vapour Phase Inhibitors (VPI) are also used for temporary protection by blowing it into the sheathing ducts. The inhibitors can also be used when pre-stressing steel is stored at site before installation in the duct.

Vapour Phase Inhibitor chemicals such as beta-naphthal dinitro benzene, diazolo ammonium nitrite are toxic. To mitigate these drawbacks, temporary protection can also be effected by keeping the steel immersed in duct in a passivating/alkaline solution, which can be flushed out just prior to cement grouting.

Studies made at CECRI show that the pre-stressing steels could be temporarily protected during storage at site or while lying in conduit (before grouting) by keeping it immersed in inhibitor solution covered by the Indian Patent.

In closed ducts, dry, clean protective gases like nitrogen may be used for temporary corrosion protection.

Before fabrication of the cables and installation in the duct, the steel can be stored in a closed shed wherein temperature is controlled and humidity is minimized.

6.2.4. Long term in-structure

6.2.4.1. Galvanizing

Zinc provides active corrosion protection. It is sacrificed to corrosion as long as there is still metal available on the steel surface. Corrosion protection by zinc primarily depends upon the consumption rate and the remaining thickness of the zinc layer. Good zinc coating moreover reduces the danger of fretting corrosion in case of fatigue loads. Band and cracked zinc layers reduce fatigue strength.

Galvanized pre-stressing steels have already been standardized in a draft of ISO 14568 prepared on the basis of French Standards NF A 35085. This standard contains tolerances for the thickness of the zinc coating. Usually pure zinc is used for galvanization. The standard ASTM A 475-95 "Standard Specifications for Zinc coated Steel Wire Strand" is not written for pre-stressing steel but may be used for comparison.

If the pre-stressing steel is anchored with wedges, care has to be taken that the zinc layer is not so thick that the teeth of the wedges are completely filled with zinc.

Zinc is not only dissolved by acids but also reacts with solutions of high pH value. Zinc is partly dissolved by fresh cement mortar, but this has no major influence on effectiveness of galvanizing, because the major protection comes from zinc which chemically combines with steel in thin layers from surface inwards with varying percentage.

6.2.4.2. Epoxy coating

Following the example of epoxy coating reinforcing steels, prestressing bars and strands have been coated, however, to a much higher quality. The layer thickness is about 0.7mm. Standard tests are carried out on coated steel. ASTM A 882 M – 91 code shall be followed.

Preparation of steel surface is of importance. The application of a primer to improve bond between steel and epoxy, a resin layer is recommended. During manufacture of epoxy, it should be ensured to get a composition, which gives a cover as tight and ductile as possible. The strand is opened during coating and closed again thereafter so that there are no voids even in the core region.

Epoxy coating withstands high temperatures upto 200°C.

6.2.4.3. Protection of HTS by grease, wax, etc.

6.2.4.3.1. Protection of unbonded stressed tendon by grease grouting

Unbonded tendons have the pre-stressing steel permanently protected against corrosion by a properly applied coating. Whereas grease, wax, plastics, bitumen and other materials have been used on the basis of present experience, it is recommended that the protective compound should take the form of grease, which also assists the free movement of the tendon during stressing.

There are various types of grease and most of them are not impervious to moisture, so that soluble harmful elements can migrate through the grease to reach the tendon unless the outer covering is completely waterproof.

Anti-corrosive grease should fulfill the following requirements:

- The sheathing should be completely leak-tight and continuous for the full length of the tendon.
- The grease should adhere to and be continuous over the entire tendon length to be protected and should completely fill the sheathing without air pockets.
- The grease should remain ductile and free from cracks and should not become fluid over the anticipated temperature range during fabrication, transport, storage, installation, concreting, tensioning and while in service.
- The grease should not contain harmful impurities such as chlorides, sulphides or nitrates.
- It should provide a self-healing film and displace water.
- It should have reserve alkalinity for long-term acid neutralization.
- They contain no solvents to leave trapped residue that might become a fire hazard to react with the sheathing material
- Anti-corrosive grease is considered to give required degree of protection if it meets the performance specification given in the following Table 6.5 :

Table 6.5

Test	Criterion	Test Method
● Dropping point	Min.373 K (100°C).	ASTM D - 566 ISO - 2176
● Water Soluble Ions: Chlorides Nitrates Sulphides	Max. 20 ppm Max. 20 ppm Max. 20 ppm	ASTM D - 512 ASTM D - 992 ASTM D - 1255
● Oil Separation Test Period - 7 days @ 40°C	Max. 5% by weight but lower value (say 3%) preferred.	DIN - 51 - 817
● Corrosion	Grade 7 after 1000 h. Grade 0	ASTM B - 117 DIN 51 - 802
● Oxidation stability	Maximum 0.06 Mpa after 100 h. Max. 0.2 Mpa after 1000 h.	ASTM D 492 - 70 DIN 51 - 808

- **The Application**

The permanent protective material must be applied uniformly to all surfaces and should penetrate the interstices as far as possible. The tendon is usually passed through a container, which is kept full of the compound at a slight pressure. The diameter of the outlet orifice controls the residual thickness of the protective film, which should be the minimum necessary to allow the tendon to move freely within the sheathing.

6.2.4.3.2. Protection of bonded tendons

The pre-stressing steel in cables is normally protected by grout injected into the conduit after completion of pre-stressing operations. This grout provides a bond between pre-stressing steel and concrete of the structure. The diameter and shape of the conduits must be such as an adequate cross section is provided for the grout passage. The material and configuration of the conduit must be such that bond forces can be transferred from the grout to the surrounding concrete. The properties of the conduit materials must be such that no corrosion attack of the pre-stressing steel is induced.

The unsatisfactory performance of bonded tendons noticed in latter half of 20th century was due to defective grouting, which left large cavities in the ducts. The cavities were due to inadequate viscosity, trapped air, void created by bleeding, differential water contents in different segments of hardened grout and injurious salts. Presently very effective methods of grouting making use of zero bleeding grout and admixtures to reduce water cement ratio (and hence free water) or grouts of tixotropic properties have been developed.

The grouting should be carried out as specified in IRC:18-2000.

6.3. Sheathing Ducts

One of the causes of corrosion of pre-stressing steel is penetration of moisture to the tendon through the sheathing or at the anchorage. It is therefore necessary that sheathing should be watertight, strong, and resistant to abrasion and damage during transportation, installation and concreting. It should be free from pinholes and should be continuous along its length. Ducts made from High Density Polyethylene or Polypropylene satisfies various requirements, and additionally it electrically insulating. They are, therefore, most suitable as sheathing materials. Both materials are tough, durable and non-reactive. HDPE is more flexible and less liable to embrittlement at extremely low temperatures, while polypropylene is more stable at high temperatures.

Ducts made from galvanized steel are also used in preference to those made from bright metal. These help to reduce friction and achieve extended corrosion protection to prestressing steel.

6.4. Anchorages

The pre-stressing anchorage is normally embedded in the concrete at such a depth that the concrete provides adequate corrosion protection. At the stressing end, however, it is necessary to gain access to the tendon, and this is usually achieved by means of local pocket, which is filled with mortar after the tendon has been stressed and trimmed to length.

The anchored end of the tendon is the most vulnerable location for corrosion attacks especially as the spaces between wires of the wires forming a strand tendon, form capillaries, which allow moisture to gain access to the most highly stressed parts of the tendons.

After it has been stressed, the tendon should be cut to length. The length of the tendon left protruding from the anchorage should be in accordance with the anchorage manufacturer's instructions and the amount of cover to the end of the tendon should be the minimum specified for corrosion protection, depending on the degree of exposure of the structure.

As soon as possible after the ends of the tendon have been cut to length, the stressing pockets should be filled with a low shrink mortar after the sides of the pockets have first been coated with a resin-bonding agent.

In no circumstances should the grout or mortar used for pocket filling contain chlorides or other chemicals known to be deleterious to the pre-stressing steel.

Before the stressing pocket is filled with mortar, the end of the tendon and the gripping part of the anchorage should be completely sealed against moisture. Either of the following methods can be used:

- (a) The exposed portion of the tendon and the gripping part of the anchorage should be coated with a material that will give permanent protection against the entry of moisture by suitable materials, which includes epoxy resin compounds.
- (b) The vulnerable parts should be coated with same corrosion protective material that is being used elsewhere on the tendon. In this case, however, it is necessary to prevent the material from being displaced or damaged during subsequent operations.

6.5. Surface Coating of Concrete

For very aggressive atmosphere coating is essential. In repair work, concrete coating has its utility. If used for new concrete two coats are recommended for new work to allow concrete to 'breath'. Whenever it is used, the following recommendations hold good.

There are number of approved coating systems available in the country and the manufacturer's recommendations may be followed.

- Acrylic Elastomeric Coating
- Epoxy polyurethane Painting system
- Epoxy Phenolic coating system

All surface coatings have limited life and require re-painting at interval varying from 3 to 5 years depending on the type of treatment. Diurnal heating and cooling, UV exposure, surface abrasion (water or wind driven sand) are some of the causes apart from stability (life) of the system itself. This factor should be carefully considered while deciding upon the surface treatment.

6.5.1. Acrylic elastomeric coating

The System :

It comprises water based (solvent free), conforming to code and specification, acrylic polymer modified with selected mineral fillers applied over the prepared surface to form an elastic elastomeric protective membrane. The coating should have anti carbonation and water vapor diffusion property and should be resistant to action of

UV radiation. It should be waterproof and capable of bridging crazings and cracks. The coating protects exposed faces of the superstructure and portions of the substructure above water level of concrete bridges from the aggressive action of industrially polluted and marine/saline environment. The shelf life for such coatings shall not be more than 6 months.

It is necessary that the system should be capable of protecting the pre-stressed and reinforced concrete from all deleterious elements such as chlorides and sulphates. The protective treatment should allow excess water vapor in the concrete to evaporate out (breathing) without rupturing itself due to vapor pressure. The protective system itself should not deteriorate from exposure to UV rays and weathering.

The Acrylic Elastomeric coating system shall satisfy the following requirements given in **Table 6.6 :**

Table 6.6

Sl. No.	Parameter	Requirement	Reference
1.	Specific Gravity	1.35 to 1.40	IS:345
2.	Solid contents	$70 \pm 3\%$	IS:345
3.	UV resistance	No colour change	ASTM-G-53/ DIN-EN-150-105
4.	IR – Spectrum	As per Acrylic Polymer	IR – Spectro meter standards
5.	Adhesion with concrete	1.5 N/m ²	ASTM-D-4541- 02/ DIN500014
6.	Dry film thickness	200-225 Microns (for minimum 2 coats)	
7.	Coverage	400- 450 gm / m ² (2 coats)	
8.	Physical properties Diffusion resistance against Carbon dioxide	Equivalent air layer thickness $S_D \text{CO}_2$ shall be > 50m	DIN 53122Part -I
9.	Diffusion resistance against water vapor	Equivalent air layer thickness $S_D \text{H}_2\text{O} < 4 \text{ m}$	DIN 52615
10.	Water proofing characteristics	Percentage reduction in flux should be > 50%	
11.	Re-coatability	Min. 2 h to 72 h or as per manufacturers specification with the approval of the Engineer in charge.	

The primer shall satisfy the following requirements

- (i) System - Single component universal polymer primer
- (ii) Base - Acrylic Resin dispersion
- (iii) Curing - Air Curing
- (iv) Colour - Milk white, transparent application
- (v) Shelf life - 6 months from date of manufacturing in tightly sealed container.
- (vi) Coverage - 75 – 100 gm/m² (depending upon smoothness and absorption of concrete surface)

Quality Assurance

The Acrylic elastomeric material should be tested in GOI accredited laboratories where such laboratories are available, otherwise in other standard laboratories where similar facilities exist for properties specified above. Approval of the material can be decided on the basis of these test results for each parameter initially. On the basis of test results approval of the material can be decided by Engineer-in-Charge. Random samples during execution should be taken from consignments brought to site to verify that the test results match with the earlier certificates produced before approval of the product. Both the two test results (prior to approval and during execution), shall conform to the requirements as per Table I, failing which the consignment shall be rejected. It should be made mandatory that the stock register for the materials should be maintained at site and signed by the Engineer-in-charge periodically.

Surface Preparation

The concrete surface shall be free from all adhesion inhibiting substances such as oil, grease release agents as well as laitance and dust. The surface shall be cleaned by wire brushing, mechanical scraping and any loose material shall be removed by chiseling with small hammer and washed with clean water. The sub-strata shall be structurally sound for effective bonding of the acrylic polymer with the concrete surface. All pin holes shall be filled with non-shrink polymer modified fine repair mortar.

Application

After preparing the surface and filling the pin holes, primer coat (75-100 gm/sqm.) shall be applied with brush/ lambskin roller/spray gun and shall be cured for 60 minutes or as specified by the manufacturer.

Subsequently, 1st and 2nd coats of polymer coating shall be applied with brush / spray gun/ roller keeping the time between coats not less than 2 h and not more than 72 h. Consumption per coat shall be 200- 225 gm/m². The total dry film thickness of the protective coating for all coats shall be in the range of 200 – 225 microns. It is recommended that wet film thickness may be measured at a number of selected locations at the time of application with painting gauges. For measuring the dry film thickness, suitably located painting gauge shall be used. At least one gauge shall be located on each face of superstructure in each span but not less than one gauge/100 sqm. For the given solid content in the application, the dry film thickness to wet film thickness ratio should be established by prior testing in the laboratory using appropriate panels like glass plates, flat concrete slabs, steel plates (300 x 300

mm) with similar coatings. Alternatively, the dry film thickness may be calculated from the measured wet film thickness by multiplying with the solid contents per unit volume.

The specifications could be applied to new bridge works immediately after completion as also to old existing bridges after suitably repairing/rehabilitating the concrete surface.

Performance Guarantee

This type of protective coating should be executed only through authorized technical applicators of standard manufacturers who have requisite work experience for having carried out similar type coating works. However, the Employer / department may take performance guarantee from the agency responsible for the execution of the work for a period of 5 years or so, which is the minimum time for defects due to corrosion of embedded steel to start becoming apparent.

6.5.2. Epoxy polyurethane painting system, epoxy phenolic protective system, etc.

These are patented systems for protective coating to concrete, developed by the promoters based on their own formulation. No generic specifications are available, unlike the other system, which conform to published standards. As such each of these systems have to be accepted by the Employer based on the assessment of the performance of the product, backed by Client's certificates, acceptance tests as per published standards, pertinently to cover the material, processes, carbonation resistance, crack bridging properties, UV resistance, etc. In all such cases the Employer is advised to insist on a performance guarantee for at least 5 years to cover the risk of non-performance of such products. However, some broad guidelines as available are given below :

(a) Epoxy Polyurethane Painting System

The System

This system is applied on the concrete surface and helps in controlling carbonation and weathering effects. The system comprises of 4 coats such as primer coat, middle coat and top coat all these are epoxy based. The fourth coat is finish coat of polyurethane and is recommended wherever the surface or part of the structure is exposed to ultra violet radiation.

Before application, the substrata is thoroughly cleaned to remove the dust, hardened cement slurry, oily residues etc., by scrubbing with coarse wire brushes, grinding or sweep blasting methods depending on site requirements. Residual amounts of de-moulding agents, curing agents should be completely removed. Any cracks, crevices or surface blemishes should be treated with sealant prior to primer application. The subsequent coats shall be applied by brush or spray with an interval of 24 hours between the two coats.

Application

Concrete surface to be painted shall be allowed maturation time of minimum 28 days before applying primer coating. Primer coating shall be applied to the cleaned surface after surface preparation within the pot life. After air-curing, intermediate and top coatings should be applied with time lag as per manufacturer's specifications. The coating shall be applied by brush or air-less spray gun.

The paint application should aim to achieve minimum dry film thickness. Some typical thicknesses used are as under:

- Primer : 100 micron
- Middle coat : 100 micron
- Top coat : 120 micron
- Finishing coat : 40 micron

The DFT (Dry film thickness) shall be measured on 100 x 100 mm steel plate attached with epoxy on the concrete surface at the rate one per 10 m².

Performance Guarantee

This type of protective coating should be executed only through authorized technical applicators of standard manufacturers who have requisite work experience for having carried out similar type coating works. However, the Employer/department may take performance guarantee from the agency responsible for the execution of the work for a period of 5 years or so, which is the minimum time for defects due to corrosion of embedded steel to start becoming apparent.

(b) Epoxy Phenolic Protective Coating system

The System

It consists of providing and applying epoxy phenolic coating in three layers with an interval of 12 to 24 hours between the coats over a prepared concrete surface.

The coating is compatible with concrete since the co-efficient of thermal expansion of film is nearly equal to concrete sub-strata. Due to this, the adhesion between concrete sub-strata and the coating film is retained even at higher fatigue limits as evidenced from technical data listed below. The Epoxy Phenolic Protection coating is breathable as its W.V.T. (Water Vapor Transmission) is < 0.15. It is a 3 coats system which gives total D.F.T. of 200 ± 10 microns.

Engineering Properties

- D.F.T. (in 3 coats) : 200 ± 10 microns.
- Adhesion (N/mm²) : > 2.5 (Conforms to BS-3900-E-2-70)
- Elongation (%) : >15 (Conforms to ASTM D-2370-73)
- Tensile strength (N/mm²) : >15 (Conforms to ASTM D-2370-73)
- Water Vapour transmission : < 0.15 (Conforms to ASTM D-1653-74)
(mg/cm²/mm/24 h)

Application

(a) Surface Preparation

Mild sweep blasting on concrete shall remove laitance, loose cementitious materials, oil residues etc. Joint and crack sealing shall be carried out with appropriate putty. It is not necessary to fill in blow holes or pug holes since 3 coats Epoxy Phenolic Coating application shall cover these variations.

(b) Methodology

Consists of application of the coating system after thorough mixing on clean and dry surface followed by subsequent coats applied by brush or spray maintaining a time interval between two coats at 12 to 24 hours.

(c) Storage Handling

Instructions from the manufacturers regarding the use of product, storage, health precaution of workers, cleaning of tools etc., shall be followed.

Performance Guarantee

This type of protective coating should be executed only through authorized technical applicators of standard manufacturers who have requisite work experience for having carried out similar type coating works. However, the Employer/department may take performance guarantee from the agency responsible for the execution of the work for a period of 5 years or so, which is the minimum time for defects to start becoming apparent.

6.6. Steel Bridges

Protective measures for steel bridges are described in detail in another guideline "Retrofitting, Repair and Rehabilitation of Steel Bridges", SP:74-2007 which may be referred.

6.7. Cathodic Protection

Cathodic protection, which is commonly used to protect buried steel pipelines and other applications is not normally used in bridges. Piers and foundations in marine environment have been protected by C.P. system on some important projects. However, by and large, the design, installation and operation of the C.P. System is a highly specialized job and reference has to be made to the experts/specialist literature for use of the same.

The basic concept in CP system is that an anode is introduced onto the surface of the concrete as externally electrically connected sacrificial material. Application of a DC voltage between anode and cathode or the difference of the electro-chemical voltage between dissimilar metals creates a current through the concrete via the pore water which acts as an electrolyte. Electro-chemical reactions take place at the steel surface (the cathode) and the anode.

For ensuring correct protection of all embedded steel it is essential to have electric connectivity between all embedded steel. This has to be ensured at time of construction itself by either welding steel to each other, or by providing sturdy electrical connectors.

The cathode reaction generates hydroxyl ions, increasing the alkalinity of the concrete around the steel and helping rebuild its passive oxide surface layer (which could have been broken down by chloride attack). Chloride ions are negatively charged and will be repelled by the cathode. They will tend to move towards the anode. An excessively negative cathode voltage can lead to the evolution of hydrogen, with the attendant dangers of hydrogen embrittlement of rebars. This is of concern for H.T. steels used for pre-stressing. For this reason cathodic protection is not normally applied to pre-stressed concrete structures.

Anodic reactions include the formation of oxygen and chlorine gas. The alkalinity formed at the cathode is consumed at the anode, potentially leading to acid attack and carbonation in the vicinity of the anode.

The conductive layer on the surface of concrete which forms the anode can be created in a number of ways as under:

- Conductive coating systems
- Thermal sprayed zinc
- Fixed anodes commonly comprising a mesh of titanium or carbon based material fixed to the concrete surface and covered by overlay.

All these systems require electrical connections to distribute the impressed current across the anode, a DC power supply and an associated control system, together with embedded monitoring probes providing data by which adjustments can be made to the voltage and current applied.

Installation of CP system in general will be divided into a number of zones, each being treated as a complete system (i.e. with its own power supply). Design of the zones need to take account a number of factors such as presence of joints, variation in reinforcement provision, continuity of reinforcement and variation in moisture and chloride content.

Commissioning is a very important stage in achieving an effective and durable CP system. It provides the opportunity to perform a variety of tests and trials establishing the initial behaviour of the CP system, make adjustments to voltage and current supplies and to verify control criteria.

Once it has been established that an impressed current CP system is providing protection to all reinforcement, it is essential that the operation of the system is monitored and properly maintained. Changes occur in concrete over the first few months of operation - for example – drying out of concrete and resistivity increase due to removal of chloride. Monitoring ensures that all reinforcement remains protected.

6.8. Non-Corrodible Reinforcement

There are 3 types of non-corrodible reinforcement as under.

- Stainless steel Grade 316
- Carbon Fibre
- Glass Fibre

All the above type of reinforcements are effective in preventing corrosion, but they add to the cost of construction.

Carbon fibres are most expensive whereas glass fibres the cheapest and they do not corrode but it is possible that the adhesive used to bond the fibres in to the form of a bar may deteriorate in alkaline (carbon fibres) alkali (glass fibres) environment.

These have lower model of elasticity and their ultimate failure could be brittle as such this necessitates change and modification in the design.

Mechanical properties of stainless steel are similar to high yield strength steel and so using it does not pose any practical difficulty.

6.9. Selection of a System and Quality Control Test

6.9.1. The quality control suggested tests in different system is summed up in **Table 6.7** given below.

Table 6.7

S.No.	Item of coating	System	Tests	Remarks
1.	Rebars	A) Metallic coating	<ul style="list-style-type: none"> • Hammer test • Knife test • Thickness test • Preece test • Hydrogen Evolution • Stripping test 	Please refer table No.6.2
		B) Epoxy based coatings	<ul style="list-style-type: none"> • Test on Epoxy/Resin :should be carried out by the manufacturer as per ASTM-A-882-M-91 and certificate shall be furnished. • Bond test : should be as per IS:14653. • Tensile strength : should be as per IS:14653. 	
		C) Cement Polymer Composite Coating System	<ul style="list-style-type: none"> • Dry film thickness and Uniformity of thickness: Use Elcometer • Adhesion • Bond strength • Abrasion Resistance • Chemical Resistance 	
		D) Fusion Bonded Epoxy Coating	<ul style="list-style-type: none"> • Chemical resistance • Resistance to applied voltage • Adhesion • Abrasion • Impact test 	

			<ul style="list-style-type: none"> • Hardness test • Thickness of coating • Continuity of coating • Holidays • Bond test 	
		E) Epoxy Phenolic Rebar Coating System	<ul style="list-style-type: none"> • Tensile strength • Elongation • Bond strength • Chemical resistance • Fatigue test 	ASTM D-2370 BS3900-E IS:13620
2.	Pre-stressing steel	A) Epoxy coating	As specified under Rebar Coating in S.No.1(B) above	Standard ASTM-A-882-M-91 shall be followed.
		B) Grease Protection	Refer Table No.6.6 for 'Type of Tests' required on Grease	This is not a coating system. However, tests on grease are indicated.
3.	Concrete coating	A) Epoxy Painting system	Tests for Epoxy as per 1(A)	
		B) Epoxy Phenolic Coating	<ul style="list-style-type: none"> • Adhesion – As per BS.3900 • Elongation • Tensile strength • Water Vapour • Manufacturer's Certificate for above should be obtained. 	
		C) Acrylic Elastomeric Coating	<ul style="list-style-type: none"> • Specific Gravity • Solid Contents • UV Resistance • Adhesion with concrete • Diffusion Resistance against CO₂ • Diffusion Resistance against Water-Vapour 	IS:345 IS:345 ASTM-G - 53 ASTM-D – 4541 DIN - 53122 DIN - 52615

6.10. Problems and Remedies

6.10.1. General

It is possible to deal with the distress caused by corrosion of reinforcement and pre-stressing steel. However, it may not be practicable to deal with them in isolation. It is therefore, advisable to take into account problems associated with concrete defects and inadequate structural strength. With this in view, the

proposed remedies are given in three parts as under :

- Remedies for defects in concrete. (**Table 6.8**)
- Remedies related to corrosion of steel in concrete. (**Table 6.9**)
- Remedies related to corrosion of structural steel. (**Ref. 6.10.4**)

6.10.2. Review of remedies for defects in concrete

Table No. 6.8

S. No.	Problem and Objective of Repair	Possible Remedies
1.	Ingress of harmful elements in the body of concrete – To reduce or prevent ingress of such harmful agents such as water, other liquids, vapour, gas, chemicals and biological agents.	1.1 Impregnation: Applying liquid products which penetrate the concrete and block the pore system. 1.2 Surface coating: This may be with or without crack bridging ability. 1.3 Sealing and filling of cracks. 1.4 Conversion of cracks into joints wherever possible. 1.5 External cladding of panelling. 1.6 Applying suitable membranes over the surface.
2.	Moisture control: To control the moisture content in body of concrete within specified limits.	2.1 Hydrophobic Impregnation. 2.2 Surface coating. 2.3 Sheltering or over cladding. 2.4 Electro chemical treatment.
3.	Concrete restoration: Restoring the original concrete of an element of the structure to the originally specified shape and function. Restoring the concrete structure by replacing it.	3.1 Manual application of polymer or epoxy mortar. 3.2 Re-casting with concrete. 3.3 Spraying concrete or mortar 3.4 Replacing elements.
4.	Structural Restoration /Strengthening: Restoring and enhancing, Structural load bearing, Capacity of an element of the concrete structure	4.1. Replacing or adding reinforcement in the concrete. 4.2. Fixing new bonded reinforcement in pre-formed or drilled holes. 4.3. Bonding of M.S./F.R.P. Plates with expansion anchors. 4.4. Providing additional concrete or mortar. 4.5. Treatment of voids, cracks and interstices by injection of Polymer/epoxy or by suitable filler material. 4.6. External post-tensioning.

5.	Concrete vulnerable to mechanical and chemical attack. To increase resistance of concrete to physical & chemical attack.	5.1 Provide overlays, coatings or impregnation.
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6.10.3. Review of remedies related to corrosion of steel in concrete

Table 6.9

S. No.	Problem and Objective of Repair	Possible Remedies
1.	Preserving or Restoring Passivity: Creating chemical conditions in which surface of the reinforcement or pre-stressing elements is maintained in passive condition.	1.1 Increasing cover to reinforcement with additional cementitious mortar or concrete. 1.2 Replacing contaminated or carbonated concrete. 1.3 Electro-chemical re-alkalization of carbonated concrete. 1.4 Re-alkalization of carbonated concrete by diffusion. 1.5 Electro-chemical chloride extraction.
2.	Increasing Resistivity: Electrical resistivity of the concrete to be enhanced.	Controlling moisture content by surface treatments and coatings.
3.	Cathodic Control: Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction.	Limiting oxygen content (at the cathode) by saturation or surface coating.
4.	Cathodic protection	Application of Electrical potential.
5.	Control of Anodic areas: Creating conditions in which potentially anodic areas of reinforced are unable to take part in the corrosion reaction.	5.1 Painting reinforcement with coatings. 5.2 Painting reinforcement with barrier coatings. 5.3 Applying inhibitors to the concrete.

6.10.4. Remedies related to corrosion of structural steel

For remedies for structural steel IRC:SP:74:2007 "Guidelines of Repair and Rehabilitation of Steel Bridges may be referred.

7. INVESTIGATION AND ASSESSMENT

7.1. Objective

It is necessary to establish clearly the aims of the investigation proposed to be done. It has a bearing on choice of test method, the extent and location of the tests and the way in which results are to be handled. Liaison between the engineer, consultant and specialist is essential to decide the aims and arrive at a formulation of the test programme in the early stage. Engineering judgment is inevitably required when interpreting results, but uncertainties can often be minimized by careful planning of the test programme.

Basic aims of such investigation for corrosion affected bridge structures shall be as under:

- Assessment of cause and extent of deterioration as a preliminary to the design of repair or remedial schemes.
- Assessment of structural integrity or safety following material deterioration such as caused by corrosion, fire, fatigue etc. Such assessment will concentrate upon identifying the presence of internal voids or cracking, materials likely to cause disruption of the concrete and the extent or risk of reinforcement corrosion. Carbonation depths, chloride concentrations cover factors relating to corrosion.
- Another aim of this investigation is to predict future performance on the basis of tests and assessment done.
- There is not just a single method which can yield complete information about corrosion and related aspects. As such, engineering judgement is used to assess the damage caused by corrosion.

7.2. Visual Inspection

Visual inspection by an expert who has previously handled similar situation is a most reliable preliminary essential step in the assessment procedure. The degradation processes are likely to become apparent much before the load bearing capacity gets seriously damaged and many advance warning signs would be available like spalling, rust stains, corrosion and other types of cracks, deflections, malfunction of joints and hinges, deformations, performance of bearing, drainage system, waterproofing etc., should be observed. It is necessary to provide proper access to various components of the bridge to ensure a thorough inspection. Temporary scaffolds or cradles may have to be erected. The use of mobile inspection unit comes very handy for such purpose and is a must for bridges with great height and when water is present in the bed.

Apart from the above, special attention should be paid to the following factors during this inspection:

- Deterioration of the concrete and its skin.
- Verification of information gathered during planning of assessment.
- Old coatings, impregnations and protections.
- Presence of cracks, their appearance and pattern.
- Exposed rebars, cables etc.
- Deformation of structure.

The data and information so collected should be presented graphically.

7.3. Types of Cracks

Different types of cracks occur in concrete, some of them occur immediately while others take some time to develop. These are:

(i) **Plastic settlement and plastic shrinkage**

These occur within the first few hours after initial set due to excessive bleeding and rapid early drying respectively resulting into loss of bond to bars and exposure of reinforcement.

(ii) **Early thermal contraction cracks**

These occur within the first few weeks in thick walls and slabs due to excessive heat generation and results into exposure of reinforcement, seeping and leaking.

(iii) **Drying shrinkage cracks**

These occur in walls and slabs and take few weeks to years for development due to loss of moisture. They create the path for seepage and leaking.

(iv) **Cracks due to corrosion of reinforcement**

These take several months and years and are predominant in corners, edges and end portions of beams. They lead to rapid deterioration of concrete.

(v) **Cracks due to alkali aggregate**

These can develop at damp locations within few months or years; on account of internal bursting forces due to expansive reaction of certain aggregates in high alkali content situations. Such cracks can lead total disintegration of concrete.

(vi) **Frost damage**

These can occur at any age in porous concrete and result in loss of section in compression.

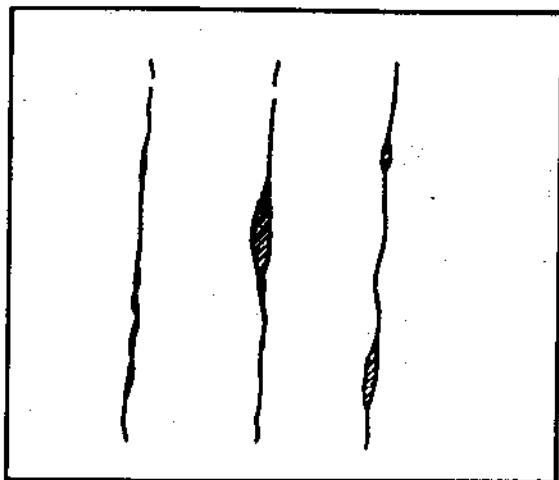
(vii) **Sulphate attack**

These take many years to develop, mostly near or below ground level on account of sulphate salts in damp ground, which can lead to disintegration and failure of foundation.

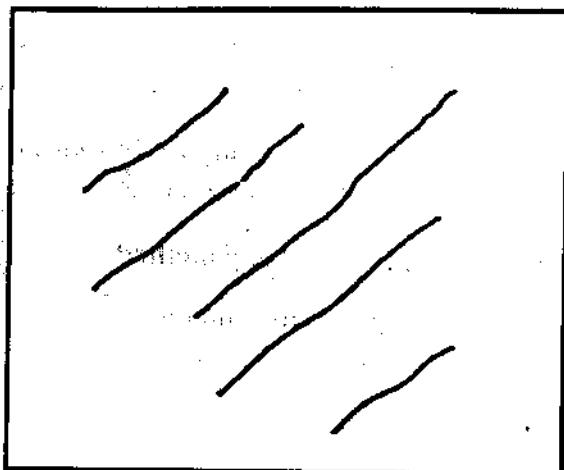
(viii) **Physical salt weathering**

This requires many months to many years for development, in the inter-tidal and splash zones or just near ground level in desert terrain, leading to deposition of salts and volume change and final disintegration.

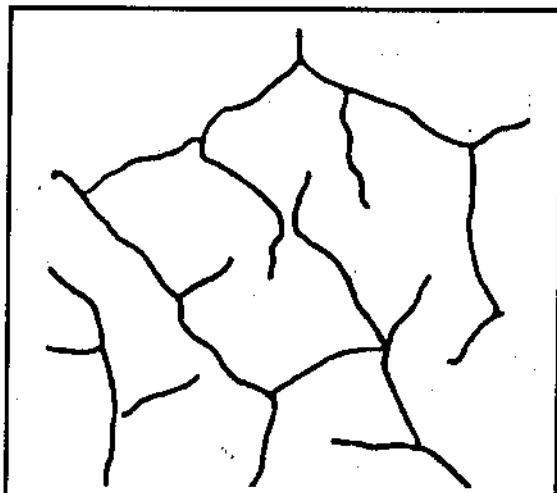
Reinforcement corrosion is usually indicated by splitting and spalling along the line of bars possibly with rust staining, whereas sulphate attack may produce a random pattern accompanied by a white deposit leached on the surface. Typical cracks are shown in Fig. 7.1 given below:



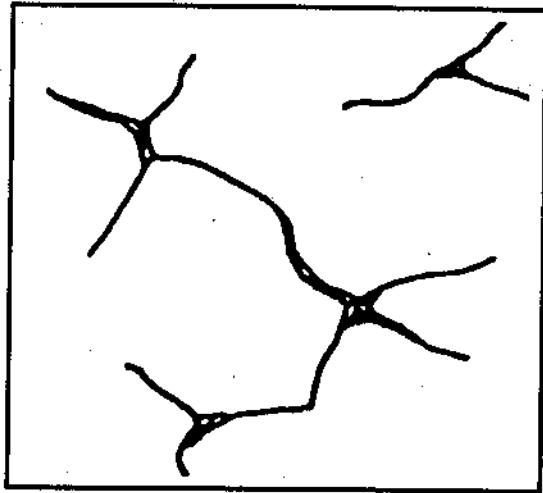
(a) Reinforcement



(b) Plastic Shrinkage



(c) Sulphate Attack



(d) Alkali Aggregate Reaction

Fig. 7.1 : Typical Crack Pattern

7.4. Tests for Assessing Corrosion of Reinforcement

Generally, assessment methods for corrosion of reinforcement can be divided into 2 types as under:

- (A) Traditional (Non-Electro-chemical) Method.
- (B) Electro-chemical Method.

7.4.1. Traditional method

The traditional method of assessment of reinforcement is to expose the rebars in critical areas by breaking the concrete cover locally and examining the type and amount of corrosion products on the steel surface and the amount of iron removal and loss in cross-section after cleaning of the rebars.

The most critical areas are often found by visual inspection of the concrete surface and hammering the concrete to search for de-laminations caused by reinforcement corrosion. However, this method will not detect corrosion in areas where it is in the early stages.

This method has a limitation as breaking of concrete to expose the reinforcement may affect integrity of the structure. It is therefore used in cases where structural integrity is not influenced significantly.

7.4.2. Electro-chemical method

It is based on electro-chemical nature of the corrosion process which is comparable to the well-known operation of a battery consisting of an anode and cathode. The methods commonly used are:

- Potential Mapping
- Resistivity Mapping

7.4.2.1. Potential mapping

As the corrosion of steel is an electrochemical reaction, i.e. it is both a chemical reaction which generates a chemical product and also a reaction which generates electrical current with a certain potential/voltage. The potential of a corroding steel surface is known as corrosion potential. The corrosion potential of steel in concrete can be measured using a standard reference electrode such as copper-copper sulphate electrode, or a saturated calomel electrode or a silver-silver chloride electrode. The test procedure for the same is detailed in ASTM C876-91. Commercial equipment for measurement of O.C.P. based on this method is available.

The steel reinforcement in concrete is located first by using a rebar locator. The electrical continuity of the reinforcement is established by exposing the rebar at two or three locations on the surface and measuring the potential difference between them with a high impedance voltmeter. If the potential difference is less than 1 mV, then the continuity is probable and if it is greater than 3 mV, the continuity is unlikely. The continuity can also be ascertained by measuring the DC resistance between two locations. A DC resistance of less than 1 ohm indicates good electrical continuity among the reinforcement bars within the structural element.

The corrosion potential of reinforcement is measured by placing the standard reference electrode on the concrete surface and connecting it to the negative (ground) terminal of a high impedance voltmeter. The positive terminal of the voltmeter is connected to the exposed reinforcement bar. The potential difference between the standard reference electrode and the steel reinforcement bar is shown on the voltmeter. The voltmeter reading is directly read as corrosion potential of the steel reinforcement at that location. **Fig. 7.2** shows the circuit diagram of measurement of O.C.P.

The interpretation of the corrosion potential values in terms of the condition of the steel surface is made based on the criteria suggested in ASTM C-876 which is given below in **Table 7.1**.

However, interpretation of a single potential value on a concrete surface based on the above criteria could lead to misleading results. Hence it has been suggested to measure the corrosion potential values on a grid pattern on a concrete surface and draw the equi-potential contours and then use the above table for interpretation of the contours. The contour map helps in identifying the corroding area and the non-corroding areas on the steel surface. A typical contour map is shown in **Fig. 7.3**

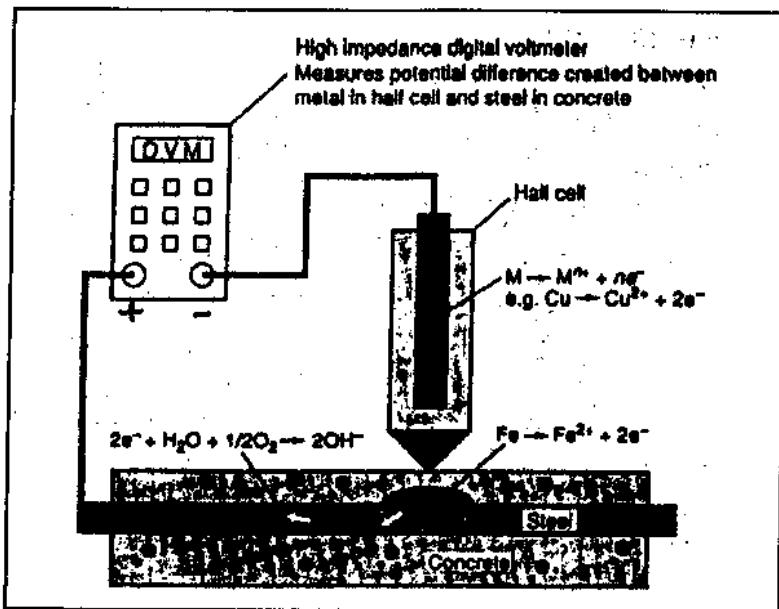


Fig. 7.2 : Circuit Diagram for Measurement of Open Circuit Potential of Steel in Concrete

Table 7.1

Cu-CuSO ₄ Electrode	Calomel Electrode	Ag-AgCl Electrode	Corrosion Condition
> - 200 m V	> - 126 m V	> - 106 m V	10% risk of corrosion (low)
- 200 to - 350 m V	- 126 mV to - 276 mV	- 106 mV to - 256 mV	Intermediate corrosion risk
< - 350 m V	< - 276 m V	< - 256 m V	90% risk of corrosion (high)
< - 500 m V	< - 426 m V	< - 406 m V	Severe corrosion

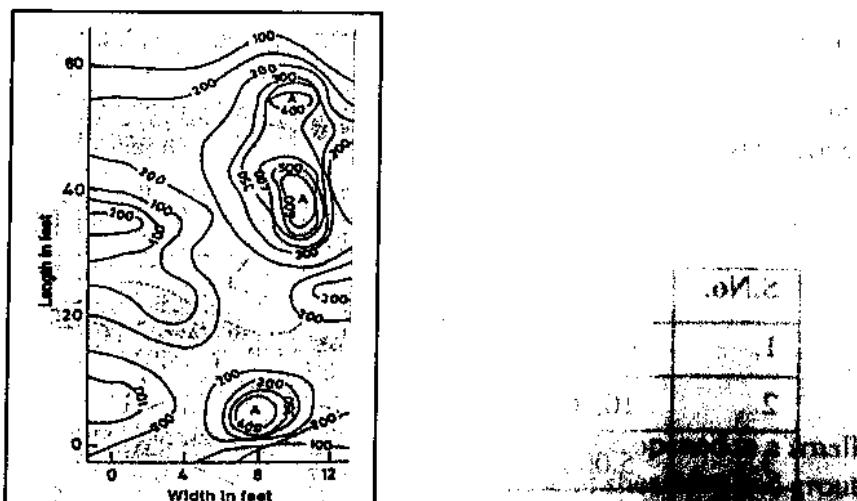


Fig. 7.3 : Typical Contour Map for Potential

It is also to note that the above criteria are applicable to structures that are above the ground and not in direct contact with soil or sea water. For the structures that are in direct contact with soil or sea water (ie. saturated structure or where the oxygen availability is limited), or in highly rainy areas where concrete above the ground also is also in a wet condition, a non-corroding reinforcement can exhibit a potential of -670 mV w.r.t. Ag-AgCl Electrode. The structures above the ground need to be wetted on the surface before testing the O.C.P. to ensure that the conducting path between the reference electrode and the steel is established. The surface wetting is necessary as it tends to get dry due to sun and wind.

The advantage of this technique is that it gives a quick indication of the condition of steel w.r.t. the corrosion. However, the limitations of this technique are that it cannot product the rate of corrosion and any attempt to correlate the potential with the rate would be futile. This technique is more suitable to measure the corrosion potential of chloride contaminated concrete. As may be noted, a good electrical contact between the reference electrode and the steel through the concrete is necessary for stable reading. The carbonated concrete exhibits higher concrete resistivity and this leads to erroneous results in the OCP measurements.

7.4.2.2. Resistivity mapping

The corrosion of a specific length of reinforcement is dependent on the algebraic summation of the electrical currents originating from the corroding sites on the steel and flowing through the moist surrounding concrete non-corroding sites. Hence the electrical resistance of concrete plays an important role in determining the magnitude of corrosion at any specific location. This parameter is expressed in terms of "Resistivity" in ohm centimeter or kilo ohms centimeter. The factors which govern the resistivity values are:

- Constituents of concrete
- Chemical contents of concrete such as moisture chloride level.
- Type of Pore - structure of the concrete.

As there are number of influencing factors as given above, it is possible that resistivity values will vary significantly over a structure. It is therefore necessary that a systematic survey over well defined grid points is suggested and a plotted map of the resistivity values can be used to assess the most potentially corroding areas. General guidelines of resistivity values based on which areas having probable corrosion risk can be identified is given in **Table. 7.2** below:

Table 7.2

S.No.	Resistivity (ohm-cm)	Corrosion Probability
1.	Greater than 20,000	Negligible
2.	10,000 to 20,000	Low
3.	5,000 to 10,000	High
4.	Less than 5,000	Very High

7.4.2.3. Corrosion risk from resistivity:

The principle of resistivity testing in concrete is similar to that adopted in soil testing. However, when applied in concrete, a few drawbacks should be realized. The method essentially consists of using a 4 probe technique in which a known current is applied between two outer probes and the voltage drop between the inner two elements is read off allowing for a direct evaluation of resistance 'R'. Using a mathematical conversion factor, resistivity is calculated as ' $\rho = 2\pi a R$ ' where 'a' is the spacing of probes. The principle of 4-probe resistivity testing is illustrated in Figs. 7.4 and 7.5 which shows a typical resistivity contour.

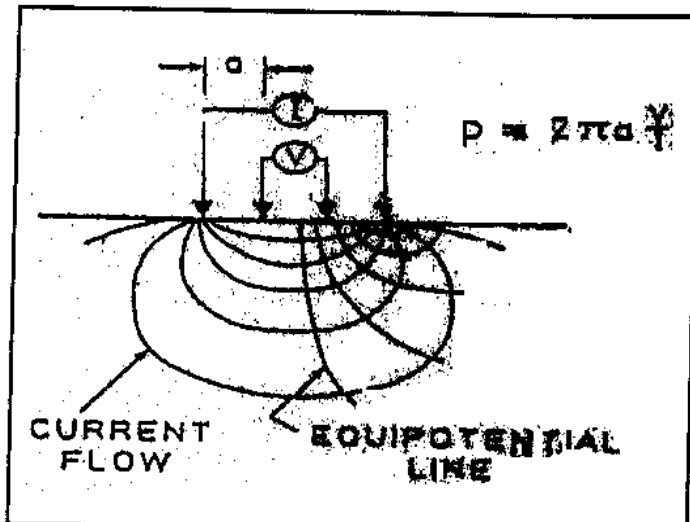


Fig. 7.4: Principle of Resistivity Testing for Concrete

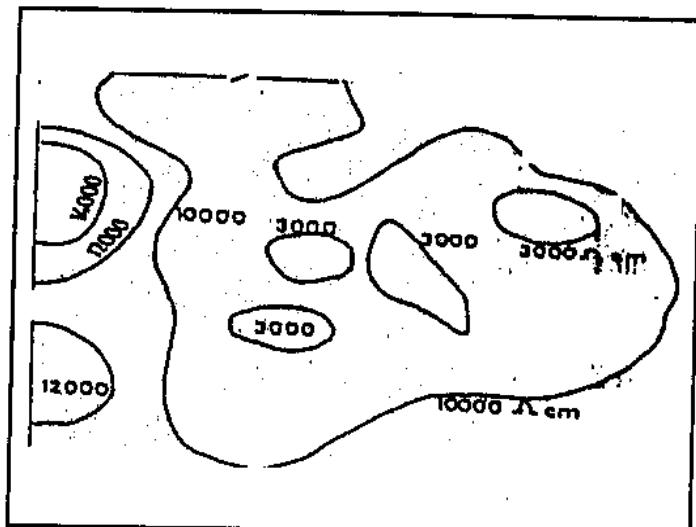


Fig. 7.5: Typical Resistivity Contour

7.5. Tests for Assessing Corrosion of Pre-Stressing Steel

Precise and well defined test to determine corrosion in pre-stressing tendons are not yet established. However, some tests are developed and are indicative about corrosion incidence of pre-stressing steel. Some tests such as visual inspection are used in certain situations and are helpful in assessment of corrosion. Tests which can be used are given below:

- Visual inspection
- Acoustic Emission
- Radiography
- Endoscopy
- Technique based on reduction of Magnetic Flux of steel.

7.5.1. Visual inspection

This is similar to inspection of rebars. The cables located near the surfaces can be opened in a small portion and inspection done to check corrosion, diameter of the wire/strand, condition of sheathing and etc. The data so collected can be used for assessing the condition of pre-stressing elements.

7.5.2. Acoustic emission

Acoustic Emission (AE) is based on the principle that acoustic (sound) energy is released when a pre-stressed wire, such as in pre-stressed concrete structures, including pavements, or in cable-stayed structures, breaks. This acoustic energy is detected by acoustic sensors (accelerometers) distributed along the key locations on the structure. Each sensor is connected through a co-axial cable to an on-site data acquisition unit. The system collects no data until triggered by an acoustic event falling within pre-set limits. On-site processing of the data at the data acquisition system eliminates most irrelevant events of no further interest. Events that meet pre-set criteria are considered as wire failures, and are recorded and transmitted over the Internet to a central processing facility where proprietary processing software is used to generate reports summarizing the time, location and classification of the recorded events. See Fig. 7.6.

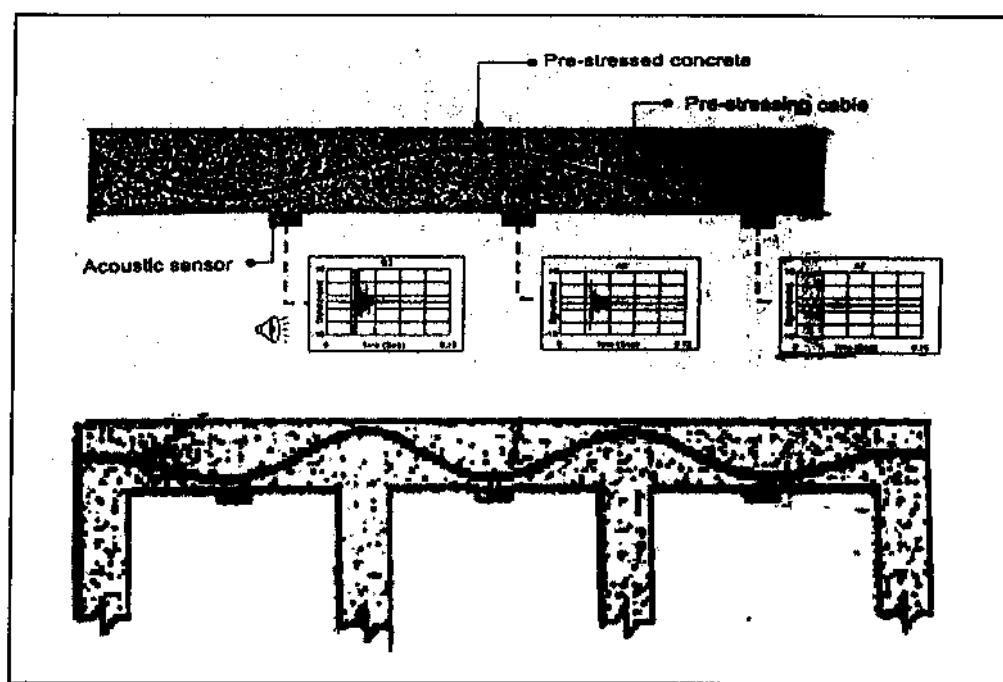


Fig. 7.6: Technique of Acoustic Emission Monitoring

The system continuously monitors the structure and records the events; hence it is often termed as continuous acoustic emission technique. The number of wire breaks help in the planning of remedial measures to be undertaken. Although the acoustic emission principle was known, it has not become popular until recently, when low-cost data acquisition system, computing hardware, powerful analytical and data management software have become available. A commercial system based on this technique is available and is being used in USA, Canada, Japan etc.

7.5.3. Radiography

In this technique, the physical condition of the reinforcement steel or pre-stressed wire or post-tensioned cable can be visually inspected in a concrete structure. It is similar to taking a X-ray photograph of an object, although in this technique, X-rays or γ -rays are used using a radioactive source. The radiation source is placed on one side of the concrete and a photographic plate (recorder) is placed on the other side. A schematic of the

ique is presented in Figs. 7.7. and 7.8 shows the likely photograph of pre-stressed concrete element tested using radiography method. Although commercial equipments are available, this technique is not widely due to safety concerns to the staff and the passers-by with the use of radioactive source of X-rays or γ -rays. It has been suggested that the traffic need to be closed and movement of people need to be limited during testing.

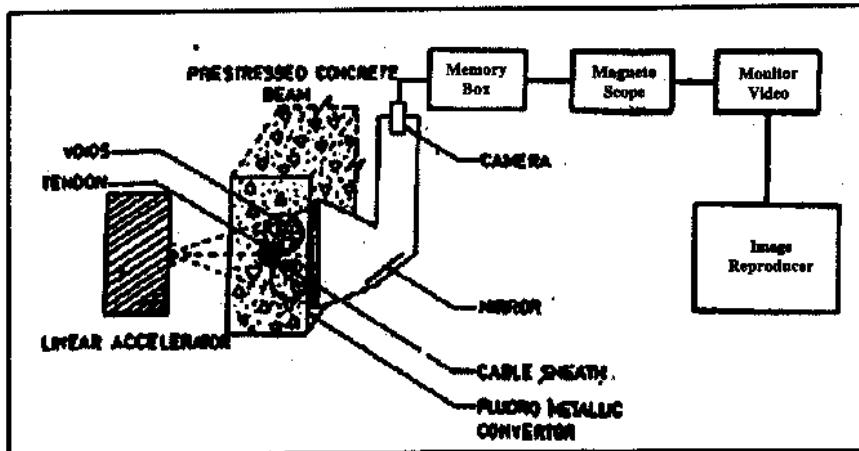


Fig. 7.7 : Radiographic Technique

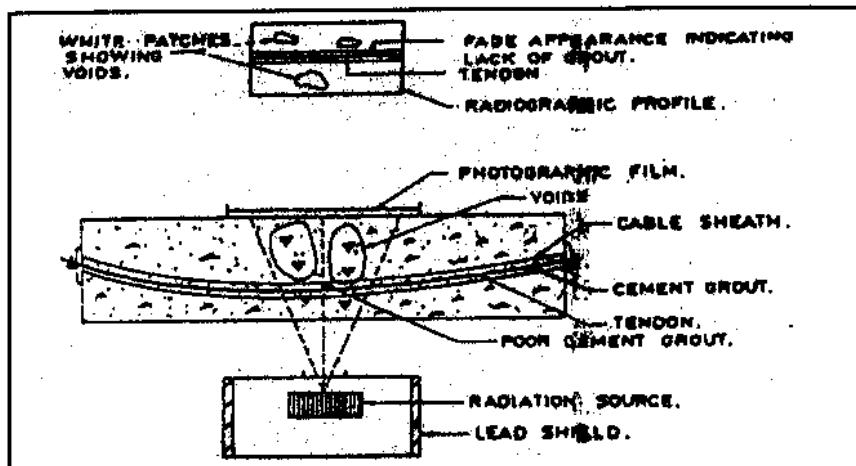


Fig. 7.8 : Testing of PSC Element Using Radiography Method

7.5.4 Endoscopy/Boroscopy

This is a low-cost technique to inspect physical condition of the reinforcing steel, pre-stressed wire or tensioned cable in a concrete structure. In this technique, a small diameter (approx. 10-15 mm.) hole is bored on the concrete surface up to the level of the steel surface and then its condition is visually inspected by pushing a rigid or flexible tube into the hole. The tip of the tube is illuminated using a light source. Commercial equipments based on this method are available. Fig. 7.9 shows the schematic of endoscopy technique. The advantage of this technique is the numerous holes made on the concrete surface and their filling with an mortar after inspection of steel.

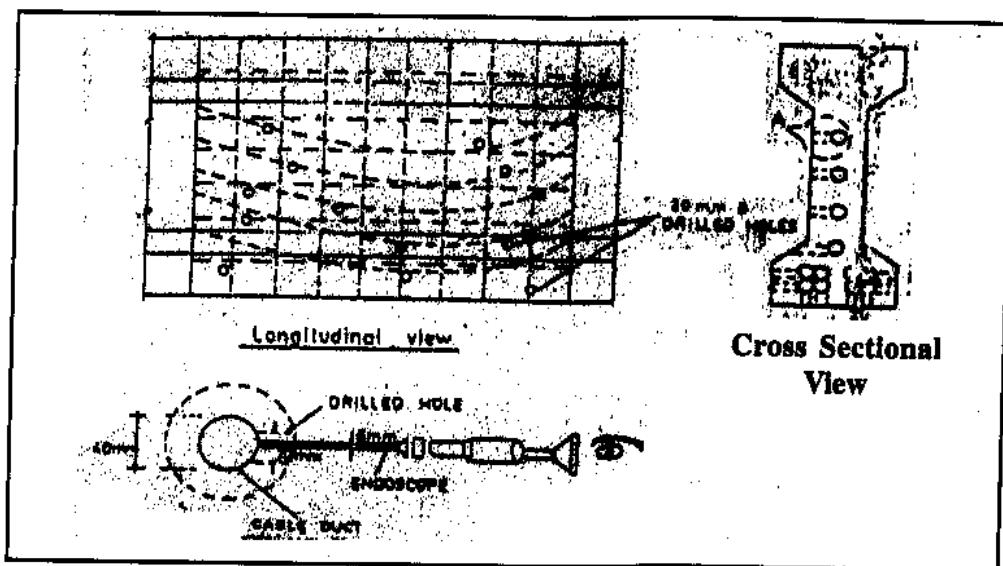


Fig. 7.9: Schematic of Endoscopy Technique

7.5.5 Technique based on reduction of magnetic flux of steel:

This method is suitable for inspection of the pre-stressing wire or post-tensioned cable in concrete structures. It is based on the principle that as the steel wire corrodes and its diameter decreases its magnetic properties change. In this technique, the tendon hidden in the concrete is located (using cover meters or GPR) and is magnetized using an exciting magnetic field H_e applied from the outside of the concrete by means of a yoke magnet. This exciting field generates a magnetization in the steel wires. Local disturbances of the distribution of this magnetization due to ruptures or reductions of the cross-section cause the emanation of a magnetic leakage flux (stray field) H_s from the member. For the generation and measurement of the stray field, a probe containing the magnetization device (yoke magnet) and the sensors is moved along the direction of the pre-stressed tendon outside the concrete surface. Fig. 7.10 shows a schematic of this method. The stray field measurement is either conducted during magnetization by the exciting field (active field) or as a residual field measurement. In the latter case, the stray field is caused by the remnant magnetization of the steel after switching off the magnetization device. In the active field measurement, ruptures of the longitudinal rebars appear as a local maximum.

7.6. Tests for Assessing Corrosion of Concrete

Contamination of concrete and other related defects can be ascertained by carrying out following tests on concrete:

- Carbonation
- Chloride content of concrete
- Electrical resistivity of concrete
- De-lamination of concrete
 - Chain Drag/Tapping
 - Grout Penetrating Radar

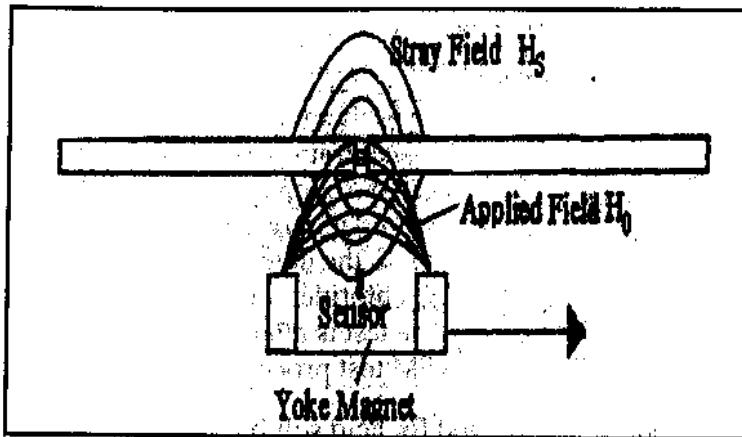


Fig. 7.10: Method of Magnetic Leakage Flux Measurement

These tests are briefly described below:

7.6.1. Carbonation

The extent to which the concrete has carbonated can be determined by sprinkling phenolphthalein solution, which is colourless, on a freshly broken surface of concrete. This solution is prepared by dissolving 1g. of phenolphthalein indicator in 100 ml. of a 50 : 50 mix of iso-propyl alcohol and water. If the colour of the solution turns purple, no carbonation is indicated and if the colour of the solution remains unchanged, it indicates that the concrete has carbonated upto that depth. The test is performed at different depths of concrete to determine the depth of concrete carbonation. However, there is a limitation to this technique. The indicator solution can detect change in pH at 8 i.e. colour changes from colourless to purple, while the threshold pH for breakdown of passive layer is 11 - 11.5 (See Fig.7.11). Hence caution is exercised in interpreting the results from the carbonation test and if required, more tests may have to be carried out for alkalinity (pH) in the laboratory.

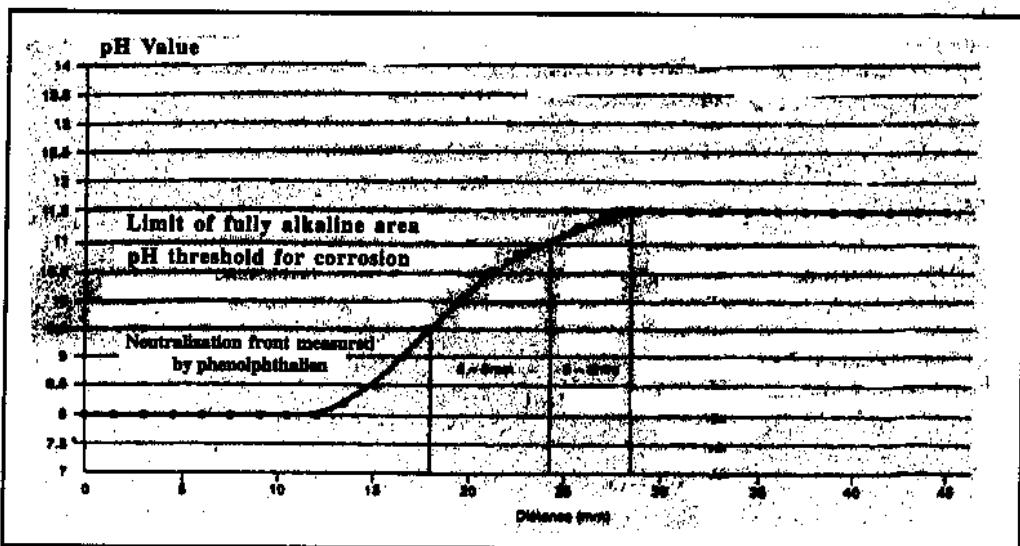


Fig. 7.11: Limit of Phenolphthalein Indicator Solution and Threshold pH for Breakdown of Passive Layer

7.6.2. Chloride content of concrete

An analysis for chloride content of concrete indicates the extent of the chloride penetration in to the concrete and the likely initiation of corrosion of steel. Steel is said to be de-passivated when the chloride front reaches the steel level in the concrete and its concentration exceeds a certain pre-determined limit. It is in this context, that the determination of chloride concentration in concrete is important. Chloride content of concrete is determined by gravimetric titration method and readily usable portable kits are available for determination of chloride content of concrete at site. In the determination of chloride ions in concrete, two types of chloride ions are identified - (i) Acid soluble chlorides and (ii) Total chlorides. However, normally it is the acid soluble chlorides that are determined as the test is relatively simple to carry out and the results are more reproducible. These tests are covered in ASTM test procedures as given below:

ASTM C 1152 Standard test method for acid soluble chloride in mortar and concrete.

AASHTO T-260 Standard test method for sampling and testing for total chloride ion in concrete and concrete raw materials.

The threshold chloride limits prescribed in various studies pertain to the acid soluble chloride content. Based on a series of experiments worldwide, the following threshold values of chloride ion have been established for initiation of steel reinforcement corrosion in un-carbonated concrete:

< 0.2% by wt. of cement	= No risk of corrosion.
0.2% - 0.4% by wt. of cement	= Low risk of corrosion.
0.4% - 1.0% by wt. of cement	= Moderate risk of corrosion.
> 1.0% by wt. of cement	= High risk of corrosion.

A chloride content upto 0.4% by wt. of cement has been generally considered safe with reference to the reinforcement corrosion. Sometimes, the chloride content is expressed in terms of weight of chloride per unit volume of concrete particularly in the case of existing structures where the records of mix proportions or cement content are not available/known. In this case, the threshold chloride content has been found to be 0.9 - 1.2 kg/cu.m of concrete. However, the ACI has imposed more stringent conditions for chloride content and the limits of chloride content as per ACI are given in Table 7.3 below:

Table 7.3

Type of Structure	Chloride content (% wt. of cement)
Pre-stressed Concrete	0.06
Conventionally reinforced concrete in a moist environment and exposed to external sources of chloride	0.10
Conventionally reinforced concrete in a moist environment but not exposed to external sources of chloride	0.15
Above ground building construction where the construction will stay dry. Does not include locations where the concrete will be occasionally wetted, such as kitchens, parking garages and waterfront structures	No limit

The IS:456-2000 stipulated the following limits on the chloride ion content of different types of concrete given in **Table 7.4** :

Table 7.4

Type or Use of Concrete	Max. Total Acid Soluble Chloride content (kg/cu.m of concrete)
Concrete containing metal and steam cured at elevated temperature and pre-stressed concrete	0.4
Reinforced concrete or plain concrete containing embedded metal	0.6
Concrete not containing embedded metal or any material requiring protection from chloride	3.0

Sometimes, the quality of concrete is tested for its ability to resist diffusion of chloride ions through it in terms of the electrical charges (coulombs) flowed through it. In this test, 100 mm. diameter concrete cores are extracted from the structure and slices of 50 mm thick are made which are then subjected to the chloride permeability test. This test is known as "Rapid Chloride Permeability Test (RCPT) and is covered in ASTM specifications as given below:

ASTM C 1202 Standard test method for electrical indication of concrete's ability to resist chloride ion penetration.

AASHTO T-277 Standard test method for electrical indication of concrete's ability to resist chloride ion penetration.

The results of this test are related to the ability of concrete to permeate the chloride ions through itself as given below :

Coulombs	Chloride permeability
> 4000	High
2000 - 4000	Moderate
1000 - 2000	Low
100 - 1000	Very Low
< 100	Negligible

Needless to say, lower the chloride permeability of concrete, higher is the resistance of steel to undergo corrosion.

7.6.3. Electrical resistivity of concrete

The corrosion reaction can only proceed when the concrete is able to conduct electrical charges between the anodic and cathodic sites. As is well known, in a corrosion reaction, the positively charged ferrous ions from anode and the negatively charged hydroxide ions from cathode travel, through concrete, towards each other and forms ferrous hydroxide. Unless the reaction products from anodic and cathodic sites are removed, the corrosion reaction cannot proceed further. Hence, the concrete must be conductive enough to allow movement of electrical charges through it to promote corrosion reaction. Conversely, if the concrete has high resistivity (due to very dense concrete or under very low RH values), the corrosion reaction comes to a standstill. Note that the corrosion reaction can be determined by measuring the resistivity (opposite of conductivity) of concrete. The resistivity of concrete can be determined using a four-probe concrete resistivity apparatus (also called "Wenner" Probe/Array). The apparatus is placed on the surface of the concrete and gently pressed against it. The probes are so made that on pressing against the concrete surface, a small amount of wetting liquid is released from them to wet the concrete surface. This would ensure a good electrical contact between the probes and the concrete surface. A DC current is applied through the outer probes and the resulting potential is measured through the inner probes. Fig. 7.10 shows the block diagram of the principle and Fig. 7.12 shows a commercially available Concrete Resistivity Logger.

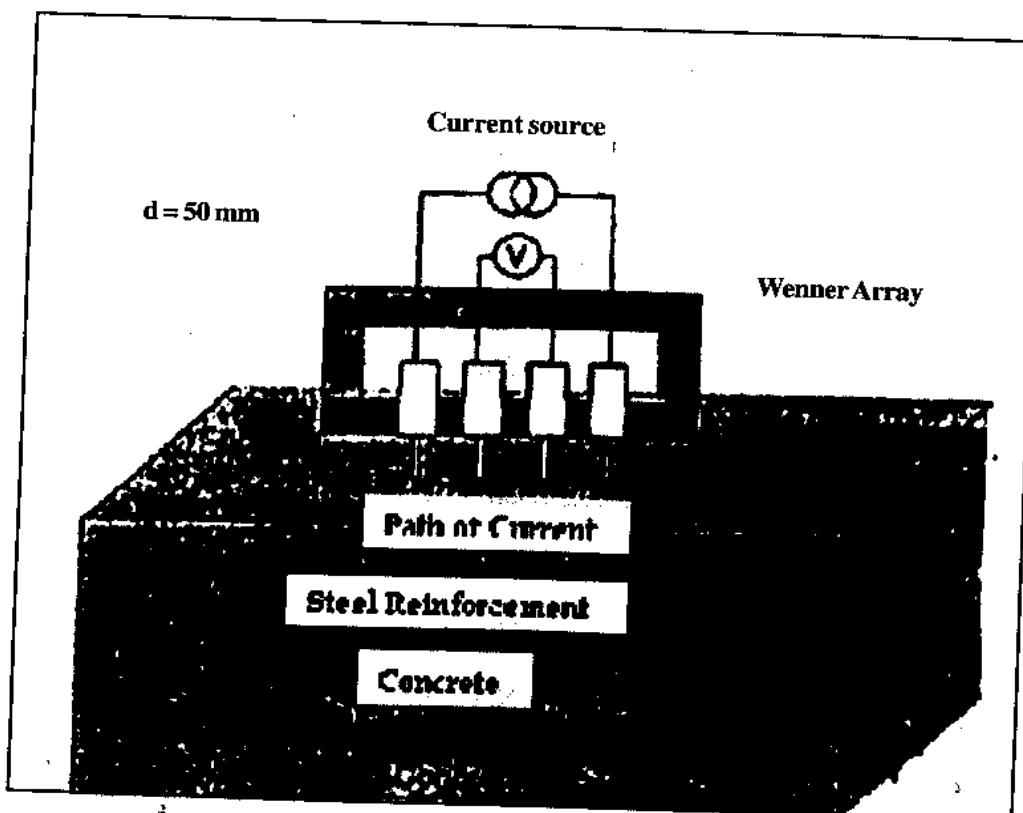


Fig. 7.12: Concrete Resistivity Testing

The resistivity is measured using the formula:

$$\rho = 2.R.d$$

Where,

ρ = Resistivity

R = Resistance. V/I

d = Distance between the probes, 50 mm.

The read-out unit directly displays the resistivity values. These are interpreted in terms of probability of reinforcement corrosion as follows:

Resistivity (ohm - cm)	Probability of Corrosion
< 5000	Very High
5000 to 10000	High
10000 to 20000	Low to Moderate
> 20000	Low

7.6.4. De-lamination of concrete

7.6.4.1. Chain drag/tapping

As the corrosion of steel progresses, the corrosion products accumulate between the steel and the concrete interface and thus cause the concrete to undergo de-bonding from the steel surface. The loss of bond between the steel and concrete is termed as de-lamination and a de-laminated concrete cannot transfer the loads to the steel. A de-laminated concrete can be easily identified as it produces a dull/hollow sound when gently tapped on its surface with a hammer or a metallic object. Moderately large concrete surfaces can be inspected for de-lamination by dragging a metallic chain along the surface. However, the limitation of this method is that it is not suitable for rapid inspection of large surfaces.

7.6.4.2. Ground penetrating radar (GPR)

Ground Penetrating Radar is a technique of recent times. In this technique, a short pulse radar is directed towards the surface of concrete. As the radar penetrates the concrete, a part of the pulse is reflected back when it encounters an interface between two dissimilar materials identified by their di-electric constants. Thus, in this method, the interfaces such as concrete-steel (un-delaminated), concrete-air-steel (delaminated) etc., can be identified. Commercial equipments based on this method are available. The results of the survey are produced on a graphical chart and it requires sufficient expertise to interpret the results. Some equipment are also available with image processing facility in which the image of the surface/interface is generated. GPR is also used for location of steel reinforcement in concrete structures where use of profometer/covermeter is time consuming.

Commercial equipments are available as both hand held type and vehicle driven type. Fig.7.13 shows a typical output of the testing of a concrete structure using GPR.

Details of tests which can be carried out for checking corrosion are summarized below in Table 7.5.

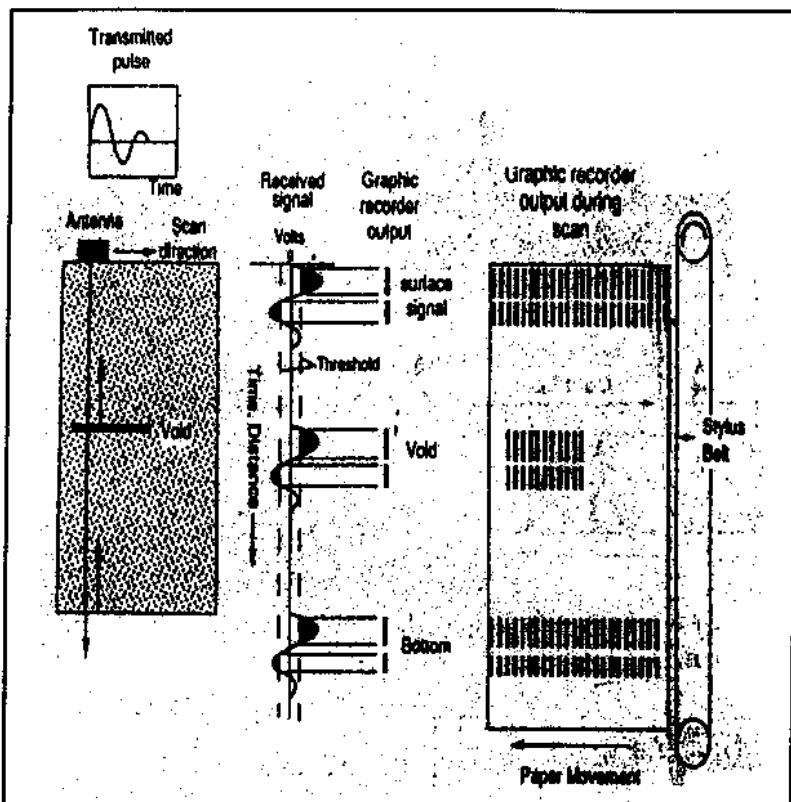


Fig. 7.13: Typical Output on a Graphic Recorder from GPR Testing

7.7. Summary of Tests for Corrosion

Table 7.5

S.No	Type of Test	Advantages	Limitations	Remarks
CORROSION OF REINFORCEMENT				
1.	Potential Mapping (Half-cell)	<ul style="list-style-type: none"> Non-Invasive. However, does need a connection embedded rebars. Measurements can be performed rapidly but overall Speed of Survey depends on grid selected Equipment generally cheap. 	<ul style="list-style-type: none"> Results are affected by the weather, coupling to electrolyte in concrete Condition of concrete surface. Results are of comparative nature. Does not provide a direct indication of corrosion rate 	<ul style="list-style-type: none"> Allows zones of varying corrosion risk to be identified. Repeat measurements help overcome environmental and localized effects. Shows long term trends and help in interpretation.

S.No	Type of Test	Advantages	Limitations	Remarks
2.	Resistivity Mapping	<ul style="list-style-type: none"> • Relatively quick to perform. • Provides indication of relative corrosion risk. 	<ul style="list-style-type: none"> • Results may be affected by weather and coupling to electrolyte in concrete etc. • Does not provide direct indication of corrosion rate. • Interpretation of results may be difficult. 	<p>1 Used to supplement information gained from half-cell mapping.</p> <ul style="list-style-type: none"> • Typically used to investigate zones of perceived highest corrosion risk

PRESTRESSING

3.	Visual Inspection, Exposure of Pre-stressing Tendons/ducts/Rebars by excavation or coring	Provides direct evidence of current status (visual condition and measurement of residual/original component, size, presence of grout etc.)	<ul style="list-style-type: none"> • Expensive. • Relatively slow and messy. • Surface gets damaged and needs repair subsequently. • Repaired zone may be site of possible future ingress/deterioration of embedded reinforcement. Needs good access for working. 	Care should be taken to avoid damage to important structural component such as - tendons and rebars.
4.	Radiography	<ul style="list-style-type: none"> • Non-evasive, produces an image of density photographic film or monitor. • Identifies rebars ducts pre-stressing elements and voids etc. • Can indicate size of bars etc. 	<ul style="list-style-type: none"> • Access required on opposite faces of the elements. • Health implication from radiation. • Expensive and slow. • Testing must be done by regulated personnel. • Maximum thickness of member to be tested is approximate 500 mm. 	<ul style="list-style-type: none"> • Specialist and expert personnel required to carry out test.

CONCRETE

5.	Acoustic Emission	<ul style="list-style-type: none"> • It is a technique that gives results directly from the process of flaw growth. • AE monitoring detects corrosion earlier than other methods (half-cell etc.) 	<ul style="list-style-type: none"> • Vehicular traffic cannot be permitted during the test. • Very expensive 	Trained and specialist staff required.
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S.No	Type of Test	Advantages	Limitations	Remarks
6.	Endoscopy/ Borescopy	<ul style="list-style-type: none"> • Viewing • Construction or • Defects greatly • Improves understanding of issues to be resolved. • Visual inspection may be the only practicable approach. 	<p>Visualization can be difficult with short range and fields of view provided by endoscope.</p> <p>This may lead to confusion. Multiple holes may be needed.</p>	<ul style="list-style-type: none"> • Requires access. • It involves drilling into elements causing surface damage which need to be repaired.
7.	Air Test to estimate Void Volume	<ul style="list-style-type: none"> • A simple method for estimating the volume of a void within a concrete element. • Can also give an indication of connectivity to atmosphere. 	<ul style="list-style-type: none"> • Involves detecting and drilling into void/duct with attendant risk of damage to pre-stressing tendons. Care required. 	Test needs only simple equipment.
8.	Carbonation Test (Phenolphthalein test)	<ul style="list-style-type: none"> • Test is easy and quick to perform on site, with clear indication in most instances. 	<ul style="list-style-type: none"> • Surface damage takes place and needs to be repaired. Tend to underestimate depth of carbonation. • Not suitable for Hac concretes. 	<ul style="list-style-type: none"> • Standard test for estimating depth of carbonation. • Accuracy about ± 5 mm.
9.	Site Chemical Test (HACH & QUANTAB) done on concrete drilling	<ul style="list-style-type: none"> • Tests are relatively quick, cheap and easy to perform by trained staff. • Enables the presence of significant chloride content 	<ul style="list-style-type: none"> • A proportion of on-site results should be verified by laboratory based testing. • Involves handling chemicals on site. 	On site testing allows sampling or testing regime to be developed and modified as investigation proceeds.
10.	Delamination Test (Sounding survey) Chain Drag	<ul style="list-style-type: none"> • Relatively quick and simple procedure which requires minimal equipment. 	<ul style="list-style-type: none"> • Can only be used on exposed surface. • Results dependent on expertise and trained ear of the operator. 	Provides an indication of delaminated areas and their extent.

8. REMEDIAL AND STRENGTHENING MEASURES

Remedial and Strengthening measures for bridge structures are already covered in IRC Special Publication 40 and Specifications for Road and Bridge Works issued by Ministry of Surface Transport under Section 2800. Reference should be made to the measures stipulated in these publications. However, some of the measures have to be added, modified and updated taking into account problems related to corrosion. These are given below:

8.1. Concrete Restoration

8.1.1. General

In all cases where concrete cover is repaired, the condition of the existing concrete in the exposed damaged area is of primary importance in the durability of the repair. The latter can be seriously compromised if there is poor adhesion between the fresh concrete of the repair and the existing sound concrete substrate. Therefore, it is important that the contact surface is in sound concrete and that all foreign materials are removed that might affect or impair the repair. In general, damaged and fractured concrete must be removed to a sound surface which must be properly treated.

Various methods may be employed for undertaking patch repairs, these include hand-applied mortar, recasting with concrete and possibly sprayed concrete on large structures/members. The approach requires the removal of damaged/carbonated/contaminated concrete in accordance with the criteria being employed and the nature of the deterioration involved. Depending upon the extent and severity of reinforcement corrosion, it might be necessary to introduce supplementary reinforcement to restore the structural strength of the elements concerned.

Concrete removal entails a range of methods which include percussive tools, pneumatic hammers, hydro-demolition, water blasting, milling machines etc. It is difficult to make comparisons of costs for the different approaches because of the commercially sensitive nature of the information. Hydro-demolition, milling machines and the like will be the most economic approaches as the areas involved increase in size. Each of the different techniques has its advantages and limitations. Hydro-jetting methods are probably the technique of choice on considerations of efficiency of concrete removal with minimal damage to the substrate, cleaning of reinforcing bars and removal to a given depth. However, there can be difficulties associated with water run-off, spray, flying debris, noise etc.

8.1.2. Hand-applied mortar

There are many proprietary patch repair materials on the market. These are pre-batched and generally achieve better performance and consistency than site-batched repair mixes. The proprietary materials are usually produced under a quality assurance scheme and come with certification defining their performance and characteristics. Some systems include bonding agents. These require close attention to ensure that the associated layer of repair material is applied at the correct time.

If the deterioration process has reached a level where a shallow surface repair is not feasible, a replacement of the missing concrete section should be considered. The technical choice of the repair material depends on volume to be replaced, the depth of the repair, the loading effects to be expected and the conditions of application on site.

The following materials for the replacement of a substantial depth loss of concrete surface should be considered: cementitious polymer modified mortar or concrete or resin systems.

(a) Polymer modified cement-bound system

This system is one in which a 5% by weight or more of plastic additive (by weight of cement) is mixed with conventional concrete or mortar mixtures. This additive improves the final properties of the concrete or grout and provides high early strength. This additive is introduced into the concrete or grout mix in the form of water dispersion, as an emulsion or as a powder. Furthermore, the compressive strength should not be significantly decreased (within 5%) nor should the modulus of elasticity be reduced. The curing of the plastic modified cement-bound systems is of equal significance as for cement repair systems. Generally, these systems are superior to the cement-bound systems as they have increased workability and exhibit fewer tendencies toward shrinkage cracking especially for thin layers.

(b) Cement mortar and concrete (possibly with additives)

The use of mortar or concrete for the repair of cover damage is dependent upon the thickness required. The maximum aggregate size should not exceed one-third of the required thickness.

(c) Pre-formulated repair materials (specific performance characteristics)

These are products which have been developed for specific repair conditions (surface roughness, minimum and/or maximum thickness of application etc.) and generally under specific environmental conditions (temperature range, degree of surface wetness etc.) They generally have specific instructions and procedures for application of the material to the damaged surface. These repair systems should only be used when their composition is known, they correspond to the requirements of existing guidelines and their successful use under similar circumstances has been demonstrated. Under these conditions an increased reliability is provided to the user and the client. The use of these repair systems does not absolve the user from the requirements of relevant standards and guidelines as well as surface pre-treatment and proper implementation.

(d) Resin systems

These are particularly beneficial when a short curing time, a high early strength and a high resistance to chemical and physical attack are required. They are suitable for repair of surface damage, of edge damage at joints and defects in the concrete. However, use of these systems should take into account that compared to cement-bound mortar systems; they have a larger thermal coefficient of expansion which can be upto 10 times higher than that of concrete. Resin systems are normally only used for limited thickness. Moreover, the resin systems do not provide an alkaline environment around the steel to maintain a passive layer on the rebars; they can be quite expensive. These facts make such systems less commonly used.

The various measures for damage repair, may in addition, require surface protection measures to provide for durability of the repair.

8.1.3. Repair of cracks

Cracks which remain after undertaking the erosion protection measures, need to be sealed. This can be done by pressure grouting with material such as epoxy resin, cement grout, polyurethane resin, acrylic resin, unsaturated polyester resin, etc. For details reference may be made to IRC : SP:40.

8.1.4. Sprayed concrete

Where the zones of deterioration are large, it may be appropriate to utilize sprayed mortar (gunite) or sprayed concrete (shotcrete) as the repair material. This can be applied by either the dry mix or the wet mix methods, with the latter reportedly becoming more popular since proprietary mixes have been developed. Care is required to avoid the presence of voids behind the reinforcement, particularly where there are multiple layers of reinforcement.

Sprayed concrete is suitable for the repair of cover damages, concrete replacement and for the strengthening of structural elements.

Pre-treatment of the surface is of prime importance when using sprayed concrete. Sand or water-blasting has proved to be an efficient surface treatment procedure: however, environmental protection regulations should be verified before use. No bonding agent is necessary because at the interface surface mortar enrichment occurs as a result of aggregate rebound.

Sprayed concrete is applied in layers of thickness 20 – 30 mm. Sprayed concrete in multiple layers requires that the preceding layer achieves a sufficient degree of hardness. Minimum reinforcement may be required for thickness larger than 50mm. This reinforcement should be fixed in position in such manner that it remains stiff and keeps its position during shotcreting operations. Addition of fibres (steel, glass, plastics) may also be used.

Curing may be accomplished by an evaporation protection, e.g. plastic sheet, to prevent rapid drying out. If a freeze-thaw/salt resistant concrete is required, a low water-cement ratio mix is used possibly with an additive added to increase tightness and compaction. Also surface protection measures may become necessary.

There are two basic sprayed concrete processes:

- a dry mix process where most of the mixing water is added to the nozzle and the cement sand mixture is carried by compressed air through the delivery hose to a special nozzle;
- a wet mix process where all of the ingredients including water are mixed before entering the delivery hose.

8.1.4. Recasting of concrete

Placing concrete in the area to be repaired should be accomplished in such a manner as not to impede concrete flow and to avoid entrapment of air. The re-cast concrete should have final properties that match the existing concrete as closely as possible (strength, modulus of elasticity).

In any structure where the repair process requires a significant proportion of the original concrete to be

removed, consideration needs to be given to the residual load capacity of the member(s) concerned. Due to the different elastic modulus, thermal, shrinkage and creep characteristics of the repair material, it cannot be assumed that this will carry load in the same way as the original material.

8.1.5. Encasement

Encasement is used more frequently in marine environments, being used fairly often to repair column and beam elements in concrete jetty structures and for some bridge columns, piers and abutments. The deteriorated/contaminated concrete will be removed prior to casting the encasement. This will no doubt contain reinforcement to control early age thermal and longer term moisture related shrinkage cracking.

Although the encasement may utilize traditional shuttered construction methods, it is also possible to use sprayed concrete.

8.2. Preserving or Restoring Passivity of the Reinforcement

8.2.1. Restoration of the protection

The decision on the necessity of removal of contaminated concrete cover, where commencement of the corrosion process is imminent, will depend upon the amount of contaminant content (chloride), the availability of moisture and the degree of carbonation.

If this is the case, the corrosion protection on the reinforcing bar has to be restored. This can be achieved by encapsulating the bar in an alkaline coating: concrete (sprayed concrete), cement mortar or epoxy resin mortar.

8.2.2. Corrosion Inhibitors

Corrosion inhibitors, which come in powder, gel and liquid form, retard the rate of the corrosion reaction. They are widely used in many industries to effectively reduce the corrosion rate of steel and other metals. Commercial products for the control of corrosion of steel reinforcement in atmospherically exposed concrete were first produced in the 1970's. They increase the time to the onset of corrosion and then act to reduce the rate of corrosion. They can be introduced into the concrete mix at the time of construction/repair or (in a suitable formulation) applied to the surface of an existing concrete structure.

There are three main types of inhibitors:

- Anodic inhibitors, which retard the corrosion reaction at the anode.

At low dosage there is concern that they will suppress generalized corrosion but may fail to eliminate all anodic sites.

- Cathodic inhibitors, which retard the reaction at the cathode and seek to prevent oxygen reaching the reinforcing steel.

At low dosages, they are effective at reducing corrosion rates but are generally less efficient than the anodic type.

- Multi-functional (ambiodic) inhibitors.

These combine the benefits of both anodic and cathodic inhibitors at relatively low dosages.

Inhibitors are consumed with time and will only work up to a given level of activation (i.e. chloride content). Calcium nitrate is one product, which has been widely used, being both added to the repair mix and applied to the surface of the concrete. Other proprietary products are available, however more is needed for enhancing alkalinity or blocking of pores.

To be effective a corrosion inhibitor applied to the surface of the concrete has to travel to the surface of the steel. This is by diffusion and in most instances, will be a slow process. The liquid inhibitor molecules are larger than chloride ions, hence will be transported at a slower rate. Vapour phase inhibitors offer an advantage that diffusion as a vapour will be faster than as a liquid. However, they may diffuse poorly through saturated concrete and may also diffuse out of the concrete. Drilling and grooving are sometimes used to speed the transport of the inhibitor to the steel surface. These processes are both damaging and expensive. The effectiveness of surface spraying/roller application will depend very much upon the quality of the concrete and its moisture content.

The principle of most inhibitors is to develop a very thin chemical layer on the surface of the reinforcement. There is a very wide range of corrosion protection performance from different inhibitor formulations, even with generic classifications. Independent evaluation and certification of performance is desirable from the specifier's point of view. However, such evaluations need to be representative of field concretes and conditions. As the true effect of an inhibitor can only be evaluated from corrosion rate measurements before and after application and by reference to a control area, such systematic evaluations are lengthy processes and are in their early stages.

8.2.3. Re-alkalization

This electro-chemical technique provides a means of restoring the alkalinity to carbonated but otherwise sound concrete. It involves the passage of a direct current between the reinforcement (the cathode) and an anode applied temporarily to the surface of the concrete. This process generates hydroxyl ions at the steel surface which locally regenerates the alkalinity of the concrete raising its pH upto about 12. This helps restore the passivating surface oxide layer to the reinforcement.

Under the applied voltage, alkali ions are drawn from the anode into the concrete. The use of sodium or potassium carbonate electrolyte is claimed to make the treatment more resistant to further carbonation. Several forms of anode may be employed. These are commonly either some form of mesh (titanium or steel) or electrolytic tanks (for vertical surfaces)/baths (for deck slab applications). Sprayed cellulose impregnated with the electrolyte is used with the mesh anode system.

The introduction of sodium ions, when using sodium carbonate as an electrolyte, may exacerbate any potential the concrete has for ASR. In these cases, plain water has been used as an electrolyte. It is understood that a lithium electrolyte has been proposed and tested but is still a subject of research.

The process typically takes between three and five days but sometimes may take several weeks. Successful treatment can be established by means of an acid/alkaline indicating solution. However, it should be noted that phenolphthalein changes colour at a pH of about 9.5 (unless a modified solution of phenolphthalein is used). This is not a passivating condition and an indicator (Universal indicator) with a colour change closer to pH 12 may be required to demonstrate that a passivating condition has been achieved.

As with cathodic protection and desalination, consideration must be given to hydrogen evolution at the reinforcement. The re-alkalization process applies some 20 – 50 V DC between the anode and the steel that must be expected to achieve steel potentials at which hydrogen evolution could take place. It seems unlikely that re-alkalization would need to be applied to pre-stressed concrete structures.

(not included)

Re-alkalization requires electrical continuity of the steel in the areas to be treated, a reasonable level and uniformity in the conductivity of the concrete, no short circuits between the cathode and the anode and no electrically insulating layers in the cover zone/bar surrounds. The process requires fewer concrete repairs than the patch repair alternative. It is also able to treat the whole surface of the zone in question. There has been strong growth in the use of re-alkalization in recent years (since the late 1980's) presumably because of its greater convenience and cost advantage over patch repairs.

8.2.4. Chloride removal

Negatively charged chloride ions (Cl^-) can be repelled from reinforcement and move towards an external anode by making the steel cathodic and passing a direct current through the concrete. This process is known by various names such as electro-chemical chloride extraction, desalination and chloride removal. It is similar in operation to cathodic protection by utilizes a temporary anode and a much higher electrical power density. The cathode reaction generates hydroxyl ions that locally enhance the alkalinity of the concrete in the vicinity of the reinforcing bars and encourages their re-passivation. Treatment periods are in the order of 3 to 6 weeks. Electrolytes employed include water and saturated calcium hydroxide.

The anode types employed are essentially the same as those used for re-alkalization protection, namely either mesh systems or liquid electrolyte systems contained within tanks. Sprayed cellulose impregnated with the electrolyte is used with the mesh anode system. These use either titanium or a steel mesh (which is consumed during the treatment).

As with other electro-chemical systems, it is necessary to have electrical continuity across the zone to be treated, no electrical short circuits between anode and cathode together with a reasonable level and uniformity in the conductivity of the concrete. The approach minimizes the amount of concrete repair work necessary.

It is claimed hat the technique can be used to treat the whole of the concrete surface and, on the basis of life cycle costs, that it should be applicable to a wide range of structures. Care needs to be exercised in relation to potential problems (as with the other electro-chemical methods, namely hydrogen evolution in the member concerned).

8.3. Cathodic Protection

The chloride extraction and re-alkalization repair techniques are temporary processes. Cathodic protection is a similar technique but permanent. It is now well established and is increasingly becoming accepted as a practical long-term solution for the rehabilitation of reinforced concrete structures suffering from chloride induced corrosion.

The basis of cathodic protection is to eliminate corrosion by reducing the potential of the steel to a more electronegative state, thereby con-verting the whole of the steel reinforcement into a large cathode. This is achieved by passing a small direct current between an external anode material and the steel reinforcement material. The anode material is connected to the positive pole of a rectifier and the negative to the steel reinforcement. The production of electrons (cause of corrosion) which are consumed by the oxygen and

water in reduction reactions does not occur at the steel reinforcement. Instead the system forces electrons into the steel to be consumed in these reactions and thus protect its integrity. The production of hydroxyl ions at the steel surface (cathodic reactions) causes the concrete to revert back to an alkaline state thus stopping the corrosion process.

Cathodic protection of reinforcing steel can be achieved by using sacrificial anodes or an external direct impressed current source. However, sacrificial anodes may not be suitable in the atmospheric zones due to the high electrical resistivity of the concrete.

8.3.1. Types of cathodic protection systems

- **Titanium mesh anode/cementitious overlay system**

This has been the system most widely used but other systems appear to have been overtaking it recently with regards to usage. It is applicable above ground/water level and provides even current distribution which minimizes the risk of over protection. If necessary multiple layers can be used to protect large surface areas of steel. However, the cementitious overlay can be susceptible to de-lamination, if not applied correctly. It also imposes extra weight on the structure and can be susceptible to impact damage.

- **Slotted/grid anode system**

A dense titanium mesh ribbon or strip is installed in slots cut into the concrete (generally 25 mm x 25 mm) and the slots are then backfilled with a cementitious mortar compatible with the parent concrete). It has a low risk of de-lamination and life can be enhanced by using a larger anode strip. Minimal concrete cover can affect the uniformity of current distribution.

- **Internal anode systems**

The internal anodes are embedded in 12mm diameter holes drilled into the concrete at depths of upto 300 to 400 mm. depending upon the length of anodes required and the structural component being protected. A graphite based backfill material or a conductive gel is injected into the holes and the 3 mm Platonized titanium rods are then inserted into the anode backfill. The life of the system is basically controlled by the consumption of the graphite backfill which is estimated by the manufacturer as 20 years, although recent use of the conductive gel suggests a life of at least 30 years. However experience is now showing that anodes of this type in service seem to have difficulties in meeting the lifetime expectations of the manufacturers. The system is especially cost effective on large elements such as beams, piers and columns but is not suitable for thin sections. Careful design is required to minimize cable requirements and to ensure optimum spacing and critical positioning of anodes in the vicinity of the steel.

- **Electro-conductive tape grease/over-wrap system**

A conductive tape/grease system which provides a conductive path to the concrete is wrapped around a column or pile followed by titanium mesh. A further layer of conductive tape over-wraps the mesh anode to secure and provide a contact surface for the outer face of the mesh anode. Mechanical protection is provided by either polyethylene or fiberglass impact resistance jackets. This system is difficult to install on large sections, is susceptible to impact loadings and appearance becomes a problem.

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- **Electro-conductive tape grease/panel system**

This system utilizes similar technology as the over-wrap system. However, it is pre-fabricated fiberglass impact resistant panel which can be bolted into position. It is suitable for soffits, columns and beams and it is easy to install. The thickness of panels and their aesthetic appearance are its negative attributes. The expected life of the conductive gel also needs consideration.

- **Water/soil anodes**

These remote anodes consist of proven materials such as high silicon cast it on, lead, silver or 3 – 6 mm diameter Platonized titanium rod embedded in coke breeze (conductive backfill) and secure din geo-textile bags in a trench. In shallow waters, the anodes are dug into the mud whilst those installed in sea water are normally located flush with piers and housed in a slotted PVC pipe for protection from boat damage. These anodes are powered by an independently controlled output from the Transformer/Rectifier and protect large areas of reinforcing steel in immersed concrete structures. These systems are mainly used in conjunction with other systems to address the problem of current dumping.

- **Sacrificial zinc anode systems**

Sacrificial zinc anodes can be clamped onto concrete columns, immersed below the waterline or dug into the mud in order to provide protection to the buried/submerged and tidal zones. The degree of polarization will vary as a result of several factors including the amount of current output from the anodes, the rate of polarization obtained in the submerged/buried section, tidal variations and tidal resistivity. Over-protection of the steel reinforcement is not a concern as zinc anodes' current output is self regulating with low driving voltage of 0.9 to 1.1 volts.

- **Sprayed zinc CP systems**

This is a very simple system with a very low initial cost outlay. It requires the application of 99.9% pure zinc by arc-spray method at a total thickness of about 400 microns. It is a sacrificial system, which has a life expectancy of 12 to 15 years that can be extended by whip blasting and re-spraying additional zinc material. It can also be installed as an impressed current system and a protective coating may be applied over it to further extend the useful life of the system. Sacrificial zinc anodes are also use din combination with these systems which protect the submerged/tidal zones and minimize current dumping.

- **Impressed current systems**

These require electrical connections to distribute the impressed current across the anode, a DC power supply and an associated control system together with embedded monitoring probes providing data by which adjustments can be made to the voltage and currents applied.

An installation will normally be divided into a number of zones, each with its own power supply. The design of the zones needs to take account of a number of factors such as the –

- variation in moisture and chloride contents (and hence the conductivity of the concrete) across the structure.
- continuity of reinforcement in different areas,

- presence of joints in the structure,
- requirements for different anode types
- variation in reinforcement provision.

Commissioning is a very important stage in achieving an effective and durable CP system. It provides the opportunity to perform a variety of tests and trials establishing the initial behaviour of the CP system, make adjustments to current and voltage supplies and to verify control criteria.

Once it has been established that an impressed current CP system is providing protection to all reinforcement, it is essential that the operation of the system be monitored and that it be properly maintained. Changes occur in the concrete over the first few months of operation (increase in resistivity due to removal of chloride ions and drying out of concrete). The objective of the monitoring is to ensure that all reinforcement remains effectively protected.

For concrete structures with old types of pre-stressing steel, the risk of hydrogen-embrittlement shall be analyzed when considered the implementation of impressed current CP systems.

8.4. Structural Strengthening

Strengthening of structural members can be achieved by replacing poor quality or defective material by better quality material, by attaching additional load-bearing material and by re-distribution of the loading actions through imposed deformation of the structural system. The new load-bearing material will usually be high quality concrete, reinforcing steel bars, thin steel plates and straps, carbon or glass-fibre tissue or plates, post-tensioning tendons or various combinations of these materials.

The main problem in strengthening is to achieve compatibility and continuity in the structural behaviour between the original material/structure and the new material/structure.

8.4.1. Strengthening of concrete sections

Strengthening of concrete sections may be achieved by:

- Increasing the concrete section itself
- Introducing new reinforcing bars in special recesses carved in the old concrete.
- Placing new reinforcing bars outside the old concrete and linked thanks to additional concrete.
- Creation of a composite structure (for instance: steel-concrete composite or advanced composite material (ACM) – concrete composite).

In each case, the new section has to be designed and checked according to the design specifications of composite structures.

Satisfactory interaction between existing concrete and new concrete is required: the two parts must act as a homogeneously cast structural component. The joint between old concrete and new strengthening materials must be capable of transferring shear stresses. The joint must be durable for the environment in question.

Differences in creep and shrinkage properties between old and new structural elements will require careful evaluation.

8.4.2. Strengthening with bonded materials

8.4.2.1. Steel plates

The strengthening of concrete structures by means of bonded steel plates is a technology existing in many countries:

(a) Short term behaviour

The load carrying capacity of this type of strengthening depends upon the strength of the reinforcement, the concrete and the adhesive. Yielding of the reinforcement will cause the adhesive to fail. Utilization of high-strength reinforcement is limited by the dimensions, concrete strength etc. Concrete strength has a large influence on the efficiency of the strengthening because the failure plane is located within the concrete.

The concrete dimensions, according to previous tests, do not appear to have any decisive effect. The surface condition of the steel is an important parameter. Suitable conditions can be achieved by sand-blasting. Oil and grease should be removed by means of an organic solvent. As cleaned surfaces corrode rapidly, a primer coating should be applied immediately.

(b) Long term behaviour

The question of long-term behaviour is of particular importance for these materials, the properties of which are highly time-dependent. Of considerable importance are: creep, ageing and fatigue strength.

- **Creep**

The creep of epoxy resin adhesives is considerably greater than that of concrete. In accordance with the current state-of-the-art, it can be assumed that the creep deformation abates relatively quickly. In thin adhesive layers (≤ 3 mm) the influence of creep is restricted by the cohesion of the adhesive.

With increasing width, there is a risk of defects in the adhesive. Therefore, the width of the reinforcing element should be limited to a maximum of 200 mm.

The thickness of the adhesive coat, within a range of 0.5 – 5 mm has no significant influence on ultimate load. With increasing thickness of adhesive, the slip between the reinforcing element and the concrete becomes greater.

- **Ageing**

Ageing is a change of properties resulting from mechanical, physical and chemical influences (e.g. air humidity, radiation, heat, weathering and water).

- **Fatigue strength**

Preliminary tests show that the fatigue strength is approximately 50 per cent of the short term strength.

- **Corrosion and fire protection should be addressed**

8.4.2.2. Advanced composite materials

The technique of repairing structures by bonding strengthening reinforcement to the element has recently been given a tremendous boost; the explanation lies in the ability to replace traditional steel plates by flexible composites using e-glass or carbon fibres as a base. These are known as Advanced Composite Materials (ACMs).

The advantages of ACMs are numerous:

- high strength and elastic moduli,
- no ability to corrode,
- low mass
- extreme flexibility in application to surfaces of any shape or nature.
- high resistance to fatigue and wear.

Thin layers of ACMs are applied using a thixotropic resin with an epoxy base. The real breakthrough in the scheme lies in the fact that the fixing is done cold and without pressure thanks to the development of glues and fibres specially designed for the application to Public works. In addition to the exceptional properties of the ACMs the technique offers simplicity of usage and handling.

Caution must be exercised in using ACMs, especially in the case of shear enhancement, where strain compatibility between the ACM and the base material must be achieved, with a maximum limiting strain of 0.004. In flexure, ACMs must have sufficient bonding area to ensure the tear strength of the base material (concrete) is not exceeded. It should also be recognized that by increasing the flexural capacity of a member, the shear demand is also increased and this must be allowed for in the design. Fire protection issues should be addressed.

8.5 . Strengthening with Additional Pre-Stressing

- **General**

In many cases, strengthening by means of external post-tensioning is a highly effective method. Both reinforced concrete and pre-stressed concrete structures can be strengthened by means of external post-tensioning with the use of stands / bars. The influence of post-tensioning on serviceability and ultimate limit states can be varied within wide limits by selecting different methods of introducing the tensioning force and using different alignment for the tendon. Both bonded and unbonded tendons can be used. In this regard, reference be made to IRC:SP-40.

- **Special design considerations**

Strengthening by means of post-tensioning can normally be designed in the same way as an ordinary pre-stressed member. When calculating pre-stress losses, however, it should be noted that the effects of creep and shrinkage may generally be less than in normal design due to the age of the old concrete. The stress in an unbonded tendon in the ultimate state will be only slightly larger than that after pre-stress losses.

- **Protection against corrosion and fire**

The post-tensioning tendons should be protected against corrosion and fire to the same extent as in a newly built structure. The requirements for concrete cover are the same as for ordinary pre-stressed concrete structures.

- **Anchorage and deviators**

As the post-tensioned tendons are not embedded in the structure in the conventional manner, special attention must be given as to how the force is introduced.

There are several methods available for the attachment of supplementary pre-stressing:

- Anchorage at girder ends (abutment)
- Additional supports either in concrete or steel fixed to the web of the box girder at existing diaphragms.
- Deviators – these devices can be either concrete or steel. They are attached to the existing webs or flanges by short pre-stressing bars.

9. MONITORING

9.1. Instrumentation

During inspection and maintenance of Bridges, several problems are encountered such as – condition of components, bearings, tendons, grout reinforcement, expansion joint, condition of concrete etc. As there is no theoretical approach to these problems, instrumentation is useful to assess them and develop solutions.

For monitoring the health and performance of the bridge and to assess strength and serviceability capacity of bridges in distress, instrumentation is used related to corrosion aspects. List of such instruments and Tests and that of supportive instruments and tests are given in Tables 9.1 and 9.2:

9.2. Instrumentation/Tests for Direct Testing

Table 9.1

S. No.	Instrument/Test	Application	Limitations/Remarks
1.	Magnetic Detector for reinforcement.	<ul style="list-style-type: none"> • To test cover thickness of concrete at site. • Range is 20 mm to 120 mm ($\pm 1\text{mm}$) 	<ul style="list-style-type: none"> • Portable and Handy instrument for Rebars. • Cannot check cover and layer of reinforcements of ducts / tendons
2.	Half Cell Potentiometer	<ul style="list-style-type: none"> • Detection of corrosion of steel embedded in concrete. • Can determine whether 	<ul style="list-style-type: none"> • Carried out in-situ on the structure. • Commonly used in investigations.

S. No.	Instrument/Test	Application	Limitations/Remarks
		corrosion is occurring and if so, extent of it.	
3.	Resistivity Meter (Wenner Probe)	This instrument is useful to determine ability of concrete to promote or resist the corrosion reaction. Values obtained from the logger indicate probability of rebar corrosion.	It is useful for assessing probability of reinforcement corrosion.
4.	Radiographic Instrument	To inspect physical condition of rebars or pre-stressing element	<ul style="list-style-type: none"> • Due to use of radiation source in this technique, not commonly adopted.
5.	Endoscope	To inspect physical condition of reinforcing steel, pre-stressed wire or post-tensioned cable	<ul style="list-style-type: none"> • Numerous holes are required to be made. • Lot of care required as during preparation of hole, wire may get damaged.
6.	Radar	It is a remote sensing electrical measurement technique used to determine damaged location and nature of damage to the pre-stressing cable. It can detect voids in the grout.	This technique is under experiment and being used in USA for concrete bridges.
7.	Phenolphthalein Test	For finding out depth of carbonation of concrete.	Simple Test and used regularly.
8.	Chloride content evaluation	To check chloride content of concrete and likely initiation of corrosion of steel	Generally, such tests are carried out in laboratory. The site kits are available.
9.	Water Permeability Test	To find out permeability of concrete	<ul style="list-style-type: none"> • Need trained and experienced personnel to conduct the test. • The meter has to be imported.

9.3. Supportive Instruments and Tests

Table : 9.2

S.No.	Instrument/Test	Application	Limitations/Remarks
1.	Vibrating wire strain gauges	Measurement of strain in concrete	<ul style="list-style-type: none"> • Commonly used. • Experienced personnel needed.
2.	Electrical strain gauges	Measurement of strain in concrete	<ul style="list-style-type: none"> • It is regularly used. • Need trained and experienced persons.
3.	Precision load	To measure deflections	<ul style="list-style-type: none"> • Used regularly.
4.	Ultrasonic Pulse Velocity	To find strength of in-situ concrete	<ul style="list-style-type: none"> • Used commonly. • Truly non-destructive and reliable method
5.	Capo Test	To find strength of in-situ concrete	<ul style="list-style-type: none"> • Partially destructive. • Not very common. • Only covered portion is tested.
6.	Core Test	To find strength of in-situ concrete	<ul style="list-style-type: none"> • Reliable method. • Care required to prepare end faces of cylinder core. • Partially destructive
7.	Rebound Hammer	To measure surface hardness of concrete which is related to strength of concrete	<ul style="list-style-type: none"> • It indicates concrete quality upto a depth of 50 mm. • Considerable variations in results occur. • It can give approximate estimation of strength

9.4. Planning of Scheme of Instrumentation

While planning the scheme for instrumentation, following points should be considered:

- (i) First of all it must be recognized that all measurements are relative to the first set of measurements which form the base. Thus, instrumentation cannot retrace the distress history viz. the causes which led to the distress, but it can only help us to monitor the condition of the bridge from the time the instruments were installed. Absolute measurements would be possible only in the case of transients, e.g. effects of live load, temperature etc.
- (ii) Another important consideration is that instrumentation is not an end in itself. Correct interpretation of data acquired through instrumentation calls for mathematical modeling involving an idealization of the structure and knowledge of the material characteristics (e.g. elasticity, homogeneity etc.)
- (iii) Instrumentation is an expensive undertaking, involving considerable expenditure on equipment

and fieldwork. Much of the expensive equipment (e.g. pressure cells, strain gauges etc.) is irretrievably lost within the body of the structure in the very nature of data acquisition in the field. The quantum and the type of instruments need therefore to be planned carefully, on the basis of the theoretically predicted structural behaviour.

All experimental work involves an element of error and instrumentation in the field even more so. Furthermore, some mortality of the instruments employed is unavoidable. These considerations at once emphasize the utmost need for adequate redundancy of measurement to obtain meaningful results in any instrumentation project. To eliminate human error in measurements, data logging should be made as automatic as possible.

- (v) A field instrumentation job comprises of diverse activities such as testing, installation, recording of sensitive instruments, each of which demands close attention to details. The human element is therefore of paramount importance in instrumentation. Meticulous planning, precise execution and a high degree of commitment to the effort, bordering on perfectionism, by the individuals involved in it are absolute pre-requisites to success in any instrumentation effort.
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(The official amendments to this document would be published by the IRC's in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual etc. from the Date specified therein)