SPECIAL REPORT **18**

State of the Art: Corrosion and Corrosion Protection of Prestressed Concrete Bridges in Marine Environment

IRC HIGHWAY RESEARCH BOARD
NEW DELHI
1996

SPECIAL REPORT

State of the Art: Corrosion and Corrosion Protection of Prestressed Concrete Bridges in Marine Environment

Copies can be had by sending advance payment to the Secretary, Indian Roads Congress Jamnagar House, Shahjahan Road, New Delhi - 110011
Price: Rs 240
(Plus packing & postage charges)

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1996

Reprinted : October, 2019

Edited and Published by Secretary, IRC Highway Research Board, New Delhi - 110011
Printed by I. G. Printers Pvt. Ltd., New Delhi - 110 020
300 Copies

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PREFACE

The corrosion phenomenon has always been of great concern to engineering profession especially in Prestressed Concrete Bridges in Marine Environment. Since the onset of the nationwide corrosion problems in bridges, a number of field and laboratory investigations on corrosion and corrosion protection have been carried out. Keeping this in view, the Ministry of Surface Transport sponsored a Research Scheme on "Supplementary Studies on Corrosion and Corrosion Protection of Prestressed Concrete Bridges in Marine Environment". The Central Electrochemical Research Institute, Karaikudi (Tamil Nadu) was entrusted with this Research Scheme in order to prepare a State-of-the-Art Report on Corrosion and Corrosion Protection of Prestressed Concrete Bridges in Marine Environment. Accordingly, CECRI collected information from various sources and submitted the State-of-the-Art Report to Ministry of Surface Transport. The Report received from Ministry was placed before the Highway Research Board in its meeting held at Panaji (Goa) on the 18th June, 1994 which approved the Report for printing subject to certain modifications. Thereafter, a Sub-Group consisting of the following members was constituted for finalising the Report:

1. Shri M.V. Sastry
2. Shri A.G. Borkar
3. Shri S.A. Reddi
4. Dr. D.N. Trikha
5. Dr. N.S. Rengaswamy

The Sub-Group finalised the Report during its meeting held on 16th March, 1995.

The State-of-the-Art Report is an exhaustive treatise on the subject and is an essence of so many references listed at the end of each aspect. It discusses in detail factors leading to corrosion and different protective aspects thereon.

It is hoped that the State-of-the-Art Report would serve as ready reference for the use of practising bridge engineers and researchers.

M.V. Sastry
(Chairman
IRC Highway Research Board)
Director General (Road Development),
Ministry of Surface Transport
(Roads Wing)

June, 1995
New Delhi

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(As on 16th March, 1995 when the Report was finalised)

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1. INTRODUCTION

Ministry of Surface Transport (Roads Wing), New Delhi vide their O.M. No. RW/NH-34015/1/87-DO II dated 12th October, 1989 had sanctioned a research scheme to the Central Electrochemical Research Institute (CECRI), Karaikudi on "Bridge Research Scheme B6 (Phase-II): Part-I - Supplementary Studies on Corrosion and Corrosion Protection of Prestressed Concrete Bridges in Marine Environment". Under this scheme, the Ministry wanted CECRI to prepare a State-of-the-Art Report on the subject which shall include:

(a) Classification of the available literature
(b) Instrumentation methods
(c) Corrosion monitoring techniques
(d) Interim suggestions for immediate application, if any
(e) Design of research programme

Accordingly, CECRI started collecting the available literature information from various sources. The methodology of approach is schematically shown in Fig. 1.1.

1.1. Literature Survey

Literature survey was made by referring to the abstracting journals, like, Chemical Abstracts, Corrosion Abstracts, International Abstracts of Werkstoffe and Korrosion, Current Titles in Electrochemistry. Global Computer Search was made through the Dialogue Network of National Aeronautical Laboratory, Bangalore.

Abstracting journals available upto the period June, 1990 have been referred to. Nearly 650 references are cited in this report.

1.2. Journals Surveyed for Full Details

The following journals were also scanned for detailed information on the subject:

* ACI Journal
* PCI Journal
* Corrosion
* Corrosion Science
* British Corrosion Journal
* Bulletin of Electrochemistry
* Transactions of SAEST
* Indian Concrete Journal
* Werkstoffe und Korrosion
* Materials Performance
* Concrete International
* Concrete (London)
* Indian Roads Congress Journal

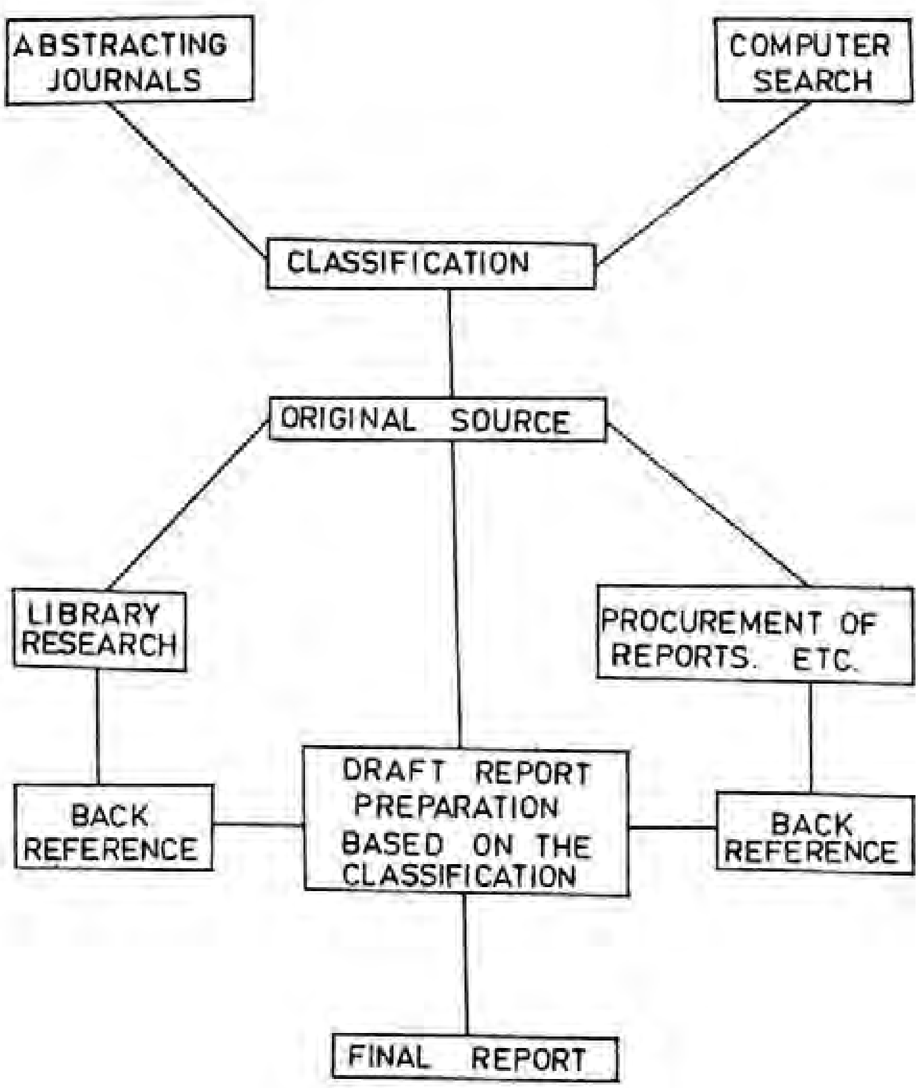

Fig. 1.1. Methodology of Approach

1.3. Reports

The following were contacted for their reports on the subject:

* FIP
* RILEM
* ACI
* FHWA
* NBS

* NTIS
* US Army Corps of Engineers

1.4. Classification

The available information was classified under five major headings, viz.:

(a) Case studies of corrosion failures
(b) Different forms of corrosion and factors influencing corrosion
(c) Monitoring aspects
(d) Protective aspects
(e) Repair and rehabilitation

Each major heading has been further sub-classified.

Under each heading, the work carried out by CECRI has been separately dealt with. At the end of each major aspect, the broad conclusions drawn from the available literature information has been included.

Case studies include failures of post-tensioned concrete and pretensioned concrete bridges, ground anchorages, storage tanks, pipelines, etc.

Stress corrosion cracking, pitting corrosion, atmospheric corrosion, stray current corrosion, etc. are covered under forms of corrosion.

Under factors influencing corrosion of prestressing steel, water cement ratio, effect of cracks, cover, curing practices, effect of chloride, etc. have been considered.

22 different techniques have been described under monitoring aspects. However, it is seen that no fool-proof non-destructive technique is available to monitor the corrosion of prestressing steel.

Protective aspects include coating to prestressing steel, anchorage protection, grouting practice, protection by grease, coating for concrete surface, admixtures, cathodic protection and alternative materials.

1.5. Summary and Conclusion

Important conclusions that can be drawn from the State-of-Art-Report have been summarised in a separate Chapter.

1.6. Scope for further Research

The existing gap in our knowledge has been identified and future R&D approach to the problem is highlighted.

2. CASE STUDIES OF CORROSION FAILURES

2.1. Post-Tensioned Concrete Bridges

2.1.1. Short-term failures (failure time - few days to few months)

Case-1: Brazil (1957)

Failure of 247 prestressing rods of a total of 252 occurred within 10 days of tensioning[1]. The stress applied was 150 ksi.

Reported cause of failure was the liberation of hydrogen sulphide from a mixture of sulphur, lamp black, kaolin and motor oil used during bridge construction by the reaction of sulphur with hydrocarbons in oil to from H_2S with SO_2 and SO_3 formed by oxidation.

Case-2: Netherlands (1964-65)

Post-tensioned wires in a concrete bridge failed within few days of 1 per cent. Na_2CO_3 being added to accelerate hardening of portland cement grout[2].

The failure was attributed to hydrogen embrittlement resulting from an electrolytic cell between the aluminium trumpet and mild steel duct which was active in the presence of sodium carbonate. Small amounts (0.05 per cent) of sulfides present in the grout served as poison.

Case-3: USA

Failures of prestressing wires even during the time of tensioning have been reported by California Division of Highways[3]. Though, complete report on failure analysis is not available, the following were reported to be the reasons for failure: (i) wires were tensioned but not grouted for several weeks and longer, (ii) exposed to climate that has considerable night and morning fog and were near refineries. Moist air and H_2S were believed to be the reasons for failure.

Case-4: England

Nine out of 240, 50 mm Macalloy high strength bars failed within days after stressing[4]. Though, galvanizing was reported to reduce the hydrogen embrittlement, cracking of galvanized layer during working and transporting lead to embrittlement.

Case-5: Austria (1952)

Heat treated wires in a prestressed concrete bridge failed several days after they had been stressed[5].

Case-6: Mississippi, USA (1984)

Post-tensioning system was installed on the Old River Control Auxilliary Structure. ASTM-A-722 bars were initially stressed to 75 per cent of UTS. After tensioning, the tendons were left ungrouted for a month, after which relaxation of load occurred. The tendons were jacked to achieve the original load. Following grouting, 4 of the 84 tendons failed on the same day[3].

Failure analysis showed that the reason for failure was environmental assisted cracking.

Case-7 (Mid 1960s)

Failure of post-tensioned rods which were used ungrouted as temporary tendons in a concrete floating bridge over sea water, occurred within a period of 2 months. Severe marine environment was found to be the reason for the failure[6].

Case-8

Loosening of prestressing tendon occurred in 2 months in the case of a floating bridge over seawater. The failure was attributed to SCC due to severe marine environment[7].

Case 9: California, USA (1960s)

A number of post tensioning wires failed during a highway bridge construction after a few months[8]. The stress applied was 160 ksi. The failure was ascribed to defects in wires raising stress intensity above threshold in cement bleed water containing sulphates and chlorides.

Case-10: USA (1961)

Failure of post-tensioned tendons occurred in 6 months after the opening of the bridge to traffic, due to severe pitting[9,10]. Massive collapse failure occurred involving 34 wires of a 40 wire bundle at the end of 4 years.

2.1.2. Medium term failures (1 year to 5 years)

Case-11: India (1982)

A 2 km long prestressed concrete bridge showed signs of corrosion from the second year of its service[11]. 10 years after construction, investigations showed severe corrosion of prestressed wires. It was found that the aggressive environment, chloride contamination of concrete and drop in alkalinity of gunite (used for repair) were responsible for corrosion.

Case-12:

Failure of about 34 prestressing wires took place in 4 years in the case of a floating bridge over sea water[7].

Case-13:

Failure of prestressing wires occurred in 5 years, requiring replacement of tendons. It was found that there was no bonding of structure to concrete pier face, giving water accessibility to tendon[7].

Case-14: Switzerland (1976)

A bridge collapsed due to failure of anchored abutment in 5 years of service. Inspite of cement grout cover in fixed length of the anchorage and polyethylene sheathed strands in free length with asphalt filling, severe corrosion occurred. Tendons were exposed to aggressive ground water containing sulphides and chlorides in the fill soil. Aggressive environment, poor construction and inadequate grouting were the reasons for the failure[12,13].

2.1.3. Long-term failures (beyond 5 years)

Case-15

Corrosion of prestressing steel in the case of a bridge in a semi-desert environment occurred in 6 years after construction[7]. Mixing of seawater in concrete was attributed to be the reason for corrosion.

Case-16

In the case of an overpass longitudinal cracking in bottom flange occurred in 7 years because of poor grouting[7]. Superficial rusting was noticed and cracking occurred because of freezing water.

Case-17

Prestressing tendons of 32 mm dia failed in 8 years in a bridge over seawater[7]. Spalling of concrete (cover 2") and corrosion of stirrups were noticed.

Case-18: USA (1977)

Post-tensioned girders of a viaduct showed signs of corrosion in 13-15 years of service[14]. Tendons failed due to corrosion and fracture of tendons occurred without reduction in cross-section. In spite of grouting and greasing, corrosion occurred in tendons and anchorage zones.

Case-19

Corrosion occurred in abutment spans of a bridge over seawater in 14 years. Spalling of concrete due to corrosion was observed[7].

Case-20

Corrosion of prestressing steel was observed in a bridge over seawater in 15 years[7]. The reasons for corrosion were found to be thinness of webs, insufficient cover and insufficient compaction of concrete.

Case-21: India (1986)

Failure of prestressing cables resulted in a major collapse of a concrete bridge after 16 years of its service[11]. Very high chloride and sulphate content in the environment, inadequate cover and chloride contamination of concrete led to snapping of prestressing wires. Cross-section reduction upto 48 per cent were noticed in some of the wires[14,15].

Case-22

Corrosion of steel conduits occurred in 20 years in the case of a bridge over a river, leading to cracking of webs[7]. The reported reasons for failure were stray currents and access of surface water through old type Freyssinet anchorages.

Case-23: India (1988)

A post-tensioned bridge after 21 years of its service showed extensive cracks due to corrosion[11]. Severe corrosion led to snapping of prestressing wires in some of the spans.

Loss in alkalinity and high chloride concentration were ascribed the reasons for corrosion.

Case-24: England

A post-tensioned segmental bridge, Y-nys-y-gwas, collapsed in 1985, after 30 years of its service[16]. Y-nys-y-gwas bridge was a single span bridge constructed using the Freyssinet system. Failure occurred at mid-span under dead load. Chlorides from de-icing salt were the primary cause of corrosion. Lack of an in-situ slab over beams, ineffective water proofing, ineffective protection of tendons, opening of segmental joints under live load and damp river environment resulted in chloride penetration and corrosion. Failure of galvanised prestressing tendons due to SCC and general corrosion has been reported.

Case-25: Denmark

Long range deterioration of prestressed concrete bridge over a rail-road yard was observed[17]. The external prestressing steel wires corroded because of the environment with high humidity and highly aggressive gases of the steam engines. Severe pitting was noticed.

Case-26: Germany

Corrosion of prestressing steel of over passes above rail-road tracks was noticed. The reported corrosion of prestressing steel was attributed to the chemical interaction of sulfides of high alumina cements with gases of the steam locomotives[12].

Case-27: Schaffgansen, Germany (1958)

Failure of a prestressed concrete bridge was reported as a SCC failure[4].

Case-28: Hood Canal Bridge, Washington, USA

Failure of some prestressed wires in a floating bridge over seawater occurred. The reason for the failure was attributed to SCC[18].

Case-29: France

Severe corrosion of prestressing cables resulted in demolition and rehabilitation of Vauban bridge. Grouting defects were found to be the reason for failure[19].

Case-30: Taiwan

Severe corrosion of prestressed steel wire of Peng-Hu bridge was reported in 1985[20]. Marine environment and high chloride content in concrete were ascribed the reasons for corrosion.

Case-31: India

In a 935m long bridge in Orissa severe corrosion of non-prestressing steel and prestressing cables including snapping of few wires in cables have been observed after 20 years of service[11].

Case-32: India

Incidences of corrosion in mild steel reinforcement as well as the prestressing cables including snapping of few wires of cables of a 556m long prestressed concrete bridge constructed in 1970 in Maharashtra has been reported in 1988[11].

2.2. Pre-Tensioned Concrete Bridges

Case 1: USA (1954)

The Route Seven viaduct, a typical span consisting of 17 precast prestressed box girders showed corrosion distress. Spalling of concrete led to exposure of strands to atmosphere and chloride laden water. Corrosion was found in all wires including the central wire[14,21].

It was found out that the chloride content was around 0.102 to 0.510 per cent which is more than sufficient to cause corrosion.

Case-2: USA

O'Hare Airport Leads Bridge having precast pretensioned girders showed corrosion distress in two major areas. Cracking and spalling of concrete led to exposure of strands to atmosphere[14,21].

Chloride concentration upto 0.742 per cent was detected and attributed to the cause of corrosion.

2.3. Conclusions

From the case studies reported, the following conclusions can be arrived at:

Short-term Failures (Failure time - Few days to few months)

There are 10 cases of failures of prestressed concrete bridge, which have occurred within few months.

Many of these failures were due to stress corrosion cracking and the rest due to general corrosion. The following were the reasons for SCC:

- Presence of hydrogen sulphide in the environment
- Presence of sodium carbonate when used as an accelerator
- Severe marine environment
- Delayed grouting
- Imperfect grouting

It must be pointed out that protective coatings when applied imperfectly lead to accelerated corrosion failure. Galvanized prestressed wires failed within two days due to cracking in galvanized layer.

Medium-term Failures (Beyond 1 year upto 5 years)

There were 4 medium term failures out of a total 32 cases. One of these cases was due to SCC and the others due to general corrosion.

Marine atmosphere or seawater was the main reason for severe uniform corrosion of prestressing wires in bridges.

It has been found that shotcreting of pier face induced seawater permeation by capillary action which resulted in severe corrosion.

Inadequate grouting and aggressive fill soil environment resulted in collapse of a bridge.

Long-term Failures (Beyond 5 years)

A majority of long term failures was due to pitting and general corrosion.

The following are the main reasons for such failures:

- Marine environment, seawater mixed concrete
- Stray current

Two of the long term cases were due to highly aggressive gases of steam locomotives and their interaction with concrete constituents.

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3. FORMS OF CORROSION AND FACTORS INFLUENCING CORROSION

The importance of the investigation of corrosion and corrosion protection in prestressed concrete structures is emphasised by the fact that corrosion damage in prestressing steel can be more dangerous than in conventional reinforcing bars. From the failure cases identified, it is found that prestressing steel can be damaged by one or more of the following forms of corrosion:

(a) Uniform corrosion
(b) Bimetallic or galvanic corrosion
(c) Pitting corrosion
(d) Stress corrosion cracking/hydrogen embrittlement
(e) Corrosion fatigue
(f) Stray current corrosion
(g) Microbial corrosion

Factors causing these forms of corrosion and preventive measures are discussed.

3.1. Uniform Corrosion

Steel undergoes corrosion damage when sufficient quantity of oxygen, water (or moisture) are present. Aggressive environments, like, marine environment promote severe corrosion. In the case of uniform or general corrosion there is no preferential attack of a particular area of the surface. Corrosion takes place throughout the surface.

Due to the heterogeniety of the surface of the material, micro-corrosion cells are formed. Moisture or water acts as the electrolyte. Oxidation or metal dissolution occurs at the anodic site. Oxygen reduction or hydrogen evolution takes place at the cathodic site.

Anodic reaction

$$Fe_{(solid)} \longrightarrow Fe^{2+} + 2e^-$$

Cathodic reaction

$$O_2 + 2H_2O + 4e^- \longrightarrow 4OH^-$$

(Neutral or alkaline media)

$$2H^+ + 2e^- \longrightarrow H_2$$

(Acid medium)

Prestressing steel exposed to aggressive environment during storage or while lying ungrouted undergo uniform corrosion. Concrete or cement grout provides an alkaline environment around the embedded prestressing steel in which the tendency to corrode is much less. A drop in alkalinity and chloride contamination results in corrosion of prestressing steel. Calcium chloride addition to concrete or cement grout provides severe general corrosion. Sulphates and carbon dioxide from industrial atmosphere also destroy the protective property of concrete.

Prevention of Uniform or General Corrosion

During storage and the period lying ungrouted use of vapour phase inhibitors, or coatings can be made. Organic coatings provide a barrier protection while cement based coatings provide a passive environment. Wires can also be temporarily protected by immersion in an inhibited solution.

A good quality concrete/grout can offer adequate protection. Coating to concrete can eliminate the deterioration of concrete and diffusion of aggressive species, like, chloride. Cement slurry based coatings provide sufficient protection by keeping the environment near the steel alkaline.

3.2. Bimetallic or Galvanic Corrosion

When two different metals are used in a corrosive environment or exposed to atmosphere and when they are in electrical contact, increased corrosion of one of the metals occur. This is known as bimetallic or galvanic corrosion.

Application of a different metallic material in the prestressing system, along with prestressing steel results in a galvanic couple. In one of the cases, failures of wires of a post-tensioned bridge occurred when an aluminium trumpet was used with mild steel duct. Apart from the accelerated dissolution of the active metal (anodic area), hydrogen embrittlement of the nobler material occurs due to hydrogen evolution (Refer Chapter 2).

Improper metallic coatings of prestressing steel results in a galvanic couple. In presence of an aggressive environment severe localised attack occurs. In the case of galvanizing, any defect in the coating leads to a small cathodic area (steel) and large anodic area (zinc). Since the anodic area is larger, overall general corrosion will be little. But, at the cathodic site hydrogen evolution will occur. Since the prestressing steel is under tension hydrogen embrittlement is a possibility.

Prevention of Galvanic Corrosion

(i) Use of dissimilar metals must be avoided.

(ii) In the event of unavoidable application of dissimilar metals, both of them may be applied with non-conducting coatings to avoid electrolytic contact. Electrical contacts of the bimetallic couple should be avoided.

(iii) Coatings applied should be perfect.

(iv) Increasing the electrolytic resistance will reduce galvanic corrosion. This can be done by using uncontaminated, good quality concrete.

(v) Among the coating systems, cement slurry based systems are preferred over organic barrier coatings, since the galvanic current flowing through an imperfection is minimum.

3.3. Pitting Corrosion

Pitting corrosion is one of the localised forms of corrosion. Whenever the passive film on a metal surface is removed partially in presence of aggressive anions localised corrosion takes place leading to pitting.

In the case of prestressing steel, the steel is surrounded by an alkaline environment of pH about 12.6. At this pH level, prestressing steel is passive. Pit initiation takes place when aggressive anions, like, chloride, sulphate are present above a threshold level.

Aggressive ions, like, halide ions, either from the atmosphere or from the contaminated cement grout/concrete depassivate the steel. Once pitting is initiated, two distinct regions are produced. The solution chemistry inside the pit becomes entirely different from that of the bulk solution. A decrease in pH occurs inside the pit, irrespective of the nature of the bulk solution. Due to the acidic nature and the presence of aggressive ions dissolution of metal takes place inside the pit. Thus, pitting is an auto catalytic process. Gravity effect makes the pit growth stable if the pit advances in the direction of gravity.

Pitting tendency of a material can be evaluated by electrochemical methods, like, cyclic anodic polarisation.

Pitting of prestressing steel is much more detrimental compared to other steel components, like, reinforcements. Since prestressing steel in service is always under tension, pits formed are narrow and deep unlike for mild steel where pits are usually shallow. Any small decrease in cross section of prestressing steel drastically increases stress level resulting in brittle failures.

Since some of the prestressed members are subjected to cyclic loading, pitting of prestressing steel may result in initiation of corrosion fatigue.

Prevention of Pitting Corrosion

To avoid pitting corrosion, following measures can be used:

(i) Presence of impurities on the steel surface should be avoided. A clean surface should be maintained.

(ii) Ingress of aggressive ions should be avoided or kept well within the tolerable levels. Hence, addition of calcium chloride to concrete should not be done.

(iii) A high pH level should be maintained. A good quality concreting/grouting keeps the environment around the steel highly alkaline and increases chloride tolerable limit.

(iv) Cathodic protection of prestressing steel completely eliminates pitting.

(v) Electrochemical removal of aggressive ions, like, chloride can be carried out to bring down the concentration of the aggressive ions to concentrations well below the tolerable limit.

(vi) Coatings can be used to prevent direct access of the aggressive species to the steel surface. Organic coatings offer barrier protection, whereas, cement slurry coating offers an inhibitive, alkaline environment and is thus preferred.

3.4. Stress Corrosion Cracking and Hydrogen Embrittlement

3.4.1. Introduction : Many instances of failures of prestressing steel due to brittle fracture have been identified (Chapter-2). Since prestressing wires are permanently under tension during service, depending on the environment, can undergo stress corrosion cracking (SCC) or hydrogen embrittlement. Because of its high strength and cold drawn surface, prestressing steel is highly defect sensitive.

Nitrates in soil, sodium carbonate added to concrete, high alumina cement containing sulphides and highly humid atmosphere have led to brittle fracture of prestressing steel in service.

3.4.2. Stress corrosion cracking and hydrogen embrittlement: Stress corrosion cracking (SCC) is the spontaneous cracking that may result from the conjoint action of tensile stress and corrosive media. In the absence of either stress or corrosion, the failure would not occur. The failure is a type of brittle fracture of normally ductile metals by the presence of specific environments. During SCC, the metal or alloy is virtually un-attacked over most of its surface, while fine cracks progress through it. It is important to differentiate clearly between SCC and stress accelerated corrosion (SAC) where structural corrosion is intensive even in the absence of stress and the effect of stress is to rupture the grain boundaries and to promote penetration of the environment.

Practically, all metallic structural systems contain alloys susceptible to SCC in some environmental conditions. Caustic embrittlement of steel boilers and seasonal cracking of brass cartridge cases are well known examples of SCC. Even pure metals may be subjected to SCC. Alloys subject to cracking are normally considered to be passive and non-corroding alloys. Environments in which cracking occurs are those in which corrosion is highly localised. Variations in composition, heat treatment, fabrication and mechanical processing affect the microstructure and consequently SCC susceptibility. SCC is well known in various aqueous media but it also occurs in certain liquid metals, fused salts and non-aqueous inorganic liquids. The presence of oxidisers often has a pronounced effect on cracking tendencies. SCC is accelerated by increasing temperature.

SCC in service results from tensile stresses at the surface or subsurface usually of considerable magnitude acting for prolonged periods of time. Stresses of this nature are usually residual, produced by methods of manufacture (quenching, cold forming, tube drawing without internal mandrel, etc.) or assembly (welding, press or shrink fits, wrapping of sheet to fit a structure, joining of poorly fitted parts, etc.). Stresses due to applied loads are seldom met in practice because of design considerations. Once a crack starts, it is possible for it to continue with no applied or residual stress but simply to be driven by pressures from corrosion products (of the order of 4000-7000 psi). SCC has never been observed to result from surface compressive stress (indeed, the introduction of surface compressive stress may be used as a preventive measure). As the magnitude of stress increases the time for total failure decreases although high stresses approaching the yield point are generally needed for SCC, frequently stresses that are small relative to the yield produce failure. For many alloy systems a 'threshold' stress (a stress below which SCC does not occur in some finite period of time) has been observed. SCC grows in a plane perpendicular to the operative tensile stress and may take either an intergranular or a transgranular path. The crack propagation is a discontinuous process.

Corrosion plays an important part in the initiation of cracks. A pit, trench or other discontinuity on the metal surface acts as stress raiser. Once a crack has started, the tip of advancing crack has a small radius and the stress concentration is great. Plastic deformation of an alloy can occur in the region immediately proceeding the crack till failure because of high stresses. The role of tensile stress has been shown to be important in rupturing protective films during both initiation and propagation of cracks.

Hydrogen Embrittlement

Hydrogen embrittlement can occur when atomic hydrogen diffuses into the core of the metal and decreases the cohesive strength of metal atoms. Presence of tensile stress in steel accelerates the diffusion rate. At a critical concentration of stress and atomic hydrogen, steel becomes too brittle and fails by decohesion. The damages of embrittlemnt increases at a higher external hydrogen activity.

For hydrogen embrittlement to occur there must be a source of atomic hydrogen, such as, steel itself, hydrogen sulphide, galvanic cells, etc. Atomic hydrogen may be present in the prestressing steel from the time of manufacturing. It may also be produced by gas dissolution from hydrogen sulfide present in the atmosphere in industrial and farming regions, in high alumina cement concrete, and in concrete made with blast furnace slag as follows:

$$H_2S_{(g)} \longrightarrow 2H_{(adsorbed)} + 1/2\ S_2\ (g)$$

In hardened high-alumina cement concrete H_2S is formed as a result of reaction between sulphur compounds and carbon-dioxide from the atmosphere.

A common internal source of atomic hydrogen is galvanic cell developed in a concrete structure. In a corrosion cell, at the cathodic sites hydrogen evolves as a result of water dissociation,

$$2H_2O + 2e^- \longrightarrow 2OH^- + H_2$$

Stress corrosion failure and hydrogen embrittlement failure can be distinguished from normal mechanical failure by the absence of necking at the snapped end.

3.4.3. Stress corrosion cracking of prestressing steels: Prestressing steel being a high strength material is vulnerable to stress corrosion cracking. Many environments have been identified which cause SCC of prestressing steel. The following is the list of environments in which susceptibility of prestressing steel have been identified.

1. Nitrate medium
2. Sulphide medium
3. Chloride medium
4. Ammonium thiocyanate
5. Phosphate medium
6. Sulphate medium
7. Carbonate medium

SCC behaviour of prestressing steel in the above media and factors influencing the failure have been discussed.

3.4.4. Nitrate medium: Structure and Composition: It was found that drawn wire with bainitic structure inferior in nitrate medium[1,2]. Addition of Si and Cr adversely affected the failure time. Addition of Al was found to be beneficial.

It was shown that only high strength bainitic wires, and fine and medium sorbitic wires failed without polarisation[3].

Cold drawn wires are considered to be more resistant to stress corrosion than heat-treated (oil-tempered) wires. However, in nitrate medium both hard-drawn and quenched and tempered wires failed at 21° and 93°C[4]. The mode of failure in these two cases were different. In the case of quenched and tempered wires crack propagated normal to the surface, and in the case of cold-drawn wire, the elongated grains caused the crack to turn and run almost parallel with the surface. This difference in mode of cracking is the reason for the superior performance of cold-drawn wires compared to heat-treated wires. This fact was supported by studies showing better stress corrosion resistance of cold-drawn wires in boiling calcium ammonium nitrate[5].

Comparative studies in oil quenched and tempered steel, and isothermally, transformed steel showed stress corrosion cracks form in areas where the structure contains fine spheroidal carbide (tempered bainite or martensite) and not in pearlitic or coarsely spheroidized structures[6].

Concentration

It was found that increase in concentration beyond 0.25m did not produce any significant change in failure time in the case of boiling ammonium nitrate[3].

C.U.R. Report 44 (1971) on W.F. tests using calcium ammonium nitrate at 102°C showed that majority of drawn wires failed in 40-72 hours while majority of drawn and stress-relieved wires failed in 72 hours[7].

It was revealed by free-loop tests that oil tempered high strength wires failed in NH_4NO_3. A decrease in concentration from 1.0 N to 0.01N increased the failure time from 1 to 1000 hours[8]. Cold-drawn wires did not fail in both media.

Effect of Potential

It was found that quenched and tempered reinforcement wire cracked in calcium ammonium nitrate at 80°C under polarised and unpolarised conditions[9]. At low cathodic current densities cracking time decreased. With further increase in cathodic current density failure time increased. At low anodic current densities failure time decreased and further increase did not produce any further reduction in failure time.

Similar results were obtained by other workers in the case of eutectoid steel wire[3]. Quenched and tempered wires failed under anodic potential and at rest potentials, in boiling 1m ammonium nitrate. Under cathodic polarisation the failure time was much greater.

One of the isothermally quenched wire failed even under cathodic polarisation.

Effect of Cold Work

Cold working was shown to decrease the susceptibility of eutectoid steel to stress corrosion cracking in 20 per cent sodium nitrate[2]. Patented wires with no cold working endured for only 3 hours, whereas, 87 per cent cold work increased the failure time to about 100 hours.

Effect of pH

Cracking of eutectoid steel has been reported in ammonium nitrate upto a pH = 12[3]. The actual threshold pH was found to depend on the type of cation (For NH^{+}_{4}, 12 and for Na^{+} 10) for a particular type of wire.

It was found that under anodically polarised conditions, susceptibility of cold drawn wires did not show any pH dependence in the range 4-9[4]. It was also shown that pH - 10 was the threshold pH level beyond which no failure occurred. On the contrary it was reported that cracking had no relationship with pH[10].

Cracking of oil quenched and tempered carbon steel has been reported in the pH range 2 to 8[11].

Temperature

Stress corrosion cracking of bridge cable wires at room temperature has been observed in the case of nitrates[4,6,12].

Stress corrosion cracking of hard-drawn, stress relieved and oil tempered bent specimens were observed when exposed to 8N ammonium nitrate at 27°C[6]. Failures occurred between 3½ and 15 months. In similar tests at 93°C, the specimens failed within 75 days.

Temperature was found to have considerable influence on susceptibility of cold drawn and stress relieved prestressing steel, in 1m ammonium nitrate. Failure time was found to increase rapidly below 60°C[4].

A particular type of eutectoid steel, (T275) showed accelerated failure both at room temperature and boiling conditions in 1m ammonium nitrate, under anodic and cathodic polarisation.

It was shown that by testing bridge-cable wires that cold-drawn replacement wires did not fail in 0.01N ammonium nitrate at room temperature upto 39½ months[1], whereas, a batch of 14 Portsmouth bridge wires failed in 3½ to 9½ months under the same conditions.

An isothermally quenched high strength steel wire failed both at boiling conditions and at room temperature under polarised conditions[3]. Failure time at room temperature was higher compared to that in boiling solution.

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3.4.5. Aqueous chloride solutions/water: It was shown that cold drawn and stress relieved high tensile steel did not undergo SCC in 3.5 per cent NaCl when subjected to 60, 70 and 80 per cent of UTS, for 801 hours[1,2]. There was no SCC susceptibility even at 93°C (200°F) for 340 hours. Coupling with Zn did not produce SCC failure. It was also shown that heat treatment, 843°C oil quenching 315°C/hr did not produce cracking tendency[3].

Cold drawn high tensile steel did not fail in concrete containing calcium chloride or in boiling sodium chloride (pH 10-11). Specimens were tested for 27 months in concrete and 400 hours in boiling chloride[4].

It was reported that bainitic heat treated high tensile steel in 1M NaCl, even under a cathodic polarisation of 1.2 V, did not undergo SCC even under 86 per cent of TYS. But when the pH was raised to 11.2 failure occurred. Increase in stress (to 99 per cent TYS) also produced SCC[5].

It was shown that cold drawn high strength steel did not undergo cracking in chloride environment[2,3].

In the case of cold drawn prestressing steel taken from various bridge sites, there was no stress corrosion crack in distilled water[6]. Similar observations were reported in the case of cold drawn prestressing steel in water, even when subjected to 15 mA/cm^2 cathodic charging at 95 per cent of UTS[7].

However, failures of notched specimen of cold drawn high strength steel have been reported[8].

Effect of Environmental Variables

Humidity

Studies carried out[9] showed that crack growth rate increased with relative humidity upto about 60 per cent beyond which no further increase in growth rate was observed[10].

Threshold intensity and crack growth rate of high strength steel in humidity greater than 60 per cent were found to be identical to that in water[11]. This was ascribed to water condensation at the crack tip, leading to crack propagation in liquid phase.

It was also shown that introduction of oxygen (0.6 per cent) arrested the crack growth in water vapour environments.

Potential

In service experience, it has been found that self polarisation of ± 200 mV may arise in high strength steel wires[5].

This can be more severe as much as ± 700mV, in less likely circumstances. Differences in potential are deliberately produced by metallic coatings of zinc, metal based paints, etc.

Precracked specimens of high strength prestressing wire showed significant reduction in the failure time by the action of anodic and cathodic polarisation by impressed current[12]. The medium consisted of distilled water containing 600 ppm chloride and 1300 ppm sulphate.

Effect of Concentration

Tests on a number of high strength steels in distilled water and aqueous solutions containing various concentrations of sodium chloride have shown no significant change in threshold K_{ITH}[13]. Concentration has little effect on crack growth kinetics in these steels[14]. For steels having yield strengths about 185 ksi, chloride concentration affects the growth rate. Changing the environment from distilled water to 3 per cent sodium chloride solution decreased the failure time by two order of magnitude[15-17]. It was also shown that concentrations greater than 20 per cent may lead to rapid corrosion and cracking blunting.

A variety of high strength steels were tested in 3 per cent NaCl and at sea coast. It was found that for precracked specimens, similar threshold stress intensity values were obtained in both cases[18]. When tested in mortar, it was shown that SCC susceptibility increased with increase in chloride concentration.

Effect of pH

In case of notched specimens, it was shown that high acidic conditions promote cracking and high alkaline conditions stop or restrict cracking[19]. Susceptibility remains same in the range pH = 3 to 10. Similar studies showed that variations in the pH range 1 to 9, had little effect on the threshold stress intensity[20]. Increasing pH to 13.6 completely stopped cracking.

Studies have shown that a 25 MnSi prestressing steel did not show SCC tendency in $Ca(OH)_2$ solution at pH = 12.5[21].

Precracked specimens of cold drawn prestressing tendon showed stress corrosion cracking susceptibility from low pH conditions upto pH = 12.8[22].

At very high pH conditions, two regimes of stress corrosion cracking, separated by an immune region was shown for cold drawn eutectoid steel at room temperature, without polarisation[23].

Effect of Additives (Promoters)

There is considerable evidence showing SCC in water and aqueous chloride environment can be attributed to hydrogen embrittlement.

A number of elements promote the entry of hydrogen into steel, referred to as 'poisons'. Sulphur, phosphorous, arsenic, selenium antimony and tellurium have been found to promote hydrogen entry[24].

Failure tendency of a high strength AISI4340 smooth specimens in aqueous NaCl solution increased significantly by prior immersion in solutions containing elemental sulfur or sulfur compounds, such as, H_2S, SO_2 and $NaHSO_4$. This was attributed to the formation of sulfided areas on the steel surface that increased hydrogen absorption[25].

Sodium arsenate additions between 0.2 per cent and 0.5 per cent caused increased SCC failure of a high strength steel wire in 1m sodium chloride solution under cathodic polarisation[?]. It was reported that a minimum of 0.05 per cent arsenic was required to cause poisoning during electrolytic charging in acid solutions[26].

It was found that soluble sulphide did not cause room temperature stress corrosion cracking in both $Ca(OH)_2$ solutions or cement paste suspensions[27]. There was no synergistic effect when added to alkaline chloride solutions.

Effect of Additives (Inhibitors)

Studies have shown that anodic inhibitors like potassium chromate and sodium benzoate reduced rate of stress corrosion cracking growth of high strength steel in aqueous environment[28].

In the case of prestressing steel in mortar, 1 per cent $NaNO_2$ increased the time to failure in the presence of chloride[20].

Effect of Strength Level: Smooth Specimens

SCC behaviour of a variety of steels exposed to marine atmosphere has been reviewed[29]. When tested at a stress level of 75 per cent yield strength, it was found that constructional alloy steels and ultra high strength steels upto 180 ksi yield strength are resistant to SCC in environment containing chloride. In the range 180-210 ksi the types of steels may be resistant to SCC, depending on the specific steel and heat treatment. At yield strength exceeding about 210 ksi these steels are generally susceptible to SCC.

It was found[30] that several different constructional steels, when exposed to marine atmosphere and sea-water, did not show SCC upto 2200 days.

Several Japanese constructional steels were exposed to outdoor environments, like, coastal, rural and highly polluted industrial. Four point bending method was employed and the steels were stressed to 90 per cent yield strength. No cracking occurred in any of these steels or weldments during 3 years exposure.

SCC failure of reinforcing rods have been reported when tested at 75 per cent of tensile strength in sand moistured with 3 per cent NaCl. The rebar was earlier heat treated by the interrupted quench method[31].

Notched Specimens

The stress corrosion resistance of about 100 heats of constructional steels were tested in distilled water with round specimens containing notch. For yield strengths upto 160 ksi the threshold stress was essentially equal to the notched tensile strength showing SCC resistance. Increasing the strength to higher levels was accompanied by a rapid decrease in threshold stress[13].

Effect of Microstructure and Composition

A number of studies have been done to improve the SCC resistance of high strength steel by changing the microstructure and chemical composition. The following have been found to have no appreciable effect on threshold stress intensity.

(a) changing the microstructure from martensitic to bainite[18,32]
(b) refining the grain size[33]
(c) increasing the alloy content with additions of Ni, Cr, Mo and Si[18,34]
(d) decreasing the sulphur and phosphorous content[18]

Results of one study indicate that the incubation time required to initiate SCC in precracked specimens increases with alloy.

Effect of Cold Work

Only limited data are available on the effect of cold work on the SCC behaviour in aqueous chloride media.

It was shown that plastic prestressing increased susceptibility to SCC in the case of HY130 steel in artificial sea-water[35]. HY180 steel also showed severe SCC susceptibility when subjected to plastic prestraining. But, 5 per cent plastic prestraining made HY180 steel immune to SCC in the same media.

It has been reported that prestraining can have beneficial effect in the presence of cracks or sharp flaws[36]. Residual compressive stresses develops at the crack tip, due to prestraining and counteract subsequently applied tensile stress.

Effect of Welding

A few studies have been done to understand the SCC behaviour of weld metal in chloride medium. Stress corrosion resistance of HY80 and HY130 weld metal was evaluated in 3 per cent NaCl solution using precracked specimens[37]. The threshold stress intensity of HY80 steel was 88 per cent under unpolarised condition and 75 per cent when applied 1.2V cathodic potential. HY 130 weld metal showed lower threshold stress intensity. For steels having yield strength greater than 180 ksi, the susceptibility depends on the alloying elements[38].

Smooth specimens of A 517 grade F weld metal did not fail when exposed to marine environment for 425 days[39]. Such results cannot be taken as conclusive evidences of immunity, since HY80 and HY130 precracked specimens failed in the presence of precrack.

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3.4.6. Hydrogen sulphide medium: Many incidences of stress corrosion cracking of prestressing steel by sulphide have been identified (Chapter-1). Most of the short-term failures were attributed to sulphide stress corrosion cracking. Much work has been done on the sulphide induced stress corrosion cracking of prestressing steel.

Specificity of Hydrogen and Sulphide Ions

It is generally accepted that failure of prestressing steel occurs as a result of hydrogen embrittlement cracking, atomic hydrogen being produced at the steel solution interface by the corrosion reaction

$$Fe + H_2S \text{ --------------- } FeS + 2(H)$$

By 'poisoning' the recombination of atomic hydrogen H_2S acts as a promoter for the entry of hydrogen into steel[1,2].

It was shown that cracking did not occur in 10 per cent sodium sulphide, whereas, rapid fracture of prestressing steel occurred in water saturated with hydrogen sulphide[3]. This indicates the specific requirement of hydrogen and sulphide ions for cracking.

Effect of Concentration

It has been shown by many workers that as the hydrogen sulphide concentration increased the failure time decreased.

When a set of high strength steels were tested, the threshold stress was found to depend on H_2S concentration[4]. For a high strength HT80 steel when H_2S concentration was increased from 20ppm to 300ppm, threshold stress decreased to 1/5th of the initial threshold stress.

Decrease in H_2S concentration was found to increase the failure time[5].

Effect of pH

Sulphide cracking susceptibility has been shown to depend significantly on pH.

Failure time was found to increase with increase in pH in the case of notched and unnotched specimens[1,6]. Similar observations have been reported in the case of high strength wires[7].

Failure did not occur beyond a pH = 9.0[8]. But, cold drawn wires were reported to crack at pH values as high as 12.0[3]. Similar observations have been reported in the case of cold drawn wires[9,10]. Soluble sulphides did not cause room temperature cracking in saturated $Ca(OH)_2$[11,12].

Effect of pH was explained on the basis of increased hydrogen absorption with increasing hydrogen ion concentration. A sharp increase in absorption of hydrogen was reported below a pH = 4.5[1].

Temperature

Cracking resistance has been reported to increase with increase in temperature[13-17].

Effect of Potential

Investigations on the influence of polarisation potential revealed that quenched and tempered high-strength wires failed both under anodic and cathodic polarisation[9]. Cathodic over-potential of 0.5V decreased failure time by two order of magnitude. Some of the heat treated wires did not fail at rest potential but failed under cathodic polarisation.

High strength steel wires were shown to undergo cracking under polarised and unpolarised conditions[18]. Cracking under cathodic polarisation was more severe compared to that under anodic polarisation.

Effects of Microstructure

Among martensitic, bainitic and cold-drawn microstructures of similar strength level, there were no differences in cracking resistance in saturated H_2S solution[9].

Quenched and tempered steel without untempered martensite was found to be more cracking resistant than the ones containing untempered martensite[19].

It has been reported that cold-drawn wires were more resistance to cracking compared to heat treated wires[20,21].

Some workers have reported that twinned martensite is more susceptible to hydrogen embrittlement that martensite that does not contain microtwins[22-24]. Increasing alloying content, especially carbon, decreases the tendency to form twinned structure.

Cracking resistance was shown to increase with decreasing grain size[25,26]. But in the case of as-quenched condition, cracking resistance was not influenced by grain size[18].

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3.4.7. Ammonium thiocyanate: In order to bring out a standardized testing procedure for hydrogen induced stress corrosion cracking of prestressing steel, a mixed Committee of RILEM-FIP-CEB decided to set-up a Working Group involving a large number of European laboratories, institutes and organisations. Based on this FIP has brought out reports on stress corrosion test for prestressing steels[1-2].

Testing in 20 per cent ammonium thiocyanate at 50°C has been used for screening different prestressing steels[5-9].

Hydrogen absorption tests have been carried out with 35 per cent ammonium thiocyanate solution, for evaluating prestressing steels[10]. It was shown that ductility of prestressing wires was lost when kept immersed in ammonium thiocyanate.

Influence of Temperature

Failure time is reduced drastically as the temperature increased[7]. Failure time is reduced by an order of 2.0 to 1.6 when the temperature rises from 45° to 50°C. Similar temperature effects have been reported in FIP Technical Report[1].

Effect of Concentration

For a quenched and tempered steels as well as for cold drawn wires, a decrease in ammonium thiocyanate concentration increases the failure time of the steel[2]. By lowering the concentration from 15 per cent to 1 per cent, the failure time increases 15 fold.

Effect of Chemical Composition of Steels

Effects of chemical composition on failure time of prestressing steel have not been examined fully.

Chromium content of the steel is found to strongly influence the failure time. Steel containing chromium above 0.45 per cent showed failure time between 3 and 20 hours, whereas, in the case of steels with contents lower than 0.45 per cent the life time was between 40 and 1000 hours[2]. The sensitivity of steels with high chromium content towards stress corrosion cracking in 20 per cent NH_4 SCN has been reported by other workers also[2].

Steels with chromiun together with silicon showed better stress corrosion resistance in 20 per cent NH_4 SCN tests[4].

Microstructure of Steels

Depending on the method of production, prestressing steels present very different structures; martensitic, bainitic, pearlitic, with or without cold deformation, etc. Influence of structure on the failure time of prestressing steel in NH_4SCN has been studied by many workers and reported[2].

Following is the sensitivity of prestressing steels in the increasing order. Cold drawn steels < quenched and tempered steels < hot rolled steels.

This statement contradicts the observations of some authors[2].

Cold drawn steels with/without subsequent stress relieving or stabilising were tested. A tendency to higher sensitivity can be seen in the following order[2]. Cold drawn steels < stress relieved steels < stabilized steels.

Surface Roughness

Though, there is no quantitative results available, it was reported that as the rough surface is larger, corrosion attack is more severe.

In the case of quenched and tempered steels, upto a pit depth of 40 μm the failure time in ammonium thiocyanate test, remains constant. When pit depths increased from 100 to 300 μm, the failure time falls to 20 to 30 per cent of the original value.

It was concluded that due to surface defects the life-time to time rupture can be reduced to 1/10th that of non-damaged steels.

Residual Stresses

Tensile stresses on the surface of steel decreases the failure time. After cold rolling, compressive stresses are produced on the surface. Such cold rolling operation increased the failure time by 10 times in NH_4SCN tests[3]. Residual stresses were quantitatively measured by Sprauel method[2]. A possible explanation was the superimposing of internal stresses and external test load, where the residual compressive stresses compensate to an extent and lower the effective tensile stress.

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3.4.8. Other media

3.4.8.1. Sulphate medium: High strength wire loaded to 95 per cent of yield strength (y.s. = 318 ksi) failed by SCC in sulphate/sulphuric acid system[1]. Wires failed in solutions containing 2 per cent ferric sulphate (pH ~ 2). Beyond 5 per cent, increase in concentration was insensitive.

Effect of polarisation was also studied[1]. Only high strength bainitically treated wires failed without any polarisation. Quenched and tempered wires failed under both anodic and cathodic polarisations. Wires with bainitic structures tempered to high strength levels failed under similar conditions. Cold drawn wire was found to be resistant to cracking under normal conditions. Very high applied stresses (~ 90 per cent yield strength) and high cathodic over-potentials were required for cracking.

Studies showed that cold drawn and heat treated wires were resistant to cracking in 8.0N ammonium sulphate solution[2]. Cold drawn wires exposed for 31 months in 0.02N ammonium sulphate did not fail[3].

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3.4.8.2. Carbonate media

Failure of high strength steel by stress corrosion was reported, in partly treated sewage containing mainly carbonates (pH = 8)[1]. However, cold drawn and oil tempered wires were found to be not susceptible to SCC in 8.0N ammonium carbonate[2].

Electrochemical polarisation tests carried out in sodium carbonate medium show that there is an active-passive transition in which stress corrosion cracking of prestressing steel could be expected[3].

Though, not much work has been carried out on the stress corrosion behaviour of prestressing steel in carbonate medium, service failures of prestressing tendons have been reported in the concrete containing carbonate admixture (Chapter 2).

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3.4.8.3. Phosphate medium: Though, stress corrosion cracking of mild steel and pure iron has been given by many reporters, much work has not been done on the stress corrosion susceptibility prestressing steel in phosphate medium.

Work done at CECRI on SCC of prestressing steel in phosphate has been discussed elsewhere in this Chapter.

3.4.9. Work carried out by CECRI

3.4.9.1. Nitrate medium

Potentiodynamic Polarisation Studies

Potentiodynamic polarisation behaviour of prestressing steel was studied in 1M ammonium nitrate under stressed and unstressed conditions. At room temperature both stressed and unstressed specimens showed identical active-passive behaviour. In boiling ammonium nitrate medium, stressed specimens showed unstable passive behaviour with larger critical current density and passivation current density. This indicated the existence of a susceptible potential zone in which stress corrosion cracking could be observed.

Effect of Potential on Failure Time

Cathodic Potential

It was observed that in ammonium nitrate medium, upto a cathodic over potential of 600mV, failure did not occur. It was established by current decay studies that there was no current decay under cathodically polarised conditions.

Anodic Potential

Under constant anodic potential, it was shown that prestressing steel failed within a range of +200 to +1200mV. Shortest failure time was recorded in the range +700 to +1200.

Current-time studies showed the presence of a current decay whenever failure occurred.

Effect of pH

Studies carried out at constant anodic overpotential of +800 mV in boiling nitrate medium at different pH levels showed three different zones of pH dependence. In the pH range 4.0 to 6 failure time increased. Failure time remained almost independent the pH range 6.0-9.0. At pH - 10.0, no failure occurred indicating a limiting pH of 10 beyond which cracking did not occur[1,2].

Effect of Temperature

Temperature is one of the most important factors affecting the failure time. Specimens were stressed to 90 per cent proof stress in 1m ammonium nitrate and the influence of temperature was studied at +800 anodic overpotential. As the temperature decreased, failure time increased. By decreasing the temperature from 106°C to 60°C, failure time increased from 25 mins. to 64 mins. Below 60°C the increase in failure time was rapid. Fracture did not occur below 40°C.

Threshold Stress

Threshold stress of cold-drawn prestressing steel in boiling 1M ammonium nitrate was found to be 60 per cent of proof stress which means that stress plays an important role.

Mechanism of Cracking of Prestressing Steel in Nitrate Medium

Passivation of steel surface takes place in the ammonium nitrate medium which is an oxidiser. Depending on the potential, the oxidising power varies. An active-passive behaviour is exhibited. Uniform metal dissolution occurs in the purely active zone and no stress corrosion cracking possibility exists. In the purely passive state the passive film is very adherent and tenacious that film rupture necessary for crack initiation becomes very difficult. Material becomes SCC susceptibile only in the active-passive transition zone.

Crack initiation takes place by the rupture of the film and crack propagation by stress concentration at the tip of the crack. Crack propagation depends on the anodic dissolution rate which is thermally activated process. Anodic dissolution is independent of pH at low pH levels. At very high pH, both initiation and propagation processes are difficult to occur.

Film formation is by anodic passivation. Cathodic polarisation does not result in film formation. Since nitrate is not a poison, like, sulphide, recombination of atomic hydrogen produced during cathodic polarisation takes place resulting in hydrogen gas evolution. Due to these facts, hydrogen embrittlement of prestressing steel is ruled out in nitrate medium.

The model depicting the different stages in the stress corrosion cracking process is shown in Fig. 3.1.

3.4.9.2. Hydrogen sulphide medium: Much work has been done in studying the stress corrosion cracking behaviour of prestressing steel in hydrogen sulphide medium.

Effect of pH

In 3.5 per cent NaCl solution saturated with H_2S, prestressing steel failed upto a pH of 9.2 beyond which no cracking was observed[3]. Failure time increased exponentially as the pH was increased from 2.6.

In the case of distilled water saturated with hydrogen sulphide the limiting pH was found to be around 7.0.

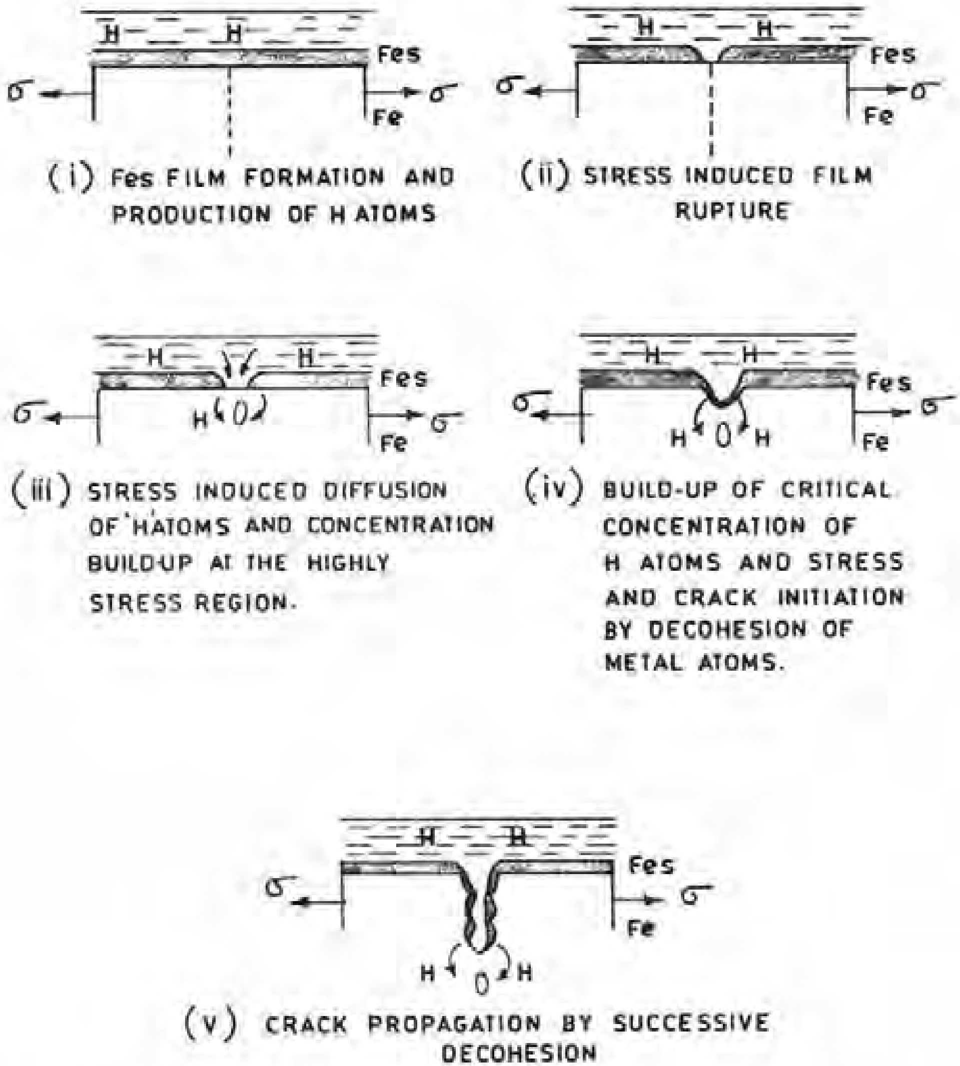

Fig. 3.1. Model Depicting the various stages in the Stress Corrosion Cracking of Prestressing Steel in Hydrogen Sulphide Medium

Increase in pH was found to increase the threshold stress. Threshold stress at pH - 2.6 was only 15 per cent proof stress, whereas, at pH - 6.5, threshold stress was bout 40 per cent proof stress[3].

Effect of Temperature

Effect of temperature was studied at pH - 4 in distilled water saturated with H_2S[3]. Increase in temperature was found to increase the failure time.

Effect of Potential

Cathodic polarisation was found to decrease the failure time drastically[1,3]. Beyond a cathodic overpotential of 130mV, failure time remains almost independent of the potential.

Anodic polarisation was found to increase the failure time.

Mechanism of Cracking of Prestressing Steel in Hydrogen Sulphide Medium

In hydrogen sulphide medium, the metal surface is covered with a film of FeS with simultaneous evolution of hydrogen. Atomic hydrogen, thus, produced can recombine to form molecular hydrogen (gas). But the sulphide ion acts as a poison for the recombination process. Atomic hydrogen gets absorbed on the metal surface and diffuses into the material. FeS film formed is brittle in nature and it ruptures at the weakest point practically along the grain boundaries.

Diffusion of atomic hydrogen depends on the concentration gradient existing betweeen the surface and the core of the material, and the applied stress. The stress induced diffusion of hydrogen results in accumulation of atomic hydrogen inside the steel in highly stressed regions. At sufficient concentration which is time and stress dependent, the hydrogen atoms bring about decohesion of the metal lattice and crack is initiated. Crack propagation occurs by successive decohesion aided by stress concentration effects and continuous build up of hydrogen atoms ahead of the crack. Brittle fracture of the material occurs. Mode of failure is predominantly intergranular.

Different stages in the cracking process have been depicted in the model shown in Fig. 3.2.

3.4.9.3. Thiocyanate medium: Stress corrosion studies have been carried out with prestressing steel in 20 per cent ammonium thiocyanate medium.

Effect of Polarisation

It was found that the failure time of prestressing steel is reduced significantly by a cathodic overpotential. An exponential decrease in failure time with increase in constant cathodic overpotential[4].

When tested at an anodic constant over potential of 500 mV, prestressing steel did not fail.

Threshold Stress

Threshold stress of cold drawn and stress relieved prestressing steel was determined under cathodically polarised condition (500 mV cathodic overpotential). Threshold stress was found to be 20 per cent of proof stress which is much below the practical design stress encountered in service.

Screening of Additives

To evaluate the performance of additives, tests were carried out with 20 per cent ammonium thiocyanate at 500 mV cathodic overpotential. Chemicals, like, thiourea, phenyl thiourea, hexamine and polyvinyl alcohal did not show any effect on the hydrogen embrittlement of prestressing steel.

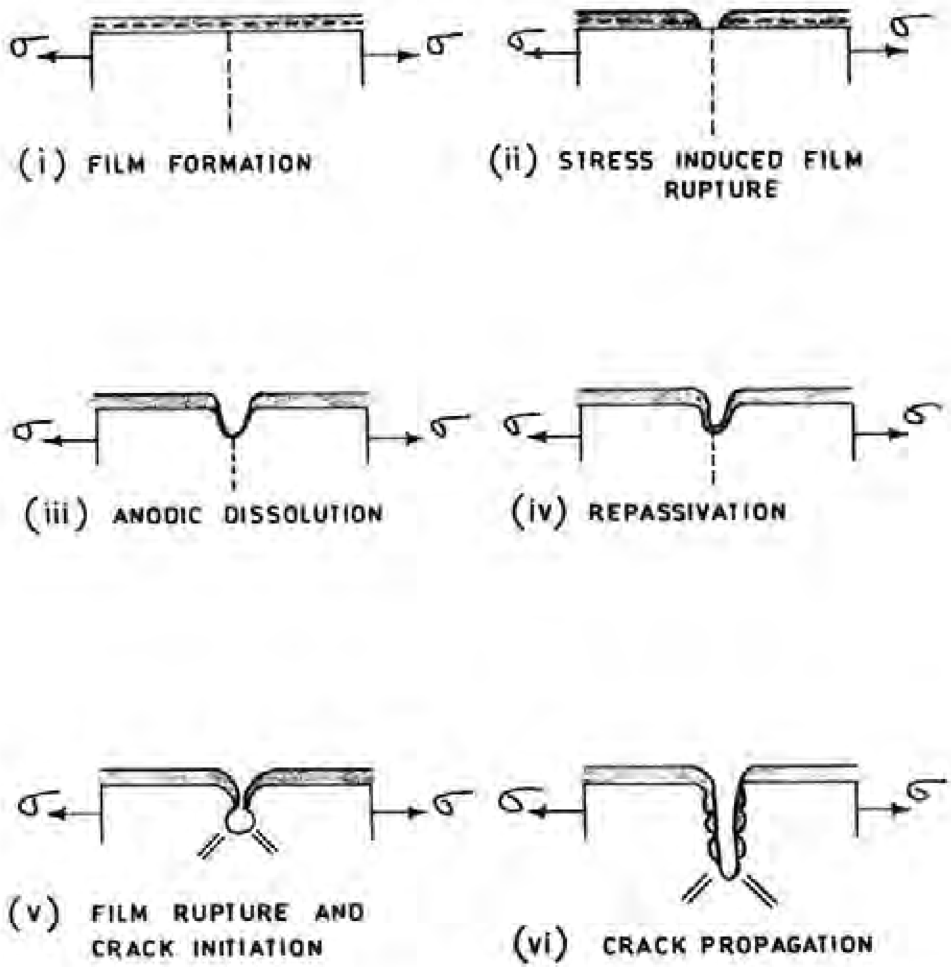

Fig. 3.2. Model Depicting the various stages in the Stress Corrosion Cracking of Prestressing Steel in Ammonium Nitrate Medium

Evaluation of the Effect of Shotpeening

Testing in 20 per cent ammonium thiocyanate under a cathodic overpotential of 500 mV, was carried out with controlled shotpeened specimens to study the effect of residual stresses induced by shotpeening. It was found that the failure time increased by 5 to 8 times by shot-peening[5].

Threshold was also found to increase by 50 per cent by shotpeening the prestressing steel.

3.4.9.4. Phosphate: It was shown that among various anions phosphate ion give rise to higher hydrogen content which can result in embrittlement[6].

Studies with prestressing steel in 1N Na_2HPO_4 have shown that failure occurred under polarised and unpolarised conditions[7]. Time-to-failure was shorter under unpolarised conditions than under anodic or cathodically polarised conditions.

Threshold stress was as low as 10 per cent of proof stress of the material. Failure was ascribed to hydrogen embrittlement.

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3.5. Corrosion Fatigue

Failure by corrosion fatigue of prestressed members in service has been identified (Chapter 2). Corrosion fatigue is the fracture of metals under repeated cyclic stress in corrosive environments. A reduction in fatigue resistance of material is encountered in corrosive environments.

Corrosion pits act as stress raisers and initiate cracking. Crack propagates until the cross sectional area of the metal is reduced to the point where ultimate strength is exceeded and rapid brittle fracture occurs. Fractured surface consists of a large area covered with corrosion products and a smaller roughened area due to brittle failure. Corrosion fatigue failure is transgranular and does not show any branching of cracks.

Fatigue Limit or Endurance Limit

For steels and other ferrous materials, the fatigue life becomes independent of stress at low stress levels. If a metal is stressed below this level it will endure an infinite number of cycles without fracture. This is known as fatigue or endurance limit of the material in that environment.

Corrosion fatigue is largely influenced by oxygen content, temperature, composition and pH of the environment.

Much work has not been done on corrosion fatigue behaviour of prestressing steel. Only limited data have been reported.

Cathodic protection using zinc coupling was shown to be an effective method to avoid fatigue crack initiation of prestressing steels in sea-water[1,2]. Further research has revealed that when flaws are present on the surface of the wire, ingress of hydrogen may promote a crack growth even faster than the crack growth at the unprotected conditions. It has been shown that cathodic protection in the optimum potential range of -700 to -900 mV (SCE) showed beneficial effects[3].

Prevention of Corrosion Fatigue

There are many ways to prevent corrosion fatigue. Some of the general preventive measures are given below:

(i) A more corrosion resistant material may be used

(ii) Compressive stresses can be induced on the steel surface which can reduce corrosion fatigue. Nitriding or carburising or quenching below transformation temperature can produce surface compressive stress. Surface rolling or shotpeening will also induce surface compressive stresses.

(iii) Coatings can be used to prevent initiation of corrosion pit which can propagate. But coatings are beneficial only when the stressing is of the reversed type with zero mean.

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3.6. Stray Current Corrosion

3.6.1. Introduction: Stray currents are produced in the ground when there is an electrical leakage or when a permanent electrical earthing is not provided. When direct current is used for traction purposes or welding, the circuit is deliberately earthed. The return current which ought to pass through circuitry elements, like, rails in the case of traction systems, often finds its way, atleast partly, through the surrounding electrolytic environment, like, soil, water, etc. Thus, an unintended path is made 'live'.

Any conductor which is present in the electrolytic environment sustains damage when the stray current passes through it. When the current enters the conductor, it causes cathodic effects. Anodic effects are produced at the point where the current leaves the conductor. In a single conducting material both anodic and cathodic effects may be present. But when the conductor is electrically connected to the corresponding electrode, only one of these effects may be present.

3.6.2. Stray current damage to prestressing steel: Stray currents can affect prestressed concrete structures, in the following situations:

(i) If the structure is associated with or in close proximity to an electrified railway system

(ii) if welding operations involving deliberate earthing are carried out in the vicinity
(iii) if the structure is in close proximity of a cathodic protection system
(iv) due to static electrical discharges of lightening, incidences

Incidences of failures of prestressed concrete structures due to stray current effects have been discussed in Chapter 2.

Stray current effects on prestressing steel is more severe compared to reinforcing steel. Prestressing steel by its smaller cross section is proportionally more severely affected. Since the prestressing steel is always under tension, the anodic and cathodic effects are highly magnified.

There is a difference between post-tensioned and pre-tensioned structures. In post-tensioned structures the full length of the wire is under tension, whereas, in pretensioned structures there is a reduction in tension at the ends of the prestressing wire. This makes the post-tensioned structure more sensitive to damage than a pretensioned structure.

Damage by stray current differs from that of atmospheric corrosion. Atmospheric corrosion in proportional to the surface area. When a thick bar is replaced by a number of thin wires, the surface area increases, thus, increasing atmospheric corrosion. In the case of stray current damage, as the surface area increases the current density decreases, decreasing the severity of the anodic and cathodic effects.

Anodic Effects

Anodic effects occur when the stray current discharges from the metallic conductor, namely, prestressing steel. This leads to an increase in potential. This causes corrosion of steel, known as stray current corrosion. As the stray current discharge is highly concentrated in one area, the corrosion damage is also localised. Both large anodic current and current densities are produced. Anodic stray current products long term deterioration of structure. Prestressing steel is more vulnerable than reinforcing steel, in that even a small reduction in cross section makes the working stress greater than the permissible stress level.

Cathodic Effects

At the point of entry of the stray current cathodic effects occur, leading to decrease in the potential (becoming more negative). This reduction in potential leads to evolution of hydrogen and/ or reduction of oxygen. Atomic hydrogen produced in the cathodic reaction may penetrate into the steel. Prestressing steel, being under high tensile stress is susceptible to hydrogen embrittlement under such cathodic effects.

3.6.3. Prevention of stray current corrosion: The extent of damage due to stray current can be greatly reduced by adopting various preventive measures, described here:

(1) Stray current can be avoided by using a proper return cable in the circuitry. The return cable of very low resistance should be used and directly connected to the power supply. The return cable should be well insulated so as to reduce any leakage.

As alternating current effect has not been found to produce stray current, usage of alternating current in place of direct current is recommended.

Deliberate earthing for return current should be avoided.

(2) Stray current effect is considerably reduced by insulating the structure. By providing bitumen, rubber bearings, etc. a high resistance layer can be introduced between leaking terminals and the structure. Such insulating type of coatings can be applied on the concrete surface so as to prevent the flow of electrical current through the system.

Elimination of stray current field can be done by providing an additional construction between return cable and the structure, which should be connected to the sypplying source. As per the specifications, reinforcing nets in tunnels for rail transport with direct connection to the substation can be used as a preventive measure.

(3) (a) **By making the nearby environment less aggressive or protective**

Use of good quality concrete, maintenance of high alkalinity and absence of chloride make the damage less severe, both under cathodic and anodic sites.

(b) By increasing the area of steel the anodic/cathodic current densities can be reduced, thus, making the attack less severe. In place of a thick bar, a number of thin wires can be used. In such cases, all the wires should be electrically connected. Metallic materials of less structural importance can also be connected electrically to the prestressing wire, to increase the surface area.

(c) By using a steel which is less sensitive to hydrogen embrittlement, cathodic stray current effects can be reduced. Application of residual compressive stresses can also reduce the sensitivity to hydrogen embrittlement.

(d) The structure can be electrically connected by properly insulated conductor to the main current line, where the electrical potential is higher. By this, discharging stray current from the electrolyte to the electrolytic environment is avoided. Thus, cathodic effects can be completely eliminated. In the case of prestressed concrete structures, by short-circuiting the bunch of wires and taking a single lead to the main supply line cathodic effects can be avoided.

(e) Newer material, like, Parafil tendons, which are electrically non-conducting can be used in place of ordinary prestressing steel.

3.7. Microbial Corrosion

Under suitable anaerobic conditions, species, like, sulphate reducing bacteria cause general or localised corrosion. Incidences of corrosion of prestressing steel wires due to sulphate reducing bacteria have been reported elsewhere.

Anaerobic Bacteria

Common form of bacterial attack results from the metabolic processes of sulphate reducing bacteria (SRB) in which sulphate is utilised under anaerobic condition. Spaces isolated from atmospheric oxygen, especially sulphater-rich clay or organic soils below water table provide conditions suitable for the growth of SRB. In pH range of 6.2-7.8 SRB are most active. Cathodic reaction in a corrosion cell results in hydrogen evolution. Depolarisation of hydrogen takes place in near-anaerobic conditions by residual oxygen. SRB metabolism results in production of sulphide ion which reacts with the metal, resulting in the dissolution of metal and formation of metal sulphides in the anodic region. Further dissolution of metal is accelerated by the presence of sulphides.

Aerobic Bacteria

Sulfur oxidising bacteria, such as, thiobacillus thio-oxidams are capable of oxidising sulfur and sulfur containing compounds to sulfuric acid

$$2S + 3O_2 + 2H_2O \text{ ------ } 2H_2SO_4$$

These species can thrive in pH levels upto 6.0.

Corrosion failures of prestressing steel due to bacterial attack have been identified (Chapter 2).

Prevention of Microbial Corrosion

Some of the preventive measures for microbial corrosion of prestressing steel.

(i) **Protective Coatings**

Coatings, like, bitumen, polyethene wrappings, etc. prevent physical access of the bacteria.

(ii) Chemical disinfection of the nearby environment can be done by adding inhibitors, like, chromates, quarternary detergents, halogenated phenols, etc.

(iii) Free air itself reduces sulphate reducing bacteria population. This can be done by placing chalk around the buried structure which can provide aerobic conditions.

Note

It has been found that hygroscopic grease provided a fertile environment for sulphate reducing bacteria resulting in the failure of prestressing steel of a pressure vessel.

3.8. Factors Influencing Corrosion of Prestressing Steel

3.8.1. Water-cement ratio

Pretensioned Concrete

Normally, it is found that concrete with higher water/ cement ratio produces corrosion[1]. A low water/cement ratio of 0.49 has been recommended. It has been also shown that even a 2 inch (50.8 mm) cover was inadequate when the water/ cement ratio was 0.62.

Percentage of corrosion for 50mm cover was less than 10 per cent in the case of w/c ratios 0.49 and 0.55. But percentage of corrosion was over 70 per cent in the case of concrete with w/c ratio 0.62[2]. Chloride tolerable limit depends on w/c ratio[3]. A water/cement ratio of 0.4 showed a threshold value of 0.26 per cent weight of cement. Interestingly, a lower w/c cement ratio of 0.28 gave a threshold value of 0.17 per cent by weight of cement.

Other workers have recommended an optimum w/c ratio in the range of 0.40 to 0.45[4]. The permeability coefficient was shown to increase rapidly beyond this limit. Standard bridge specifications (USA) has prescribed 0.35 and 0.50 as the lower and upper limits respectively[5].

It has been reported that long term chloride permeability to the 1 inch (25mm) depth level was reduced by about 80 per cent when water/cement ratio was lowered from 0.51 to 0.40[5]. The reduction was about 95 per cent when w/c ratio was further reduced to 0.28. FHWA recommends a w/c ratio in the range 0.32 to 0.44 for corrosion protection.

In both conventional concrete and concrete containing calcium nitrite, corrosion process was reduced when lower water/cement ratio concretes were tested[1].

Post-tensioned Concrete

In the case of post-tensioned concrete, the effect of w/c ratio of grout on the corrosion has not been studied in detail. But, a survey of post-tensioned bridges in USA reveals that the average w/c of the grout is 0.35 to 0.45[6]. Over 57 per cent of the post-tensioning systems in USA have used grout with w/c in this range.

3.8.2. Effect of cracks: Corrosion of steel in concrete depends on the width of cracks. In the case of prestressed concrete, both prestressing and high strength requirements are conducive to dense, impervious and non-absorbant concrete. Thus, cracking possibility is much less in the case of prestressed concrete than conventional reinforced concrete.

Sources of cracking in prestressed concrete bridges are, (a) deflection, (b) Shrinkage, (c) unsafe aggregates, (d) unsafe aggregate-cement combination, (e) corrosion and (f) frost.

In the case of pretensioned structures, there are reports that fine cracks even in heavily polluted atmosphere were not detrimental to prestressing steel[7].

Extensive laboratory and field tests have shown that the amount and extension of the corrosion increases with increasing crack widths[8]. Crack widths upto 0.004 inch (0.1mm) did not produce any corrosion. For crack widths in the range 0.1-0.25mm severe corrosion of steel was noticed.

Longitudinal cracks frequently occur in the anchorage zones of prestressed concrete members set-up by the concentrated forces[9,10].

In the case of post-tensioned structures when tendons in cement grouts are stressed cracks within the tendon bond length tend to occur at about 50 to 100mm apart[11]. Based on the work done by various workers, an upper limit of crack width of 0.1mm has been specified from the point of view of corrosion.

Cracks formed in the grout drastically increases the ingress of aggressive ions and gases. Carbonation front develops at the crack tip and proceeds to the surface. Availability of oxygen and aggressive ions make the environment near the steel highly corrosive.

3.8.3. Cover: For pretensioned concrete structures an adequate cover over the prestressing steel must provide required permanent corrosion protection.

It is not enough to specify a nominal clear cover. The clear cover and the bar diameter together as the ratio C/d_b determine the corrosion resistance[2]. From two year exposure studies, it has been shown that for a w/c ratio of 0.49, an optimum range of C/d_b ratio was found to be 2.5 to 3.0 for adequate protection.

It has been found from the available literature that the effectiveness of concrete cover is relative since it depends on its permeability, quality and on the environment. A minimum cover thickness of 1 inch (2.5cm) has been recommended by FIP[12]. In aggressive atmospheres the cover has to be increased to 1½ inch (nom. 3.8 cm) while for marine conditions a minimum cover of 2½ ft (nom.6.3 cm) is required.

It has been shown that when the cover was reduced from 4 cm to 1 cm for a concrete with w/c ratio = 0.50, the permeability number increased more than four times[4]. Quality of cover depends on the quality of concrete. Quality of cover can also be expressed in terms of its permeability. DnV specifies a permeability of 10^{-12} m/sec.

It has also been shown that the risk of long-term corrosion increased when the actual cover was less than $1^3/_4$ inch[3]. For usual bridge members clear cover of 2¼ inch or less increases the risk of early corrosion. For concrete with 0.44 w/c ratio, a clear cover of 50 mm provided good protection during two years exposure. At the end of 2 years, the chloride content at 50mm depth was well below the tolerable limit. For prestressed concrete bridge superstructure the minimum clear cover to the untensioned reinforcement and prestressing cable has been stipulated as 50 mm and 75mm respectively by IRC[13].

3.8.4. Curing Practices: In the case of pretensioned concrete members, the method of curing has been shown to affect the permeability of concrete and corrosion of steel.

For a 2 per cent calcium chloride containing concrete, steam curing at 82°C (180°F) resulted in more severe corrosion than in concrete cured in water at 82°C (180°F) after 3 years of exposure to industrial atmosphere[14].

Gray bars in the precast prestressed concrete members were better protected from chloride induced corrosion if heat cured, compared to moist cured members[3]. Over 85 per cent of gray bars in heat cured prestressed concrete piles and bridge decks did not develop significant corrosion eventhough the final chloride content was found to be 2-6 times greater than the tolerable limit.

It is interesting to note that eventhough heat cured precast prestressed members showed final chloride level exceeding threshold level at depths of about 37.5mm to 43.8 mm, they absorbed 30-50 per cent less chloride in first 1" than moist cured concrete. Permeability of heat cured AASHTO - Quality Concrete has been found considerably lower than that of 3 days moist cured concrete.

For the moist cured concrete, the chloride level at the 25mm depth was around 15-20 times that at 50 mm depth. But for heat cured concrete the chloride level at 25mm was around 20-30 times that at 50mm depth.

3.8.5. Atmospheric environment: There have been incidences of failures of prestressing steel during storage and during the period lying in the duct till grouting. The reason for these failures was found to atmospheric corrosion. Atmospheric corrosion is the material damage caused by chemical reaction of the environmental constituents with the material. In the case of prestressed concrete structures marine environment, environment containing hydrogen sulphide and industrial atmosphere containing sulphur oxides are detrimental. Carbon dioxide in the atmosphere deteriorates the concrete.

Putrefaction of organic compunds and sulphate reducing bacteria produce hydrogen sulphide. Prestressing steel kept in coil form during storage is under tension. Hydrogen sulphide attack will result in embrittlement of prestressing steel. Also, during the period lying in duct after tensioning, hydrogen sulfide contaminated atmosphere results in failure of prestressing steel. Failure of prestressing wire occurred within 10 days of tensioning during a bridge construction. Use of lamp black, sulfur, motor oil and kaolin at the site resulted in hydrogen sulphide production.

Carbonation occurs when atmospheric carbondioxide reacts with concrete. This results in alkalinity decrease. Any further ingress of aggressive ions, like, chloride is accelerated.

Highly humid and aggressive gases of steam locomotive have resulted in long range deterioration of bridges and over-passes.

High humidity of river environment and marine environment also result in the corrosion of prestressing steel.

It has been established that the relative humidity in non-injected ducts reaches very high values, even upto 100 per cent. Composition of water within the duct was found to contain considerable amount of chlorides and sulphates. Chlorides in the range 220-335 ppm and sulphates 2800-4880 ppm were found 8 out of 14 ducts. pH value of the water was in the range 7.1 to 11.1. Laboratory studies showed that prestressing steel in these conditions could undergo hydrogen embrittlement[15].

3.8.6. Grouting and guniting defects: Improper grouting practices result in voids within the ducts. Very high humidity is developed inside these voids which results in localised corrosion of tendons.

As high alumina cement contains calcium sulphide, hydrogen sulphide liberation can occur under moist conditions. Therefore, use of high alumina cement will lead to accelerated corrosion of prestressing steel. FIP Commissions Report of Prague Congress has banned the use of alumina cements obtained from blast furnace slag[16].

In the case of pretensioned concrete guniting carried out for rehabilitation purposes, if not proper, results in accelerated corrosion. If the gunite mortar does not have proper bonding with parent concrete, fine capillaries are produced as the interface of the two. This results in rapid seepage of water and chloride. Service failures of some prestressed concrete tanks were due to similar lack of compatibility between new and old concrete. Formation of sand pockets during guniting, also serves as a source of moisture. Localised attack of prestressing steel occurs in these regions.

3.8.7. Effect of calcium chloride addition: There have been many incidences of service failures of prestressed concrete, due to the addition of calcium chloride for quick setting. It has been reported that structures built with addition of 2 per cent calcium chloride by weight of cement failed under very light load and the reinforcement was found to be corroded more than half-way through[17].

Irrespective of the type of cement used prestressed wire was found to be corroded in 4 per cent calcium chloride solution[18]. It was also shown that the addition of calcium chloride reduced the tensile properties of prestressing steel in concrete. As the concentration of calcium chloride was increased reduction in tensile strength was more. By the same workers it has been reported that stress-relieved wires suffered more damage than as-drawn wires.

Severe pitting of prestressing steel was observed in concrete containing 2 per cent chloride exposed to industrial atmosphere for 3 years[14,17]. The type of cements and combination of cements showed little effect.

Prestressing steel in high strength concrete, 2 or 5 per cent $CaCl_2$ addition produced superficial rusting during 12 months exposure to synthetic seawater[19,20].

Severe pitting of prestressing steel wires has been reported by the addition of calcium chloride to concrete[14]. Results of these workers were in line with earlier studies in that an increase in calcium chloride addition resulted in severe pitting. It was also shown that prestressing steel in both ordinary portland cement and sulphate-resisting portland cement showed almost the same degree of pitting.

Cyclic wetting and drying induced corrosion of prestressing steel in concrete with 2 per cent calcium chloride[21].

Pitting of prestressing steel was reported in concrete containing 5 per cent calcium chloride[22]. The Author also suggested that cracking of prestressing steel in chloride contaminated concrete was due to pitting leading to plastic fracture.

In concrete containing calcium chloride the presence of large voids adjacent to the wire may result in increased corrosion[18]. Other workers have also reported that poor quality or porous concrete allowed a more severe corrosion in the presence of calcium chloride.

In the case of prestressed concrete piles, a 16 years exposure test has shown that additon of calcium chloride more than 1 per cent by weight of cement resulted in wide spread rusting throughout the prestressing steel bar[23]. In piles not containing calcium chloride prestressing steel was almost free of corrosion. Calcium chloride addition more than 1 per cent resulted in deterioration in the elongation. On the contrary, calcium chloride upto 2 per cent did not induce any deterioration in the mechanical properties of prestressing steel[24].

Chloride Tolerable Limit

Though use of calcium chloride has been discouraged by many agencies, significant research has been done to find out maximum permissible chloride content for prestressed concrete.

It has been found that there exists a tolerable limit of chloride beyond which corrosion effects are significant. Taking into consideration, all the sources of chloride contamination, various agencies have formulated standards for permissible chloride levels, Table 3.1.

Table 3.1. Maximum Permissible Chloride Level for Prestressed Concrete

Standard Specification	Chloride by wt. of cement
BS: 8110-1985	0.10
ACI: 318-83	0.06
IS: 456-1978 & 1343-1978	0.06
CEB	0.20
ACI 357-84 (for offshore structures)	0.06
IRC: SP-33-1989	0.06

In comprehensive study to determine the permissible chloride level in prestressed concrete, threshold water soluble chloride limit was between 0.11 and 0.17 per cent by weight of cement[12]. An upper chloride level of 0.05 per cent was recommended by some other workers.

In Germany permissible levels of chloride are 0.1 per cent for cement, 0.3 per cent for mixing water and 0.02 per cent for aggregate.

Data indicate that a single permissible level of chloride does not apply for all environments. Dissolved oxygen, electrical resistivity of concrete and moisture was found to affect the chloride tolerable limit.

It has been shown that the tolerable limit reduces as pH reduces. This clearly brings out the necessity of a high pH environment near the steel.

It has been found that in 0.04N NaOH medium, prestressing steel had a chloride tolerable limit of 1000 ppm[25]. It was also shown by the same Authors, using break-through potential measurements, that a chloride content of 3000 ppm could induce pitting corrosion[18].

In the presence of other aggressive anions, chloride tolerable limit was found to decrease rapidly. For example, it was shown by anodic polarisation technique that presence of 500 ppm of sulphate ions, reduced the chloride tolerable limit from 150 ppm to 105 ppm in 0.04N sodium hydroxide environment.

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4. MONITORING ASPECTS

INTRODUCTION

In this Chapter, the various monitoring aspects relevant to non-prestressing steel as well as prestressing steels are discussed. In addition, techniques used for studying the structural response are also included. Examination of cement grout is an important aspect and hence techniques which have been used to examine the condition of cement grout and to determine the length of ungrouted portions have been dealt with in detail.

The entire aspect has been considered under the following six major heads:

1. Corrosion monitoring of non-prestressing steel and of prestressing steel in pre-tensioned concrete
2. Corrosion monitoring of prestressing steel in post-tensioned concrete
3. Direct examination of cement grout and prestressing steel
4. Detection of defects in concrete
5. Monitoring the structural response
6. Field inspection techniques and laboratory chemical analysis

Wherever possible the equipment details and the addresses of suppliers have been included.

It has been shown that as on date no foolproof non-destructive technique is available for quantifying the corrosion of prestressing steel in post-tensioned concrete structures. Even with regard to quantification of corrosion of non-prestressing steel, it is to be pointed out that though, electrochemical techniques, such as, Impedance Spectroscopy and Linear Polarisation Technique appear promising, large scale field data are lacking.

4.1. Corrosion Monitoring of Non-Prestressing Steel and of Prestressing Steel in Pre-Tensioned Concrete

4.1.1. Open circuit potential measurements: The tendency of any metal to react with an environment is indicated by the potential it develops on contact with that environment.

In reinforced concrete structures concrete acts as an electrolyte and the reinforcement will develop a potential depending on the concrete environment which may vary from place-to-place.

The probability of steel reinforcement to corrode is assessed by measuring the open circuit potential (OCP) of embedded steel with respect to a standard reference electrode[1]. OCP being a thermodynamic quantity, as such will not indicate the extent and rate of corrosion. In the vicinity of a corrosion site in a structure, the value of corrosion potential will become increasingly negative. Accordingly, potential measurements made between a single half cell and the reinforcement may indicate probabilities of corrosion risk in reinforced concrete structures.

4.1.1.1. Principle: The principle involved is essentially the measuring of corrosion potential of rebar with respect to a standard reference electrode.

4.1.1.2. Equipments needed

High Impedance Voltmeter: A voltmeter with an input impedance of more than 10 m Ω with an accuracy of ± 10 mV is to be used[2].

Address of suppliers

(a) HCL Limited, Instruments Dn.,
41 Deepak Building,
No. 13, Nehru Place,
New Delhi-110 019

(b) Hindustan Instruments Ltd.,
704, Vishal Bhavan,
95 Nehru Place,
New Delhi-110 019

(c) Hindustan Instruments Ltd.,
80, Pantheon Road,
Egmore, Madras-600 008

(d) Meco Instruments (P) Ltd.,
Bharat Industrial Estate,
T.J. Road, Sevree, Bombay-400 015

(e) Universal Instruments Manufacturing Co.,
237 Rajamarial Vilas Extension,
Bangalore-560080

Reference Electrodes

The electrical potential of steel in concrete is compared with that of the reference cell used. The potential of any reference electrode with reference to standard hydrogen electrode should remain stable and any secondary effects, such as, thermal coefficients, etc. should be negligible and be a known quality.

Three reference electrodes commonly used for potential monitoring of steel in concrete are:

1. Saturated calomel electrode
2. Silver/silver chloride electrode
3. Copper/copper sulphate electrode

Calomel electrodes are widely used for condition survey of reinforced concrete structures as the system offers good stability[3]. The system of the electrode is $Hg/HgCl_2$/Sat. KCl. It has a stable potential of +242 mV at 25°C relative to the standard hydrogen electrode. Calomel electrode is prepared as shown in Fig. 4.1.

The silver/silver chloride reference electrode has a potential of +220 mV relative to standard hydrogen electrode. This system is stable and compact but costlier than the other electrodes. It has been reported that current carrying ability of this electrode is low, but photovoltaic effects cause measurement perturbations[4]. Details of silver/silver chloride electrode are shown in Fig. 4.2.

The commonly used reference electrode is copper/saturated copper sulphate electrode, which at 25°C has a potential of +340 mV relative to standard hydrogen electrode. It is rugged for field use. Though, the stability is good, copper sulphate solution functioning as a bridge between reference electrode and concrete reacts chemically with pore fluid present in concrete and hence this electrode is not preferred by some workers[5]. This type of electrode is prepared as shown in Fig. 4.3.

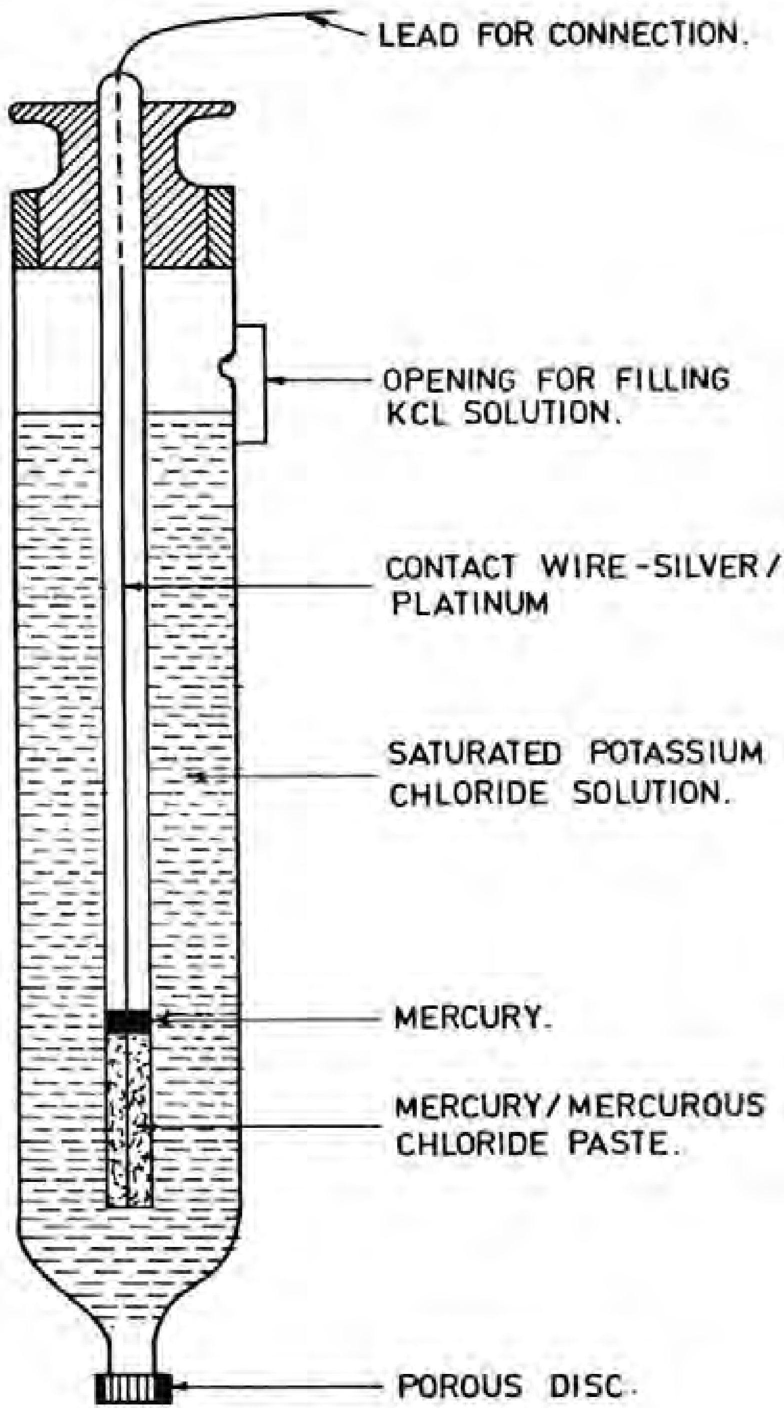

Fig. 4.1. Saturated Calomel Electrode

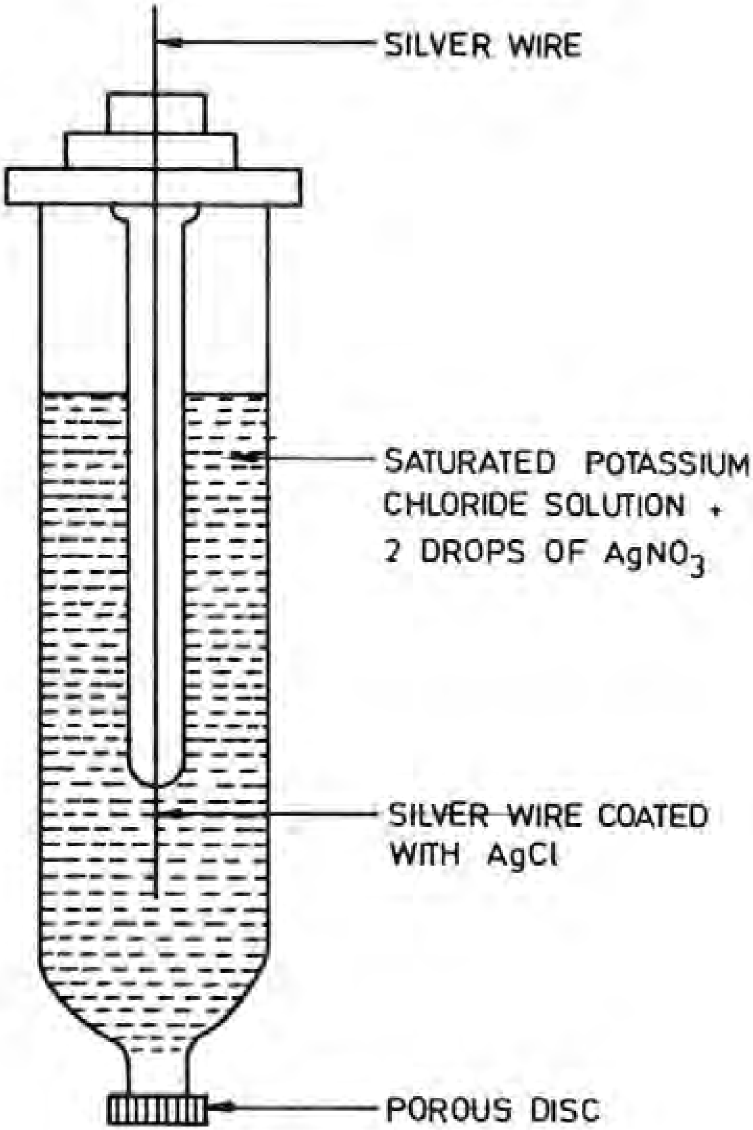

Fig. 4.2. Silver/Silver Chloride Electrode

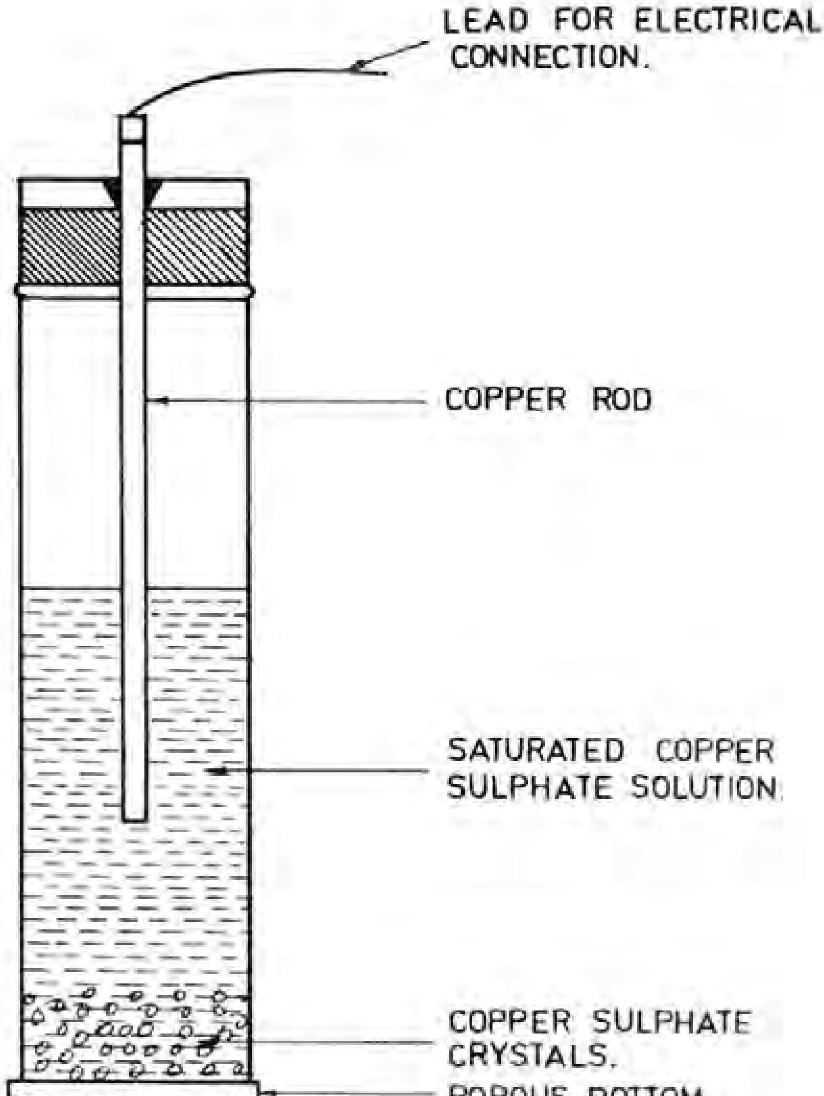

Fig. 4.3. Copper/Copper Sulphate Electrode

Address of suppliers

(a) Elico Instruments Ltd.
Somaji Guda,
Hyderabad

(b) Toshniwal Brothers,
No. 14, Burkit Road,
T. Nagar, Madras.

(c) Elico Pvt. Ltd.,
B-17, Sanat Nagar Industrial Estate,
Hyderabad-500 018

4.1.1.3. Technique: The open circuit potential measurements are made with respect to a reference electrode. The steel rebar should be accessible in few locations for giving electrical connections as shown in Fig. 4.4. The positive terminal of high impedance voltmeter is connected to exposed rebar and negative terminal (common) to reference half cell. The surface of concrete is divided into number of grids. Measurement spacing depends on the overall dimensions of area under consideration. For bridges and similar structures, it has been found convenient to use a 2 ft (0.6 m) survey grid[6,7]. The reference electrode is moved along the nodal points and corresponding potentials are recorded. These are referred to as either open circuit potential or corrosion potentials. As per ASTM standards[28], the probability of reinforcement corrosion is as follows:

OCP Values in terms of		Probability (per cent)
mV vs SCE	mV vs CSE	
more -ve than -275	more -ve than -350	>90
Between -275 & -125	Between 350 & -200	Uncertain
More +ve than -125	More +ve than -200	<10

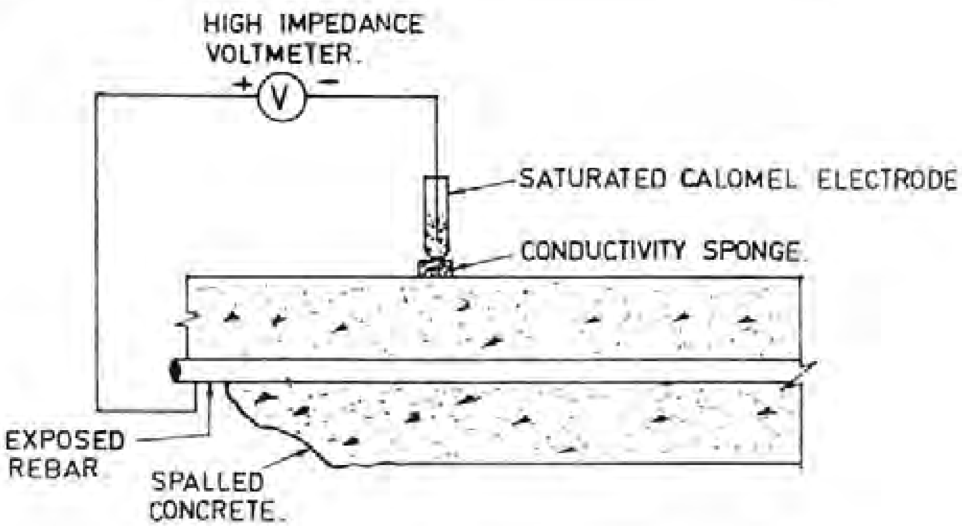

Fig. 4.4. Electrical Circuit for Open Circuit Potential Measurements

4.1.1.4. Laboratory investigations: Potential measurements made on 124 concrete slabs of 1.2 x 1.5 x 0.15 m disclosed that OCP value greater than -510 mV vs CSE was found in some slabs indicating the active corrosion of longitudinal rebar embedded in concrete. The probability of reinforcement corrosion was more than 90 per cent[9].

Work Carried out by CECRI, Karaikudi

Extensive studies carried out by CECRI showed that potential measurements are influenced by moisture content, chloride ion concentration, nature of cement and richness of mix used[10]. Potential measurements were carried out using saturated calomel electrode in active (chloride contaminated) environments and in passive conditions. Steel in concrete exhibited passive conditions in presence of CECRI inhibitor admixture and in presence of CECRI coating. Studies were done in dry and wet conditions. Steady state potentials are more negative in wet conditions. It was also found that potential range was almost the same in chloride contaminated and chloride free concrete in wet conditions. Hence, potential criteria is not reliable for distinguishing between active and passive state in submerged zones of the concrete specimens.

4.1.1.5. Field applications

(A) Applications to Reinforced Concrete Structures

It has been reported by many workers that this technique has been extensively used to assess bridge deck corrosion in USA[11-13]. Potential studies were carried out by California Division of Highways on three different bridges in USA[6]. As the deck slab was covered with asphaltic concrete overlay, it was attempted whether there will be a correlation between potential measurements on sealed deck and actual probability of corrosion. Open circuit potential measurements obtained on the deck were subject to much uncertainty that a definite relationship between the top deck surface and soffit potential could not be established. Potential studies made on asphaltic concrete overlay on bridge deck showed that potential values varied with time leading to large errors in potential value. A considerable increase in potential value was observed on an epoxy sealed bridge deck which indicated that open circuit potential measurements cannot be taken as a reliable criterion in assessing the condition of rebar on sealed deck.

Half-cell potential survey was employed in four storeyed concrete parking structure for ascertaining the corrosion of rebar embedded in concrete. Each floor areas was 3995 m^2 and can accommodate 128 cars. On second floor, 3 areas and on third floor, 2 areas were tested. Only in one area, 32 potential data out of 48 were above the threshold value of -350 mV vs CSE. Highest values obtained were -550 mV which indicated 90 per cent probability of reinforcement corrosion[14].

It has been reported that extensive cracking in floors of seven storeyed parking garage constructed in 1971 in North America showed heavy deterioration due to reinforcing bar corrosion. It can park 1627 vehicles. A total area of 7530 m^2 was surveyed for electric potentials. About 5.9 per cent of floor area measured potentials between -200 mV and -350 mV vs CSE indicating that corrosion activity had initiated only 0.2 per cent of floor area measured potentials greater than -350 mV vs CSE indicating more than 90 per cent probability of corrosion[15].

Activity of corrosion of rebar in concrete columns and balconies was tested using this technique. Corroded reinforcing steel was identified in the 12th and 20th floor of the building using open circuit potential measurements[16]. Other reports which include results of bridge survey are given in[17-21]. Potential survey technique has also been employed in chimney structure[22] for assessing the condition of reinforcing rods in concrete.

Work Carried out by CECRI, Karaikudi

Potential studies in the case of a bridge structure I, which was under distress indicated more than 50 per cent probability of corrosion[8]. Only one portion of the deck showed a highly negative OCP value of -680 mV vs SCE indicating 90 per cent probability of corrosion. Similar potential measurements when made on a bridge structure II which was in good condition showed values in this range -26 to -40 mV vs SCE in 3 locations indicating less than 10 per cent probability of corrosion.

Extensive studies carried out on different members of various bridges indicated 60-80 per cent probability of corrosion in Bridge III while in another Bridge IV, deck slab showed 70-90 per cent probability of corrosion. Substructure showed 100 per cent probability of corrosion[23].

Potential measurements made on various members, such as, roof slab, sunshades, beams of a multistoreyed residential building revealed that in many places, OCP measurements indicated 90 per cent probability of severe corrosion[24]. This agreed with the visual observations made on the structure which revealed the presence of spalled portion of concrete and steel rebars were also exposed in rusted condition.

(B) Application to Prestressed Concrete Structures

Potential survey technique has been employed in 33 pre-tensioned concrete reservoirs in California[25]. Prestressing wire to mortar potential was obtained using CSE. High potential reading was due to effect of galvanized mesh placed over the prestressing wire for holding the mortar. A non-linear relationship between corrosion damage and potential was obtained with the Scatter diagram. Potentials of 16 reservoirs were distributed with a mean value of - 150 mV with variations of ±40 mV vs CSE. This technique was used for ascertaining whether prestressing wires were corroding or not.

This technique was also employed for assessing the extent of corrosion activity in prestressing steel of Route-7-viaduct, pretensioned bridge structure, Chicago[26]. Highest negative potential observed in one area for corrosion activity, did not agree with the results of visual examination, whereas, dome areas showed good correlation between visual examination and potential survey. Surveys of electrical potential measurements in prestressed girders of O'Hare Airport Bridge Leads structure showed that 'Girder 3' was worst affected due to corrosion. About 53 per cent of the potential data was in the range -200 to -350 mV vs CSE and 14 per cent of the readings were more negative than - 350 mV vs CSE indicating 50 per cent and 90 per cent probability of corrosion respectively.

Potential survey made on six post-tensioned girders of Sixth-South Street Viaduct revealed 50 per cent probability of corrosion as the OCP values were within the range -200 to -350 mV vs CSE. However, inspection of prestressing tendon in post-tensioned girders was not feasible.

This technique was applied for inspecting the corrosion of prestressed wires of Rodeo Reservoir Prestressed Concrete Tank of East Bay Municipal Utility Dist., California. Electrical connection was given to exposed prestressing wire. Corroded wires were found where OCP values were between -300 to -620 mV vs CSE. But in some areas, wire corrosion and potential measurements did not correlate[27].

Potential survey made on post-tensioned girders of the Gandy Bridge, Florida, revealed the extent of total corrosion activity in both steel ducts and rods. OCP values more negative than -350 mV vs CSE was obtained on the steel ducts of girders 1 and 2 of Span No.274 indicating severe corrosion[33]. 5 prestressed and 2 post-tensioned beam of Intra Coastal Canal Bridge, Padre Island Drive, USA were surveyed using this technique. In post-tensioned girders, potential readings were obtained on internal surface of walls of box girders. OCP values were less negative than -200 mV vs CSE indicating no corrosion activity. This was in agreement with visual examination.

Work Carried out by CECRI, Karaikudi

OCP measurements in various structural members, such as, girders, diaphragms of a prestressed concrete bridge in Orissa was made by CECRI to understand the extent of rusting on reinforcements[28]. Most negative OCP values in different sections of the Span No.9 ranged from 49 to -380 mV vs SCE. Very high negative potentials of more than -300 mV was obtained on girders and diaphragms indicating 50-90 per cent severity of corrosion.

Since the prestressing wires are housed with the metallic cable sheaths, cable sheaths will act as an intermediate shield and it is not possible to make use of this potential survey technique for monitoring the conditions of prestressing wires in post - tensioned system. Potential monitoring can be applied particularly for assessing the corrosion condition of metallic cable sheaths only. This technique can be applied to prestressed concrete members for assessing the probability of corrosion of non-prestressing steel.

Other New Techniques used for Monitoring OCP of Steel in Concrete

The Research Laboratory at Taywood Engg., U.K., has developed the following corrosion monitoring devices:

1. Potential Wheel, and
2. Data Bucket

These two equipments provide efficient method of measuring corrosion potentials for in-situ measurements.

Potential Wheel

Potential wheel can be used to identify areas of high and low corrosion risk by surveying the structure. It enables a thorough and quick investigation of the structure. Potential wheel is actually a half-cell with a wheel at tip. Wheel is placed in contact with the concrete. It is then drawn on the surface of the structure to provide a continuous record of corrosion potential[11].

Path Finder

In situations, where large number of potential measurements are to be carried out, it is convenient to use 'Path finder' which is incorporated with 8 reference half-cells to enable continuous monitoring rapidly. Measurements are made against copper/saturated copper sulphate electrode.

A large number of bridge decks of Swiss Highways have been investigated using eight wheel electrode-measurement system with computer controlled data facilities. On comparing state of corrosion of rebar with equipotential mapping, potential values obtained showed that potential criteria fixed as per ASTM C876-80 (-350 mV vs CSE) active corrosion does not exist[29].

Address of suppliers

Potential Wheel and Data Bucket

CNS Electronics Ltd.,
61-63, Holmes Road,
London NW 5 3AN

Path Finder

Colebrand Limited
Colebrand House,
20, Warwick Street,
London W1R 6 BE

4.1.1.6. Limitations: This technique gives only a qualitative data. As OCP values are influenced by moisture content in concrete[30,31], valid electrical potentials may be obtained, with proper precautions at any time of the year but during dry season, concrete should be prewetted at the points where OCP is to be taken[6]. Monitoring of reinforcement corrosion by this technique is not reliable in submerged zones and hence applicable only to an apparently dry surface[10]. OCP values are temperature dependent[32].

4.1.1.7. Conclusion: As this technique gives only a qualitative data, OCP measurements in itself cannot be taken as a reliable criterion for assessing the condition of rebar in concrete. OCP measurements prove to be erroneous when concrete surface is sealed with some overlay or epoxy type of coating. This technique can be used for ascertaining probabilities of corrosion of metallic cable sheaths lying in post-tensioned prestressed concrete structures. Equipments, like, path finder, potential wheel, data bucket, etc. are available to make continuous potential measurements. They can at best indicate active areas but are subject to the same limitations connected with open circuit potential measurements. They can give only qualitative data.

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4.1.2. Surface potential measurements

4.1.2.1. Principle: During corrosion process, an electric current flows between the cathodic and anodic sites through the concrete and this flow can be detected by measurement of potential drop in the concrete. Hence, surface potential measurement is used as a non-destructive testing for identifying anodic and cathodic region in concrete structure and indirectly detecting the probability of corrosion of rebar in concrete[1].

4.1.2.2. Equipments needed

(1) A high impedance voltmeter
(2) Two standard reference electrodes

4.1.2.3. Technique: Two reference electrodes are used for surface potential measurements. The electrical circuit for this system is shown in Fig. 4.5. No physical connection to the rebar is necessary in this technique.

In this measurement, one electrode is kept fixed on the structure. The other electrode, called moving electrode is moved along the structure on the grid. The surface of the structure is suitably marked both vertically and horizontally at 20-30 cm intervals. The potential of movable electrode when placed at nodal points is measured against the fixed electrode using a voltmeter. Equipotential contours are plotted as shown in Fig. 4.6 to form a contour map of potential gradients. A more positive potential reading represents anodic areas where corrosion is possible (Note: In the case of OCP, a more negative potential region represents anodic areas). The greater the potential difference between anodic and cathodic areas, greater is the probability of corrosion.

4.1.2.4. Laboratory investigations

Work Carried out by CECRI, Karaikudi

Surface potential measurement studies carried out in reinforced concrete specimen kept

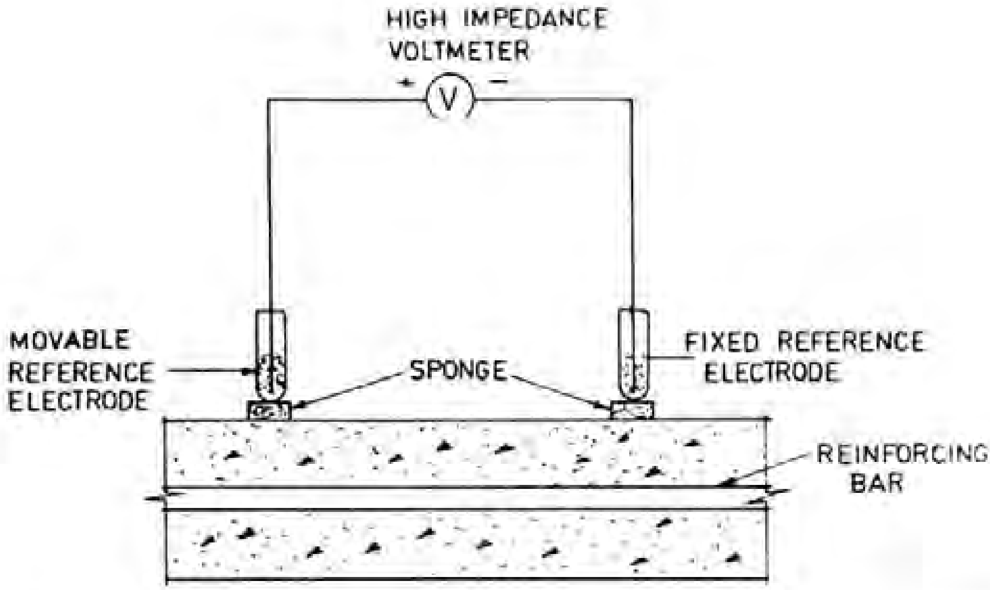

Fig. 4.5. Electrical Circuit showing two reference Electrodes used for Surface Potential Measurements

partially immersed in 3 different solutions, namely, distilled water, 0.1N NaOH and 3 per cent NaCl solution indicated that only in the case of 3 per cent NaCl, immersed portion remained anodic throughout the test period of 200 days[2]. In other environments anodic and cathodic regions shifted positions with time. No appreciable potential difference between 2 regions was observed for concrete specimen subject to normal atmosphere conditions but a drastic increase of potential difference as high as 300 mV vs CSE was observed in saline conditions. This work by CECRI established that only chloride concentration cell can bring about appreciable potential difference. It was also shown that no correlation was obtained between surface potential measurements and corrosion rate.

4.1.2.5. Field applications

(A) Application to Reinforced Concrete Structures

Stratful, in 1957 has adopted surface potential measurements as a method of detecting active corrosion area in San Mateo Hayward bridge, California[3]. Potential measurements were made with reference to copper-copper sulphate electrode. Potential Distribution Pattern identified the affected areas of corrosion in concrete pilings, caps and main deck beams of the bridge structure. Potential mapping was also made on 16 different spans of this bridge. It was observed that anodic and cathodic areas were separated by a distance of 2-10 feet in R.C. bridge deck.

This technique was employed on an abandoned Reinforced Concrete Pilings which was left for 27 years in salt water environment at the bridge site. The measurements indicated that anodic and cathodic areas were separated by a distance of one or more feet. In another pile, maximum potential differences between anodic and cathodic areas on the surface of concrete was -500 mV vs CSE which is indication of steel rebar remaining in active corroding condition.

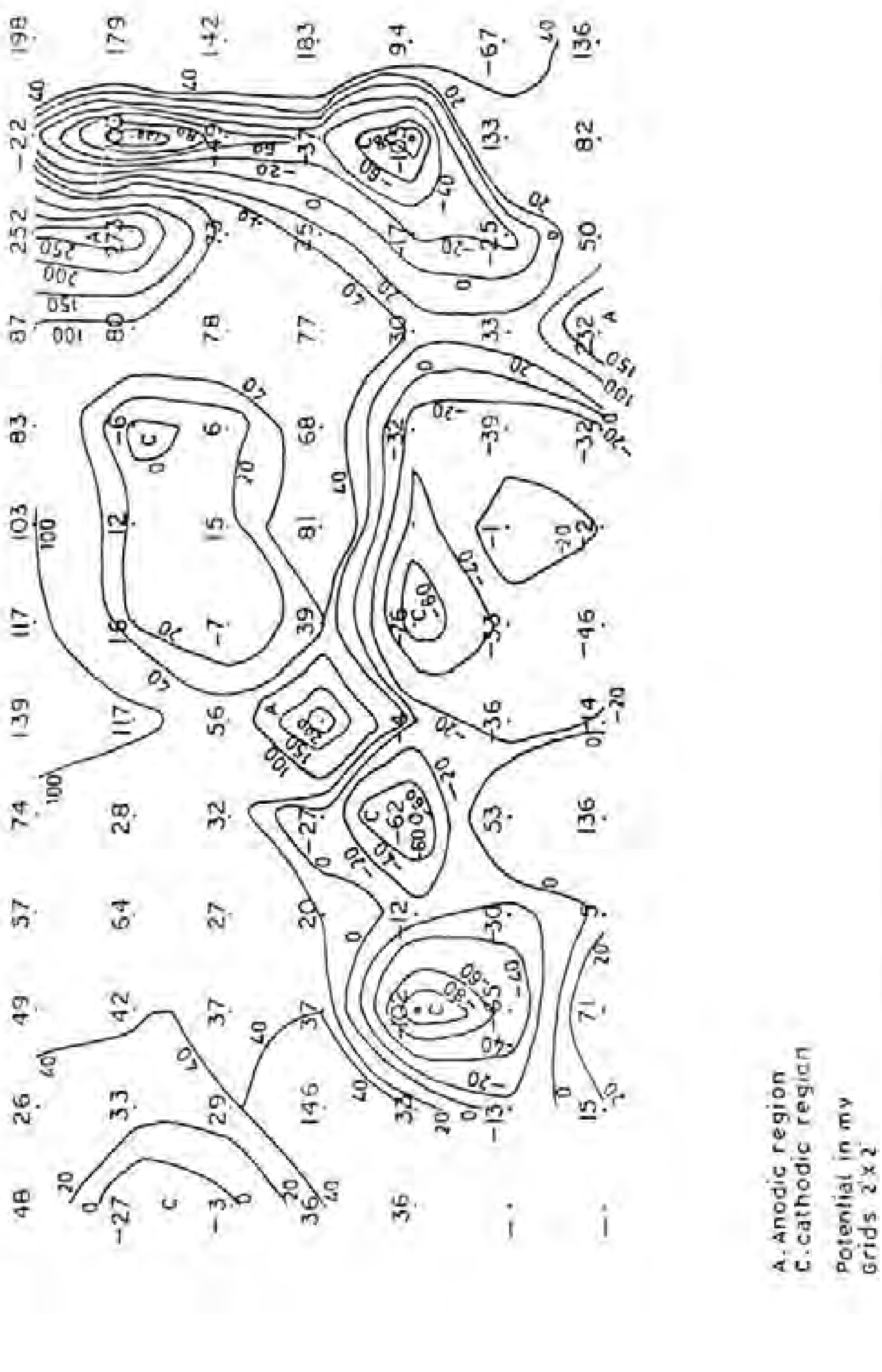

Fig.4.6 Typical Equipotential Contours obtained on a Bridge in Orissa

Deck evaluation was done using this technique. Anodic sites were identified in the bottom of beams and wavy lines obtained on the equipotential contour of surface potential measurements represented cracks in concrete.

It has been reported that whenever a potential difference obtained on concrete structure is not more than 30 mV, it indicates that steel remains in passive condition, whereas, if surface potential difference exceeds 100 mV vs CSE, it indicates active corrosion condition[4].

It has been reported that for precision of locating anodic sites on the bridge structure, a grid spacing of 2 ft is necessary[5]. This technique has been employed by Danish Corrosion Centre for assessing the probability of corrosion in cantilever Balconies. The measurement spacing chosen was of a close grid of 5-10 cm.

Hans Arup et. al.,[6] have carried out surface potential measurements on 7 swimming pools, 2 parking decks and also on concrete tank and concluded that this technique can be used to locate actively corroding areas of steel in concrete.

Work Carried out by CECRI, Karaikudi

This technique has been used for identifying anodic and cathodic regions in various members of bridge structures.

Surface potential measurements was employed in different members of bridge structures for identifying the more anodic areas in the structure[7]. Maximum surface potential differences were obtained on girders, diaphragms and soffit of deck slabs which indicated that active corrosion has set in many places of the above said structure.

Surface potential measurements obtained for a Bridge 'A' and for another Bridge 'B' is shown below[8]:

Details	Max. S.P. difference (mV vs SCE)			
	Location 1	Location 2	Location 3	Location 4
Bridge 'A' (under distress condition)	360	278	268	398
Bridge 'B' (under good condition)	285	150	148	262

It can be seen that potential differences in the case of a deteriorated bridge are invariably higher when compared to those values obtained in an apparently sound bridge.

Investigations carried out in a power house building revealed that this technique, when made after appearance of cracks in concrete structures was not useful in identifying the corrosion sites as initial cathodic and anodic areas had shifted. This was due to repassivation by contact with air due to earlier corrosion[1].

(B) Application to Prestressed Concrete Structures

It has been reported that surface potential differences of about 150 mV vs CSE was found in pre-tensioned prestressed concrete tanks, California, where anodic areas were present indicating the corrosion condition of prestressing wires[9]. Cherry and Miller[10] have investigated the corrosion of prestressed concrete pipe using this technique and observed that initial anodic and cathodic areas had interchanged position after the test period.

Work Carried out by CECRI, Karaikudi

Surface potential measurements were made on different spans of a prestressed concrete bridge in India[11]. Readings obtained on box girders showed that more than 75 per cent of the portions of two spans indicated that variation in maximum potential difference of 200 mV exists which may possibly be due to salt concentration effect.

CECRI's investigation revealed that maximum potential difference in various sections, like, girders, diaphragms, footpaths of a bridge structure in Orissa varied from 68-428 mv[12]. In most cases, values were more than 150 mV except 2 values which were less than 100 mV. In another span, maximum potential difference ranged from 105-429mV indicating higher probability of corrosion.

4.1.2.6. Limitations: Some of the earliest workers[2,13] have reported the successful use of this technique. Later publications by State of California Research Team[5] advised against the use of this system due to its inability to monitor absolute potential. Eventhough this technique does not rely on any electrical continuity in the reinforcement, choosing of a sound area of structure should be made for the static electrode position to avoid any problems due to delaminations in concrete, which may increase the variability of measured potentials. Although, anodic and cathodic areas are identified, the surface potential measurements recorded are only for comparative studies which help in understanding the changes occurring on the structure[5]. This system introduces a Resistivity factor for ascertaining the probability of corrosion in concrete structures.

4.1.2.7. Conclusion: Surface potential measurements by itself will not indicate corrosion behaviour of rebar embedded in concrete. It is to be coupled with the Resistivity measurements to obtain a parameter 'Corrosion Cell Ratio' for assessing the corrosion of reinforcement'.

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4.1.3. Concrete resistivity survey

4.1.3.1. Introduction: Resistivity of concrete is a fundamental property which characterizes the quality of concrete non-destructively. Electrical resistivity of concrete is an important parameter which can be related to various other aspects, such as, strength, porosity, deterioration, etc. It is well known that the reinforcing steel embedded in concrete is protected by the concrete cover and that this protection is mainly due to the high alkalinity and the fairly high electrical resistance of concrete.

As the electrical conductivity in hardened concrete is an electrolytic process which takes place by ionic movement in the pore fluid of cement matrix, conduction process through concrete is influenced by effects of aggregate and capillary pores in concrete[1-3].

Resistivity of concrete is dependent on the properties of cement paste and moisture content[4].

During any corrosion process, corrosion current has to flow from anode to the cathode sites through the electrolyte and the resistivity of the electrolyte has an influence on the flow of this corrosion current. In the case of reinforced concrete structures, the high electrical resistance can impede the flow of such currents. However, resistivity of concrete has been found to vary considerably depending on the moisture content and other soluble salts present in the concrete. Hence, depending upon the resistivity of concrete, the corrosion process can be stiffled or accelerated. Several workers have tried to correlate resistivity of concrete with the possibility of corrosion. Typical data is given below:

	Concrete Resistivity (K-Ohm-cm)	Possible Rate of Reinforcement Corrosion	Reference
1.	< 5	Corrosion almost certain	Cavalier P.G. & Vassie[5]
	5-12	Variable	
	>12	Corrosion unlikely	
2.	< 5	Very high	Taylor Woodrow; Research Lab (1980)[6]
	5-10	High	
	>20	Insignificant	
	>50	Negligible	

(Contd.)

(Contd.)

1	2	3	4
3.	<7 <20	Almost certain corrosion Some corrosion possible	-do-[7]
4.	<10	Correlates with deterioration	Wens R.[8]
5.	>60	Corrosion Nil	Stratful et. al.,[9]
6.	5-10	Rapid corrosion	Browne[10]

Concrete resistivity, thus, becomes an important parameter influencing the durability of reinforced concrete structures.

4.1.3.2. Principle: It can be seen from Fig. 4.7 four metallic probes are placed over the concrete surface at an equal spacing of 'a'. A known current, 'I' is impressed on the outer probes and the resulting potential drop 'V' between the inner probes is measured. The influence of current flow is such that it spreads out vertically and horizontally and takes the shape of hemispherical surface and depth of its influence is proportional to the distance 'a' between the probes. Resistance 'R' is given by V/I. The equation relating resistivity to measured resistance has been derived for the 4-probe method[11].

Resistivity of concrete 'ρ' = $2 \pi a R$

Where a = Inter electrode spacing in cm

R = Measured resistance in K-Ohm

'ρ' = Resistivity of the concrete in K-Ohm-cm

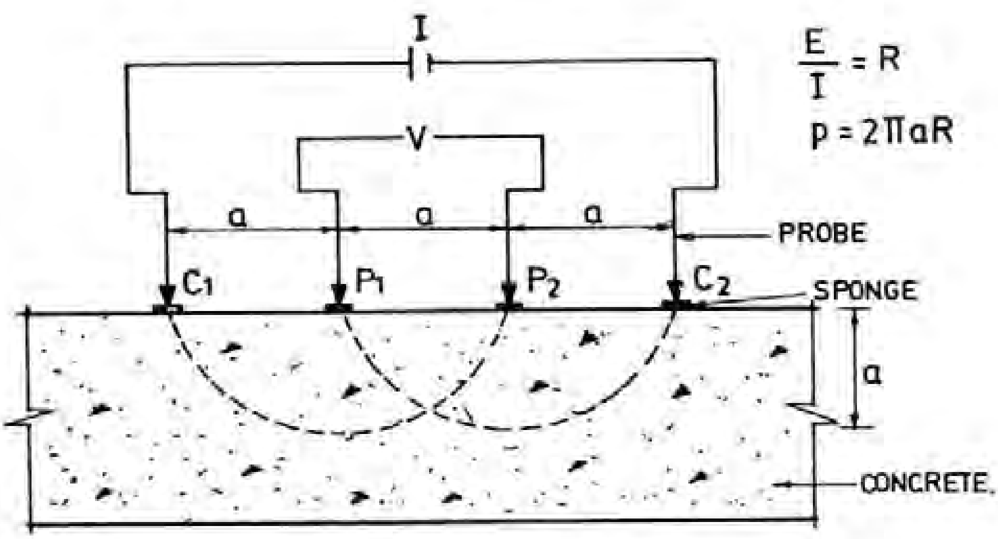

Fig.4.7. Circuit of FPR Meter

4.1.3.3. Equipment needed

Four Probe Resistivity Meter

A four probe resistivity meter[12] based on the above principle has been developed by CECRI and employed for monitoring resistivity of concrete structure.

The instrument consists of two units:

(a) Four Probe Unit

This unit is provided with tips wrapped with sponge or cotton wetted to have contact with concrete when pressed against its surface.

(b) An Electric Circuit Board

This system converts current voltage data in terms of resistance and scaling of the data to display the resistivity directly.

The advantages claimed with this meter are:

1. The meter with built-in-probe is battery operated
2. Rugged type, hence, applicable for field use
3. Portable

Address of supplier

CECRI, Karaikudi, has developed and patented the direct reading Digital Resistivity Meter.

An automatic system based on Rockwell AIM 65 Microcomputer system designed for continuous monitoring of concrete resistivity is also available[13]. It consists of a timing circuit, resistance measurement, temperature measurement and computer system. It yields accurate resistivity and repeatable data.

4.1.3.4. Technique: The tips of the 4 probes of the resistivity meter are wrapped with sponge and saturated with potable water for making effective contact with the concrete surface. The four probes when pressed against the concrete surface indicates the resistivity of concrete directly on the digital panel provided in the meter as shown in Fig. 4.8. This meter gives the average resistivity of concrete in the cover portion provided that the inter electrode spacing 'a' is less than or equal to cover thickness. If 'a' is more than the cover thickness value, then one has to take care to avoid interference effect from steel reinforcements.

4.1.3.5. Laboratory investigations: Studies showed that the resistivity of the concrete decreases as the deterioration increases. Resistivity measurements obtained on concrete specimen subjected to different concentrations of chloride revealed that no significant difference in resistivity was observed in moisture saturated condition, also no great difference in concrete resistivity was observed in the case of equally absorptive concrete. But when concrete was oven-dried after soaking in water for 48 hours, added chloride caused great reduction in resistivity. Studies showed that reduction in resistivity after oven drying increased with increasing chloride contents[14].

California Division of Highways has indicated that Resistivity of saturated concrete increases with time upto about 5 years. Resistivity measurements have been used for detecting defective areas of concrete in water retaining structures[15]. Areas of low resistivity or if particular area had large variations in resistivity values, indicates poor quality of concrete which might be due to salt contamination, excessive carbonation, etching or regions of poor compaction. Studies also revealed that surface measurements of resistivity cannot represent the average true resistivity of the concrete due to skin resistance effect, hence measured value is only apparent resistivity. Because there are polarization effects when measuring resistivity of concrete, an A.C. system in the frequency range of 100-150 Hz is preferred to overcome the high resistance of the concrete. Also, recommended that resistivity measurements on concrete structures are to be made in conjunction with 'Corrosion potential' measurements during a potential mapping survey to assess rebar corrosion.

It has been reported that tests carried out with bimetallic electrodes (steel and copper) embedded in concrete have proved the existence of direct proportionality between the rate of metallic corrosion and electrical conductivity of concrete[16].

Low resistivity values of concrete coinciding with more negative electrical potentials also give information about the corrosivity of concrete[17].

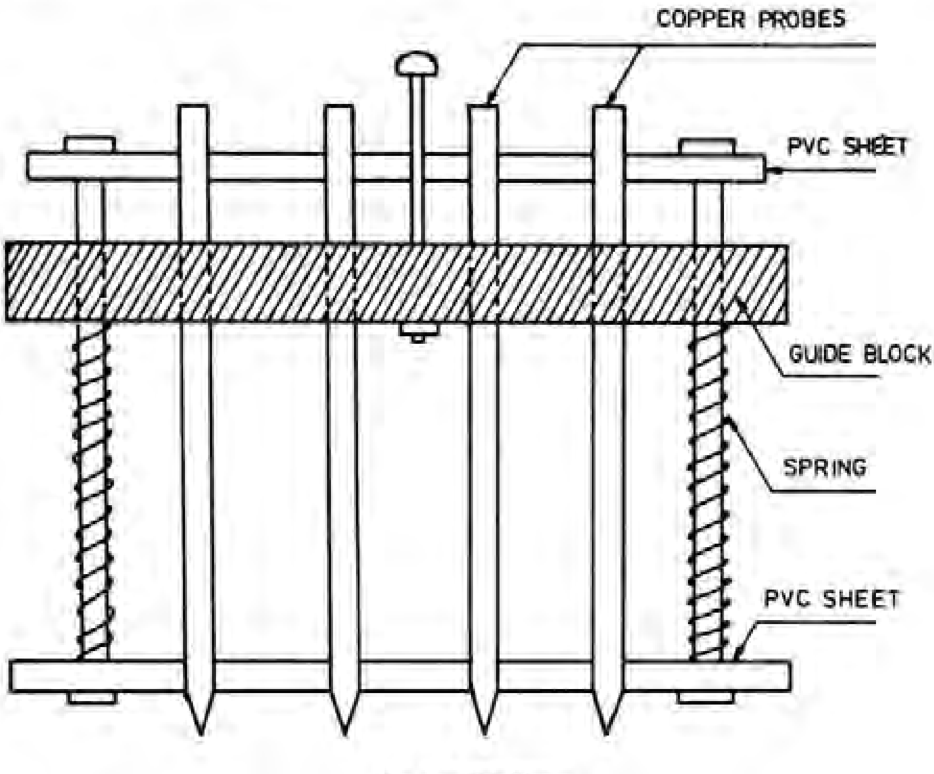

Fig. 4.8. FPR Meter

According to FIP 'Guide to Good Concrete' on Inspection and Maintenance of Reinforced Concrete and Prestressed Concrete System, Electrical Resistance Measurements are taken to assess the efficiencies of water-proofing membranes where the membrane is a dielectrode material[18].

It has been reported that electrical resistivity of concrete of different cement contents of different W/C ratio gradually increased with time and became almost constant after 28 days curing[19]. Hence, water content has a major influence on resistivity of fresh concrete mix. Studies showed that coke, when used as a substitute for conventional concrete, resistivity is lowered and is in the range of 20-122 K-Ohm-cm. Another additive of carbon fibres to concrete along with coke further reduced resistivity yielding value from 13-24 K-Ohm-cm. This study was mainly undertaken for increasing the conductivity of concrete so as to protect the structure using cathodic protection system. When carbon black alone was used, resistivity of concrete ranged from 0.6 to 2.9 K-Ohm-cm. Conductivity of concrete was improved slightly by using carbon black as an additive to concrete.

Work Done by CECRI, Karaikudi

Resistivity measurements have been used for continuously monitoring the strength development in concrete specimen[20]. A linear relationship between resistivity and cube compression strength under continuous curing was established.

Resistivity measurements made on painted surface of concrete and an unpainted surface could indicate the effectiveness of the protective system[21]. Resistivity data was collected on the coated surface of concrete prior to testing and after 70 days of alternate wetting in 3 per cent NaCl solution and drying[22].

In the case of uncoated specimen the resistivity continuously decreased with time and had a value of 4 ± 2 K-Ohm-cm. Except three systems, all other systems had a minimum resistivity of 20 K-Ohm-cm at the end of the test period. It has been reported that electrical resistivity measurements are to be performed for accepting a coating system for concrete surface of any structure.

Studies showed that resistivity varied with moisture content and time[23]. Concrete specimen of size 10 x 10 x 10 cm with 6 mm mild steel rod placed at the centre of the cube was used for the test. Resistivity measurements were carried out in specimen under 3 environment, namely, in air, distilled water, salt spray and 3 per cent NaCl solution for a period of 300 days. Prior to measurements, the specimen were taken out of the respective environments and air-dried. It was observed that specimen cured in distilled water have higher resistivity when compared to the specimen exposed in salt spray and 3 per cent NaCl environments. The decrease in resistivity in 3 per cent NaCl environment was because of chloride permeation and subsequent deterioration of concrete.

4.1.3.6. Field applications

(A) Application to Reinforced Concrete Structures

An investigation of San Mateo Hayward Bridge, California, in 1957, revealed that amount of concrete cracking occurring in the structure varied with resistivity of concrete[27]. Electrical Resistivity measurements made on concrete beams of 12 different spans revealed that at zero deterioration or when concrete is uncracked, resistivity value was more than 60 K-Ohm-cm[14].

Work Carried out by CECRI, Karaikudi

Resistivity measurements have been reported to be useful and non-destructive[20]. Resistivity studies carried out by CECRI on bridge structures[21] revealed that resistivity values of concrete in a distressed bridge structure, 'A' was on the lower side in the range of 13-36 K-Ohm-cm indicating the poor quality of concrete. In another bridge 'B' which was apparently under good condition, resistivity values on the higher side 700-800 K-Ohm-cm indicating 0 per cent deteriortion. In one of the girder of the Bridge Structure 'A', resistivity values lower than 60 K-Ohm-cm was also observed in some areas[28]. Soffit of the deck slab showed very low resistivity values. Electrical resistivity measurement was made at 2'-3' intervals along the deck surface to ascertain the quality of concrete. Average resistivity value of different spans was in the range of 25-38 K-Ohm-cm which was on the lower side. Low resistivity values obtained (less than 60 K-Ohm-cm) indicate the poor quality of concrete. Deteriorated condition of concrete on the top of the deck surface was, thus, identified using this technique.

Studies showed that initial resistivity for all concrete was 3 K-Ohm-cm[24]. Two-fold increase in resistivity with time was observed with Porland cement concrete. Pozzolana concrete showed 3-6 times increase in resistivity value. One will, therefore, be tempted to conclude that in the case of pozzolana cement concrete, concrete is more tightly packed leaving less connected pores which is due to reaction of pozzolana with free time liberated during setting time to form calcium silicate compounds. But, however, subsequent observations on the field performance of pozzolana cement concrete indicated that pozzolana concrete was more porous than OPC concrete. So, this increased resistance has to be attributed to the increased porosity.

Electrical resistivity was measured on different grades of concrete and for different curing periods in both wet and dry conditions[25]. Parameter 'ρ_{dry}/ρ_{wet}' was used as an index for assessing the porosity of concrete specimen. It was found that the ratio gradually increases as the mix becomes leaner. M10 was found to have a ratio of 8.63 at 28 days curing compared to 3.87 obtained for M40 mix.

Electrical resistivity of deteriorating concrete was measured for different grades of concrete and for different ages of concrete[25]. The media used as deteriorating agents were sulphuric acid, hydrochloric acid and sodium chloride 5 per cent solution. A graph relating resistivity with age of concrete is shown in Fig. 4.9. It can be seen that there is a good correlation.

Electrical resistivity technique was tried as a tool for studying the porosity in concrete structures. Electrical resistivity of such porous concrete will be very high under dry condition 'ρ_{dry}' because of pores, when saturated with water, pores are filled with water and hence will be very low[25]. Higher 'ρ dry/ ρ wet' ratio indicates higher porosity of concrete. Typical resistivity rate values obtained for RCC column under good and deteriorated conditions are given in Table 4.1.

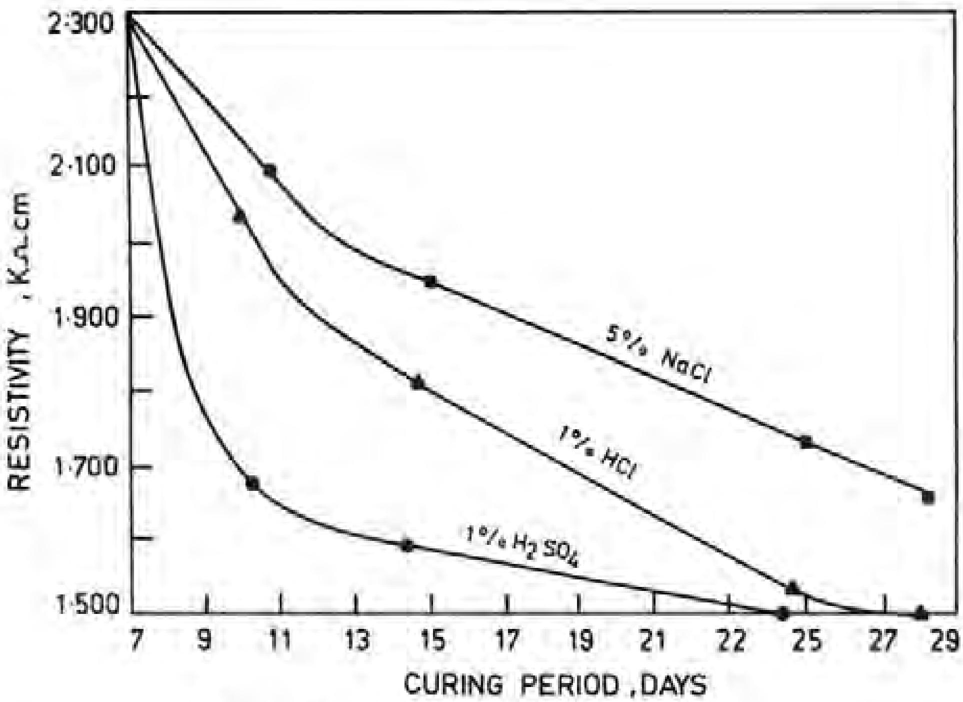

Fig. 4.9. Deterioration of Concrete in Chemical Solution

Table 4.1. Resistivity Ratio Values Obtained for Existing Concrete Structures

Details	ρ dry	ρ wet	ρ dry/ρ wet
RCC Column (Good condition)	90 ± 5	55 ± 5	1.65
RCC Electric post (Highly deteriorated)	58 ± 13	6.5 ± 1.5	8.92

This clearly brings out that the ratio 'ρ dry/ρ wet' is about 5 to 6 times higher for deteriorated column when compared to undeteriorated column. Thus, 'ρ dry/ρ wet' ratio gives an idea about the porosity of concrete.

(B) Application to Prestressed Concrete Structures

Work Done by CECRI, Karaikudi

Little information is available for prestressed concrete structures. Concrete resistivity on the girders of a prestressed concrete bridge structure in Maharashtra showed values greater than 60

K-Ohm-cm indicating the good quality[29]. It was found that porous concrete also had very high value of resistivity. Cross diaphragm which appeared to be porous in nature, when surveyed using this technique showed a value in the range 495-947 K-Ohm-cm.

Resistivity studies were carried out on painted and unpainted surface of the girders and diaphragms of prestressed concrete bridge structure in Maharashtra[30]. Solvent free epoxy system had higher resistivity value than aqua epoxy coated surfaces. The unpainted surfaces showed lower values. Hence, performance of the protective system on concrete can be monitored using resistivity measurements. Resistivity survey was also made in number of bridges in various States, namely, Orissa, Goa, Kerala, etc.[31] In one of the worst affected bridges, resistivity values were invariably less than 10 K-Ohm-cm.

4.1.3.7. Limitations: Correlation between resistivity of concrete with deterioration of concrete and corrosion of steel in concrete is not fully established. Just resistivity alone will not be indicative of corrosion of steel embedded in concrete. It has to depend on parameter 'Corrosion cell ratio'. Porous concrete can also give high resistivity. This necessitates careful interpretation of the obtained data.

4.1.3.8. Conclusion: Resistivity technique can be used as a quality control tool. Periodic monitoring of resistivity measurements enables to identify the deterioration of concrete. However, cautious approach is necessary for interpretation of data obtained. Porosity in concrete structures can be monitored using the parameter Resistivity ratio'ρ dry/ 'ρ wet'. The technique can also be used for monitoring the durability and effectiveness of coatings on concrete surface.

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4.1.4. Calculation of corrosion cell ratio: During any corrosion process, corrosion current flows from anodic to the cathodic sites through the concrete. Hence, the resistivity of concrete has an influence on this flow of current. The surface potential measurements indicate only the anodic and cathodic regions in any particular area of reinforced concrete structures. However, either of the above two techniques by itself alone does not give a quantitative picture of corrosion of reinforcement. The usefulness of both these measurements already described in sections 4.1.2. and 4.1.3 can be realised only on combining both the techniques enabling the calculation of a parameter called "Corrosion cell ratio" (CCR Value) which helps in assessing the condition of rebar embedded in concrete. Stratful[1] has already reported the application of CCR values in assessing the condition of rebar in concrete structures.

4.1.4.1. Definition of corrosion cell ratio: Corrosion cell ratio is expressed as the ratio of maximum surface potential differences in millivolts measured on the surface between anodic and cathodic areas to the average electrical resistivity of concrete in K-Ohm-cm in the anodic region.

4.1.4.2. Calculation of CCR values: For calculation of CCR values, we have to make use of the equipotential contour maps and resistivity contour maps. From the equipotential contour maps, the anodic region and the cathodic region are identified. The maximum potential difference is obtained on substracting the potential obtained in the cathodic region from that in the anodic region. The average electrical resistivity around the anodic region is obtained from the resistivity contour map. CCR for each area/structural component is calculated by dividing the potential difference by the average resistivity.

It has been reported that generally if the ratio is greater than 5, embedded rebars may be undergoing corrosion in the anodic region and concrete is likely, to be cracked[1,2]. CCR value less than 5 indicates that concrete is free from cracking. However, it may be undergoing corrosion corresponding to the CCR value.

4.1.4.3. Field applications

(A) Application to Reinforced Concrete Structures

It has been reported that corrosion cell ratio values calculated for different spans of decks and beams of 16 spans of San Mateo Hayward Bridge revealed the severity of corrosion[1]. Corrosion cell ratio obtained on each span of the bridge structure was correlated to percentage deterioration of concrete. CCR values were greater than 5 in the case of cracked concrete beams.

Work Carried out by CECRI

CECRI has carried out corrosion studies on number of bridges and structures. CCR value has been calculated on many of the bridges. CCR value has been correlated with distress in the structure.

Bridges

Corrosion cell ratio values were calculated for different portions, like, foundations, substructure, deck slab and hand rails for assessing the corrosion susceptibility of different concrete bridges along east and west coast parts of India[2]. CCR values of less than 1 were obtained for the foundation and handrails of two bridges indicating that steel rebar was unaffected by corrosion. Highest CCR value of 9 was also obtained on the deck slab of a bridge in Tamil Nadu.

Survey made by CECRI revealed that CCR values calculated at four locations of one bridge which was under distress were in the range 10 to 20.6 indicating severity of corrosion, whereas, values were in the range 0.18 to 1.09 for another Bridge which was apparently in good condition[3]

Residential Building

Corrosion survey made by CECRI showed that CCR values determined in one of the buildings in Delhi were in the range 0.2 to 1.04. This CCR ratio agreed with the visual observation made on the structure which indicated the absence of cracking and spalling of concrete[4]. Maximum CCR value of 11.75 was obtained in the case of sunshade over W.C. and Bathroom of another building. This CCR value being higher than 5 indicated corrosion damage of concrete. This was also confirmed by visual examination which revealed the presence of spalled portions in many areas of the sunshade of the building.

Industrial Building

Corrosion cell ratio values were calculated for different structural members of a fertilizer industry in Tamilnadu[5]. CCR value obtained was always invariably less than 2 in the case of apparently unaffected portions. However, higher CCR values in the range 5.25-127.66 were obtained in the apparently affected portions indicating severe corrosion.

(B) Applications to Prestressed Concrete Structures

Work Carried out by CECRI, Karaikudi

Corrosion survey made by CECRI on the prestressed concrete T beam of a bridge in Maharashtra disclosed that CCR values obtained was 6.00 indicating chances of corrosion of steel in concrete[2]. CCR values calculated for cross diaphragms were less than 5 revealing less probability of corrosion. This agrees with the visual examination made on the cross diaphragms of the bridge which revealed that concrete in many portions was in good condition.

Studies on prestressed girders of a major bridge structure revealed that CCR values calculated in some sections of the girders were less than 5 in general and in many cases less than 1 indicating that non-prestressing steel reinforcements can be considered to be unaffected by corrosion[6]

Condition survey made by CECRI on a major prestressed concrete bridge in Goa showed that active corrosion had set in many places of the girders, cross diaphragms and soffits of the deck slab[7]. CCR values greater than 5 were obtained in 6 portions of the girder and higher CCR values upto 20 were also obtained indicating severity of corrosion. CCR values as high as 10 and 19 were obtained in the case of cross diaphragm. This agreed with the general deteriorated condition of the bridge.

CCR values obtained in the case of different spans of a prestressed concrete bridge in Orissa considerably[8]. CCR values calculated in one of the span varied from 0.47 to 8.41. In some protions, CCR values were greater than 5 indicating active corrosion. CCR values obtained less than 5 on the left face of one of the girder agreed with the detailed visual examination made confirming the good condition of concrete. Visual examination data on another girder showed that the girder appeared to have been affected more severely. Steel reinforcements had rusted at many places leading to spalling of concrete. However, CCR values obtained on the same girder were of the order of 3.5, hence, CCR values cannot be fully relied upon for assessing the corrosion condition of rebar in concrete particularly when the concrete is highly porous which leads to very high electrical resistivity values and consequently low CCR values.

Appreciable CCR values of ≥ 5 were obtained at few places of box girder in the two spans of a major bridge in Goa indicating higher probability of corrosion[9]. Visual observation data collected by CECRI showed that concrete had spalled off at many locations and were of poor quality.

Another survey made by CECRI on a prestressed concrete bridge in Kerala revealed that CCR values obtained at the two end spans of the bridge were not appreciable[10]. CCR value varied only from 0.19 to 1.19 indicating good condition of steel in concrete. This was confirmed by the visual observations which revealed no rust stains or spalling of concrete.

4.1.4.4. Limitations: As corrosion cell ratio values depend on resistivity measurements, lower values of CCR may also be due to higher resistivity values obtained in the case of porous concrete. Thus, CCR values may be misleading in some cases. However, this parameter CCR value gives only an additional information in assessing the condition of rebar embedded in concrete and judicial interpretation of the CCR values are inevitable.

4.1.4.5. Conclusion: CCR values calculated from the data obtained from surface potential measurements and resistivity data can give some useful information on the condition of embedded steel reinforcement. It may not be useful for quantifying the actual corrosion damage.

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4.1.5. Electrical resistance probe: Electrical resistance technique has been used not only for monitoring the corrosion of mild steel element embedded in concrete but also for monitoring the quality of concrete cover. Recently, electrical resistance technique has been used to get some information on the condition of prestressing steel in prestressed concrete bridges.

Corrosion Monitoring of Steel Reinforcement

The method is based on the fact that when a conductor corrodes the metal lost is replaced by an insoluble non-conducting film which adheres to the metal, or is carried away by the corrosive medium. Metals and alloys have much lower specific electrical resistances than their corrosion products. Since the electrical resistance of a metal depends on its cross-sectional area, a decrease in thickness of a specimen due to uniform corrosion may be evaluated.

4.1.5.1 Principle: In the case of a cylindrical wire/rod, electrical resistance 'R' is related to the cross-section by the equation

$$R = \frac{\rho l}{\pi \gamma^2} \qquad \text{... Eqn. 1}$$

Where ρ is the specific resistance

l is the length of the wire

r is the radius of wire

If corrosion is expressed as a percentage then,

$$\% \text{ corrosion} = \frac{r_o - r \times 100}{r_o} \qquad \text{... Eqn. 2}$$

Where r_o is the initial radius and r is the final radius of wire element. By combining equations 1 and 2, we get

$$\% \text{ corrosion} = 100 \left(1 \sqrt{\frac{R_o - R_e}{R - R_e}}\right) \qquad \text{... Eqn. 3}$$

Where R_o is the initial resistance

R_e is the fixed resistance of the unexposed portion

R is the resistance at any given time

Resistance changes can also be followed as voltage drops, where

$$\% \text{ corrosion} = 100 \left(1 - \sqrt{\frac{Vo - V_e}{V - V_e}}\right) \qquad \text{... Eqn. 4}$$

It is apparent that in using resistance probes one must balance sensitivity with length of the test. In order to achieve increased sensitivity, it is convenient to use thin wire or ribbon elements.

4.1.5.2. Corrosion monitor: Central Electrochemical Research Institute, Karaikudi carried out detailed studies on various probe designs and worked out an optimum probe design which can be conveniently used in reinforced concrete bridges[1,2]. In this design, one section of the probe is protected from the environment by encasement in epoxy resin while the other section is exposed to the same concrete environment as that of the reinforcement.

If d_p and d_e are the diameters of the protected and exposed elements respectively, then the resistance of the protected portion

$$R_e = K/(d_p 2) \qquad \text{..... Eqn. 5}$$

resistance of the exposed portion

$$R_e = K/(d_e 2) \qquad \text{..... Eqn. 6}$$

combining Equations 5 and 6

$$\frac{d_e}{d_p} = \sqrt{\frac{R_p}{r_e}} \qquad \text{..... Eqn. 7}$$

Therefore, per cent corrosion in diameter = $100\,(1-\sqrt{R_p/R_e})$ ..... Eqn. 8

Thus, by measuring the resistance R_p and R_e, the percentage reduction in diameter due to corrosion can be obtained. By using a protected element in series with the exposed element, the effect of temperature on resistance is compensated. By using alternating current, instabilities and errors due to magneto resistive effects can be eliminated. A direct digital display of the percentage reduction in diameter has been achieved. This corrosion monitor is now being manufactured by M/s System Controls, Bangalore[3].

4.1.5.3. Probe installation and instrumentation in bridges: Suitable locations for installation of probes are first selected by the concerned bridge authorities. Probes are normally installed prior to concreting after completion of all formwork and after laying of steel reinforcement network. The probe is kept in position and tied up with reinforcement rod to avoid any dislocation. Since the probe is a representative element for steel reinforcement, the exposed element of the probe is kept as close as possible to the reinforcement and the same cover is maintained. The exposed probe element, thus, represents a steel reinforcement rod of smaller diameter but exposed to the same concrete medium at a specific location. A protected PVC cable brings the signal from the probe. The changes in resistance of the exposed element with respect to the protected reinforced element is measured by a transmitter embedded in the probe itself and sent to a centralised remote location for further processing. Thus, a number of probes installed in various parts of the bridge structure can be centrally monitored. The system uses a microprocessor and all the signals are handled in digits.

4.1.5.4. Limitations: Accuracy of measurement depends on the form of corrosion. If it is uniform corrosion, then fairly accurate values can be obtained. If the corrosion is highly localised and confined to one or two relatively isolated points of the exposed element then some errors may be introduced. Uniform pitting of metal surface may not affect the data adversely[4]. There is a linear relationship between resistance and number of pits of same size[1].

Since the probe gives the corrosion data for the specified location where it is installed, number of probes are to be installed in carefully selected locations.

Installation of probes is to be done skilfully.

4.1.5.5. Feasibility studies: CECRI, Karaikudi initially installed some experimental probes in few girders of Pamban Bridge, Tamil Nadu and monitored its performance for a period of 18 months. Subsequently, Pamban bridge authorities awarded the work of installation of probes to M/s System Controls, Bangalore, who installed such probes in many griders. These probes are being monitored by the Pamban bridge authorities.

In other countries, electrical resistance probes are being extensively utilized for monitoring cathodic protection of steel in concrete[5].

Likely suppliers of resistance probes and corrosion monitors.

1. System Controls,
55 Railway Layout,
Pillana Garden, III Stage,
Bangalore - 560045

2. Cormon Ltd.,
Cormon House,
South Street Lancing,
West Sussex NB 15 Baj (England)

3. Marvel Engineering Company,
9, Deiva Sigamani Road,
Royapetta,
Madras-600014

4. Rohrback Instruments Ltd.,
5A, Oxford Road,
Reading Berks RG1 70G,
(England)

5. Instrumentation Associates,
Hornby Building, 2nd Floor,
174 DN Road,
Bombay-400001

4.1.5.6. Conclusion: This technique has been found to be useful in Monitoring Corrosion of Reinforcements in concrete Structures.

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4.1.6. Polarisation resistance technique

4.1.6.1. Introduction: Techniques for monitoring the corrosion of reinforcing steel in concrete structures are at present limited to mechanical inspection, chemical analysis and iso-potential mapping. These techniques can only provide information on the likelihood or otherwise of the presence of corrosion. They do not provide information on the rate or type of corrosion. Only electrochemical techniques are capable of detecting the onset of corrosion at early stages.

Among the electrochemical techniques, the best known technique for evaluation of instantaneous corrosion rate in the laboratory and in the field is the resistance polarisation method, developed by Stern and Geary in 1957[1].

4.1.6.2. Principle: There is a linear relationship between potential and applied current at potentials only slightly shifted from the corrosion ptential. Based on the kinetics of electrochemical reactions and the concept of the mixed potential theory postulated by Wagner and Traud[2] an equation has been derived which relates quantitatively the slope of the polarisation curve in the vicinity of the corrosion potential to the corrosion current density (i_{corr}) as folllows:

$$i_{corr} = \frac{\beta_a \times \beta_c}{2.303\ (\beta_a + \beta_c)} \times \frac{\Delta I}{\Delta E} = \frac{B}{Rp}$$

$$\text{where } B = \frac{\beta_a \times \beta_c}{2.303\ (\beta a + \beta c)}$$

Here, β_a Anodic tafel slope constant
β_c Cathodic tafel slope constant
$R_p = \frac{\Delta E}{\Delta I}$ = Polarisation resistance

This principle can be applied for estimating the corrosion rate (i_{corr}) of rebars embedded in concrete.

4.1.6.3. Equipments needed: Potentiostat, galvanostat with suitable IR compensation interface, voltage scan generator.

Address of suppliers

EG & G Princeton Applied Research,
Electronical Instruments Division,
P.O. Box: 2565, Princeton,
N.J. 08540, (USA)

Accutrol Systems Private Ltd.,
No.16, 11th Main Road,
Jayanagar, V Block,
Bangalore 560041

Petrolite Equipment & Instrument Group,
5455 Old Spanish Trail,
Houston, Texas 77023,
P.O. Box: 2546, (USA)
Telex: 775312

Concord Instruments Pvt. Ltd.,
38C, Burkit Road,
T. Nagar,
Madras-600017

Cormon Ltd.,
Corrosion Monitoring Systems,
Cormon House, South Street,
Lancing, West Susex BN 158 AJ
(England)
(Telex: 877855 CORMON G)

Marvel Engineering Co.,
9, Deivasigamani Road,
Lakshmipuram,
Royapettah, Madras 600014
(Telex: 041-7200 & 041-6660)

SOLEA,
TACOSSEL Electronique,
72 a 78 rud' Alance,
F-69627 Villurbanne (FRANCE)
(Telex: 300175 F Soleant - Fax (33) 78688812)

Instrumentation Associates
Hornby Building, 2nd Floor
174, D.N. Road
Bombay 400004

4.1.6.4. Technique: In this technique, a small amount of D.C. current (ΔI) is applied to the embedded rebar and the corresponding (ΔE) is monitored. This is called polarisation and this can be carried out from - 10 mV to +10 mV in the vicinity of open circuit potential (OCP). There are three methods to carry out this polarisation.

(i) Galvanostatic method

By applying a small increment of current, the change in potential is monitored. For each increment of current a waiting time of 10 minutes is necessary in order to obtain corresponding ΔE values so that 'Rp' values obtained is free from transitory component[3].

(ii) Potentiostatic method

By applying a small increment of potential, the change in current is measured. For each increment of potential (ΔE) the current value (ΔI) is recorded after 30-60 seconds[4].

(iii) Potentio-dynamic method

By using potentiostat coupled with voltage scan generator the polarisation can be carried out at a particular sweep rate. The best result can be obtained at the scan rate of 5-10 mV/min[4]. Longer waiting periods or slower sweeper rates are not desirable because of the characteristic corrosion process of the steel in concrete, where some changes on the electrode may be introduced by allowing longer period.

Polarisation can be carried out by any one of the above methods and E vs I plot obtained. From this plot, R_p value can be calculated. This is nothing but slope of the curve near zero current.

In order to calculate the instantaneous corrosion rate for reinforcing steel. B values of 26 mV for steel in corroding state and 52 mV for steel in passive state are assumed. These assumed B values have been deduced as those that produce a better agreement between the gravimetric losses and electrochemical results[5].

IR Compensation

As the resistivity of the concrete is very high, ohmic drop (IR) between the reference and working electrode should be eliminated. So, the equipment used to carry out these measurements must have provisions for IR compensation. If IR is not compensated, the R_p value will be over estimated and calculated i_{corr} will be smaller than the real one.

4.1.6.5. Electrodes and instruments: A simple block diagram of polarisation resistance instrumentation is shown in Fig. 4.10. Either two electrode system or three electrode system can be used.

In two electrode system, two rebars of same size are embedded in concrete specimen of suitable size. One can serve as a working electrode (WE) and another as a counter electrode (CE). Saturated calomel electrode (SCE) or a copper/copper sulphate electrode ($Cu/CuSO_4$) or a silver/ silver chloride electrode is used as a reference electrode (R.E.). Nowadays, embeddable electrodes are used to minimize ohmic drop effect. Either graphite, pure zinc, or molebdenum/ molybdenum oxide can be used. This electrode of suitable size is embedded at a distance of 25 mm from the working electrode. Generally, this system can be used for laboratory evaluations only.

In three electrode system, a platinum or stainless steel plate or a lead ring is used as a counter electrode. This system is suitable for field conditions. The counter electrode and the reference electrode are mounted on a wetted absorbant sponge or towel and are supported by a suitable holder.

On using the above mentioned system for field monitoring there is some error introduced due to larger area of working electrode and smaller area of counter electrode. The degree of polarisation gradually decreases with the distance from the position of the counter electrode and

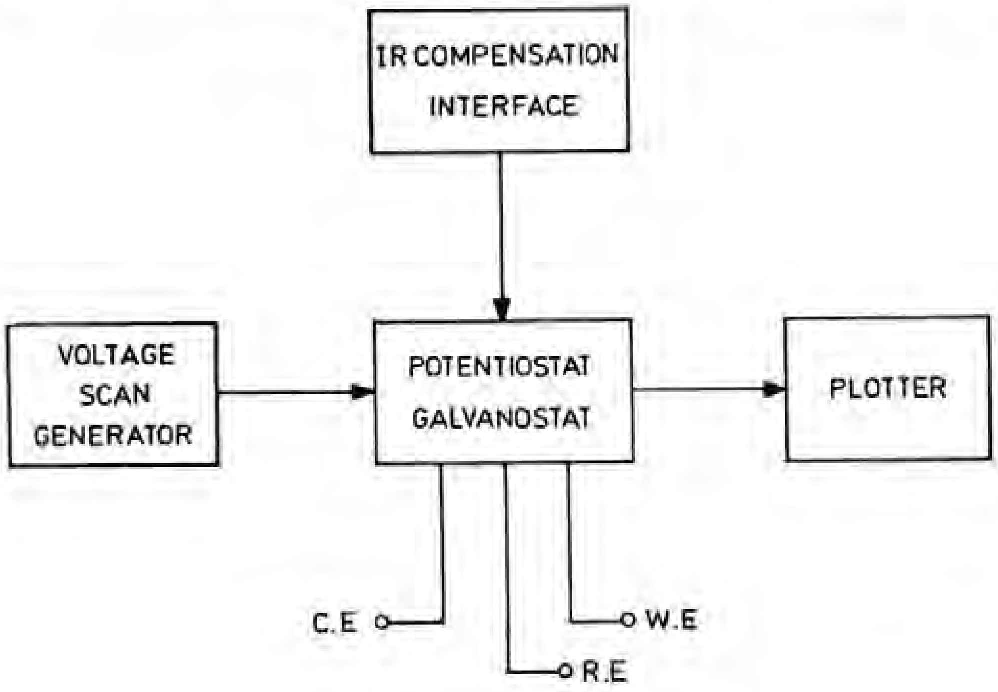

Fig. 4.10. Block Diagram

so the true 'R_p' value can't be obtained. This means that the corrosion rate can't be estimated accurately along each position of rebar.

Above problem is reported to be eliminated by Guard Ring Technique[6]. The arrangement of electrodes are shown in Fig. 4.11. The main concept of this technique is to simultaneously polarise the rebars in concrete by a central counter electrode together with another surrounding counter electrode, and obtain the polarisation current from the well-confined area typically 10-100 cm^2. By this mehtod, the corrosion current density can be accurately determined.

4.1.6.6. Laboratory investigations: Gonzalez and coworkers have attempted polarisation resistance measurements for evaluating kinetic variables of reinforcement corrosion process[7]. They have attempted on cement mortar specimen using two-electrode system. 2 per cent $CaCl_2$ was added as a corrosive agent and 3 per cent $NaNO_2$ was added as an inhibitor to know the effect of carbonation and inhibitor addition on reinforcement corrosion. They also compared the corrosion rate determined by the electrochemical method with the gravimetric method. From the result they concluded that the R_p method can be a reliable, rapid and quantitative technique for measurement of i_{corr}. There is a good correlation between the corrosion rate determined by electrochemical method and gravimetric method.

Gonzalez and Andrade also performed this technique to compare the corrosion rates of galvanized steel and bare steel[8]. The effect of humidity, carbonation and presence of chlorides were taken as the parameters for evaluation. From the results they proved once again that the polarisation resistance is an efficient tool to evaluate the corrosion rate in concrete structures.

Andrade and Casilo have tried the R_p measurements by two electrode system with embeddable reference electrode on cement mortar specimen[5]. From the results they concluded that longer waiting periods or slower sweep rates are not acceptable because of the characteristic corrosion process of the steel in concrete. Therefore, polarisation time used in R_p measurements should be long enough to allow the transitory component to disappear and short enough to avoid changes in the surrounding of steel bar.

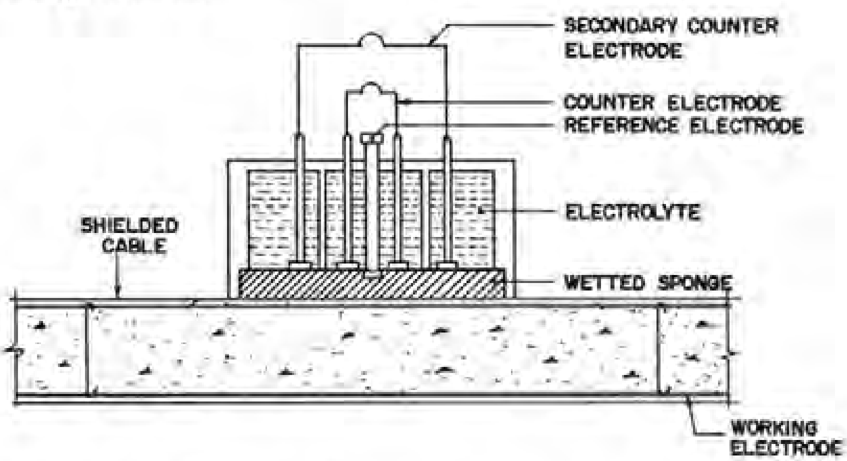

Fig. 4.11. On-Site Corrosion Measurement by Guard - Ring Technique on Slab

Escalante and coworkers used computerised model as a basis for control of the measurement[9]. From the result they showed that there is some correlation between the corrosion rate obtained from the polarisation resistance method and the weight-loss method.

Locke and Oladis Rencon have attempted this technique on concrete cylinders to study the passive behaviour and film stability of the rebar by rapid scan technique at the scan rate of 14 mV/sec[10]. From the results it was concluded that the addition of inhibitor to the concrete can be easily evaluated by this technique.

Feliu and coworkers have explained that the estimation of corrosion rate in large concrete structures by polarisation resistance technique gives only apparent resistance. This polarisation resistance, can be deduced mathematically by 'transmission line' model[11]. They attempted this model study on a reinforced concrete beam of size 160 cm long and 6 x 10 cm cross-section and estimated R_T (True polarisation resistance) by two methods. In one method R_T is related to the apparent polarisation resistance (R_p) and the resistivity of the concrete, whereas, in another method it is related to the decay of the potential applied with the distance from the counter electrode.

The same Authors have also tried, transmission line model to determine the polarisation resistance in reinforced concrete slabs of size 1.3m x 1.3m x 0.07m[12]. From the extensive studies they pointed out that with this two proposed methods it has been possible to clearly distinguish between the behaviour of the steel in passive state (non-corroded) with R_T values in the order of 10^5-10^6 Ohm cm^{-2} and the behaviour in active state (corroded) with R_T values in the order of 10^3-10^4 Ohm cm^{-2}. They also confirmed that in large reinforced concrete structures, the response of electrical signal applied with the aid of smaller counter electrode can be effectively interpreted by transmission line model. In this work, they have not established any relationship between the R_T values and the actual corrosion rate.

Matsuoka and coworkers have criticised the above transmission line model and devised a new on-site as well as laboratory model to find the R_p value using numerical simulations by two dimensional finite element method[13]. They observed that, by transmission line model, corrosion rate cannot be estimated accurately along each position of rebar and the rate may be affected by the reinforcement geometry, the resistivity of concrete, surface film and so on. The Authors rectified the above problems in their modelling analysis and proposed methodology to obtain the true corrosion resistance from the measured polarization resistance (R_p) by utilizing the conversion graph. They have done this modelling analysis for both single counter electrode and double counter electrode methods. From the studies they concluded that the double counter electrode method more effectively confined the polarizing current flow into the constant area than single counter electrode method. The current distribution along the longitudinal direction of rebar had more significance on corrosion resistance data than cross-sectional one.

4.1.6.7. Field applications

(A) Applications to RCC Structures

John and Dawson have attempted the Guard Ring Technique to determine the rate of corrosion on a slab containing a mat of steel bars[6]. From the results they concluded that the polarisation resistance may be sufficient for calculating corrosion rate. However, these measured 'R_p' values should be compared with 'R_p' values measured by other electrochemical methods, such as, electrochemical noise and A.C. impedance spectroscopy to arrive at better conclusion.

Escalante and coworkers measured the corrosion rate using computerised model on three different bridges constructed at different periods[9]. The results are given in Table 4.2. From the studies it has been concluded that it is difficult to assess the reliability or accuracy of data obtained. However, the results can be compared to the visual appearance of the bridge deck in the immediate viscinity of the electrochemical measurements.

Cigna and coworkers used this technique for measuring on-site corrosion rate in a viaduct[14]. The viaduct is near lake Maggiore, Italy at a height of 800 m above sea level. The whole viaduct consists of 38 spans, nineteen for each carriageway. Only ten spans were monitored. The measurements were taken at nine places in each span and ninety places for the whole viaduct. From the measurements, they concluded that the measured corrosion rates were probably too high, because the technique mainly depends upon the local aggressivity of the environment. The mix preparation and the casting conditions are probably responsible for local differences in concrete.

4.1.6.8. Field problems: The most serious problem is that the electronic equipment may be damaged due to the mechanical vibration generated during transportation between laboratory and the field. Another problem is electrical noise generated during measurement, due to interference from A.C. sources, interferences from A.C. motor used to power the oscilloscope, finally interference from current generated by the corrosion of counter electrode. These are eliminated by using shielding cables, battery powered oscilloscope and by using lead as counter electrode. Irreproducibility of working electrode is also another field problem.

Table 4.2. On-site Corrosion Rate on Three Bridges Located at Frederick County, (USA)

Bridge	Average Corrosion Rate MDD*	Visual Observation
54 year-old	0.5	Small cracks randomly scattered over the surface of the deck
17 year-old	1.1	Crack free
12 year-old	1.9	Cracking of concrete, rebar exposed

* MDD - Milligrams per square decimeter per day

(B) Applications to PSC Structures

This technique has not been attempted so far in monitoring prestressed concrete structures.

4.1.6.9. Limitations: From the electrochemical studies, it is inferred that absolute corrosion rate cannot be determined. In many investigations, apparent resistance may be estimated to assess on-site corrosion in a practical situation. By the use of assumed 'B' values in calculating corrosion current density the corrosion rate of steel in concrete with different compositions and exposed to different environmental conditions may yield misleading results. The corrosion reactions are usually very slow, i.e., they have large time constants and field measurements require considerable measurement period.

4.1.6.10. Conclusion: This technique is still in experimental stage and has not been applied to actual RCC or PSC structures on a sufficiently large scale to prove its efficacy.

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4.1.7. Impedance technique

4.1.7.1. Introduction: It is well-known that electrochemical techniques are available for monitoring metallic corrosion and commercial equipments based on electrochemical techniques are being made use of in many industries. However, when we consider monitoring of corrosion of steel embedded in concrete, the fairly high electrical resistance of the concrete poses a major problem in the effective utilization of such techniques.

In recent years, A.C. Impedance Spectroscopy is being experimented as an useful non-destructive technique for quantifying corrosion of steel rebars embedded in concrete. Advantages claimed in this technique are:

(i) the high ohmic resistance of the concrete (which is often a source of error in electrochemical measurements) can be measured and eliminated.

(ii) since very small A.C. signal amplitude ($\simeq$ 10 mV) is used, the surface condition of rebar is least disturbed during testing.

(iii) very low corrosion rates can be measured

(iv) even an idea of the corrosion mechanism operating on the rebar can be obtained by using the frequency spectrum.

(v) the technique gives instantaneous corrosion rate.

Eventhough, the technique is fairly reproducible its usefulness becomes limited if the rebar is essentially passive. If the rebar is active corroding, then only well developed impedance data are obtained.

4.1.7.2. Principle of the technique: In this technique, an A.C. signal is applied to the embedded rebar and the response is monitored in terms of the phase shift of the current and voltage components and their amplitude. This is done in the time or frequency domain using a spectrum or frequency response analyser.

The equivalent circuit of a corroding reinforcing bar embedded in concrete can be represented as shown in Fig. 4.12.

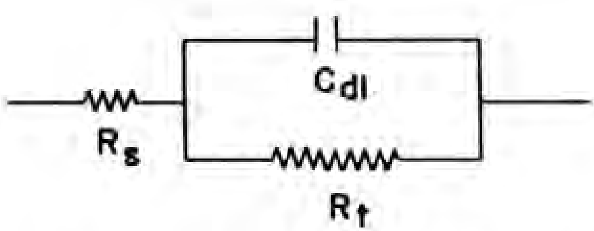

Fig. 4.12. Equivalent Circuit of a Corroding Rebar Embedded in Concrete

R_s is the resistance of concrete,

C_{dl} is double layer capacitance, and

R_t is the charge transfer resistance.

Impedance Z is the ratio of A.C. voltage to A.C. current. An alternating voltage of about 10 to 20 mV is applied to the rebar and the resultant current and phase angle are measured for various frequencies.

As per the circuit, cell impedance

$$|Z| = R_s + \frac{R_t}{1 + j\,C_{dl}.R_t} \qquad \text{... Eqn. 1}$$

Where $\omega = 2\pi f$ and $j = \sqrt{-1}$

As $\omega \rightarrow 0$, the cell impedance $|Z| = R_s + R_t$ and $\omega \rightarrow \infty$, the cell impedance $Z = R_s$. Hence, substraction of cell impedance Z at high frequency from that of low frequency gives R_t.

Corrosion current i_{corr} can be evaluated from R_t using Stern-Geary equation

$$i_{corr} = \frac{b_a + b_c}{2.3\,[b_a + b_c]} \times \frac{1}{R_t} = \frac{K}{R_t} \qquad \text{.... Eqn. 2}$$

b_a, the anodic tafel slope and b_c - the cathodic tafel slope are to be determined experimentally. Since b_a and b_c are constants for a given system, the constant factor K can be calculated. Normally, K has a value of about 25 mV for steel in concrete.

Impedance data for steel in concrete are obtained over a wide frequency range (70 kHz - 10 mHz) to produce a complex plane plot or Nyquist plot (Fig.4.13). Voltage and current measurements are related to impedance by a complex number relationship and therefore the plot contains both real and imaginary impedance components.

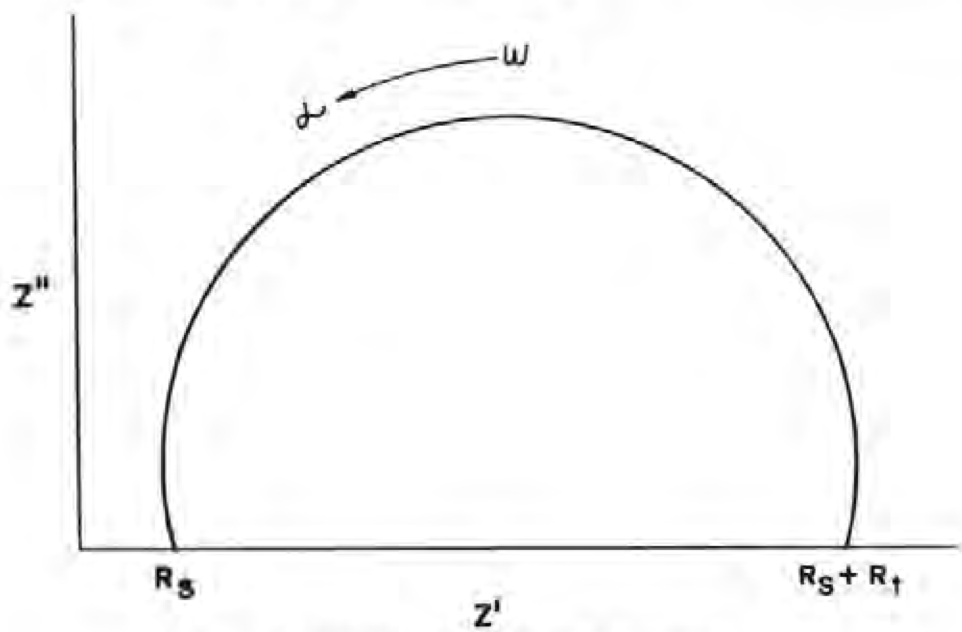

Fig.4.13. NYQUIST Plot for Steel in Concrete

Cell impedance |Z| is resolved into two parts.
Real part $Z' = Z \cos \phi$ and
Imaginary part $Z'' = Z \sin \phi$

Z'' is plotted against Z' at various frequencies. In the case of an actively corroding steel in concrete, a semi-circular curve is obtained. The interaction of the impedance curve with Z' axis yields values for the concrete solution resistance R_s and charge transfer resistance R_t.

R_s is obtained at high frequency, whereas, combined resistance $R_s + R_t$ is obtained at low frequency. Z' obtained at high frequency is subtracted from Z' obtained at low frequency to get R_t value.

From practical situations, it may be possible to get R_t from just two measurements once at high frequency and another at low frequency. Central Electrochemical Research Institute, Karaikudi has developed a corrosion monitor based on this technique[1]. The typical circuit is shown in Fig. 4.14.

4.1.7.3. Equipments needed: Measuring system usually consists of a frequency response analyser (FRA), potentiostat and micro-computer (CPU). Data can be stored in floppy disk.

Address of suppliers

Solartron Instruments,
Victoria Road, Farnburrough,
Hampshire GU147PW, England

HCL Limited, Instruments Division,
41, Deepak Building, 13 Nehru Place,
New Delhi-110019

EG&G Princeton Applied Research,
Electrochemical Instruments Division,
P.O. Box 2565, Princeton, N.J. 08540, USA

Accutrol Systems Private Ltd.,
No. 16, 11th Main Road,
Jayanagar 5th Block,
Bangalore-560041

4.1.7.4. Electrodes and instruments: A simple block diagram of impedance instrumentation is shown in Fig. 4.15. In the three electrode system used, steel rebar embedded in concrete is the working electrode (WE). Electrical contact points using insulated wire should be made at suitable locations. A saturated calomel electrode (SCE) or a silver/silver chloride electrode (sometimes Cu/$CuSO_4$ electrode) is used as a reference electrode (RE). Potential of the steel rebar (WE) is monitored against RE.

A platinum or stainless steel plate can serve as a counter electrode (CE) for passing current through the system. Dimensionally stable electrodes can also be used.

A suitable fixture consisting of CE and RE is made and kept on the concrete surface at the location where measurements are to be taken. This fixture serves as a probe sensor and a wet sponge is kept between the sensor and the concrete surface. Suitable leads are taken from the rebar (WE) to complete the circuit. The sensor is moved over the concrete surface along the rebar profile and readings taken.

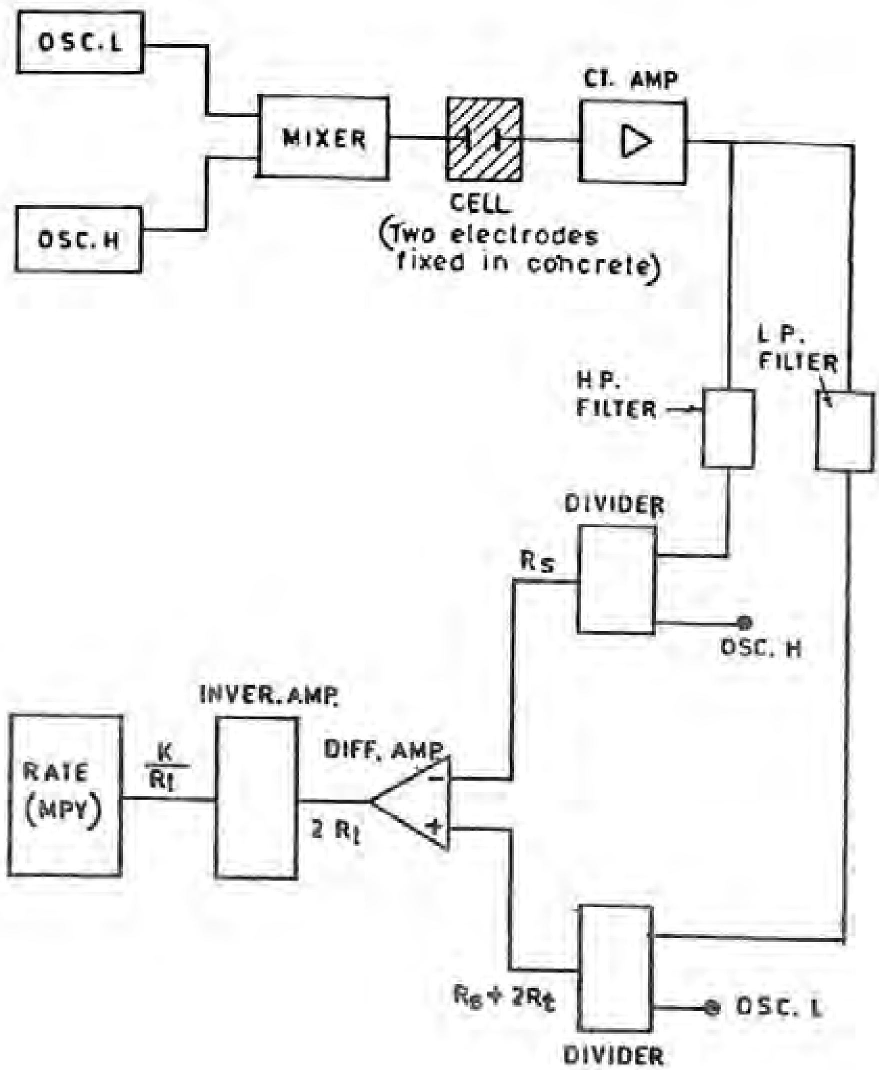

Fig. 4.14. A.C. Corrosion Monitor

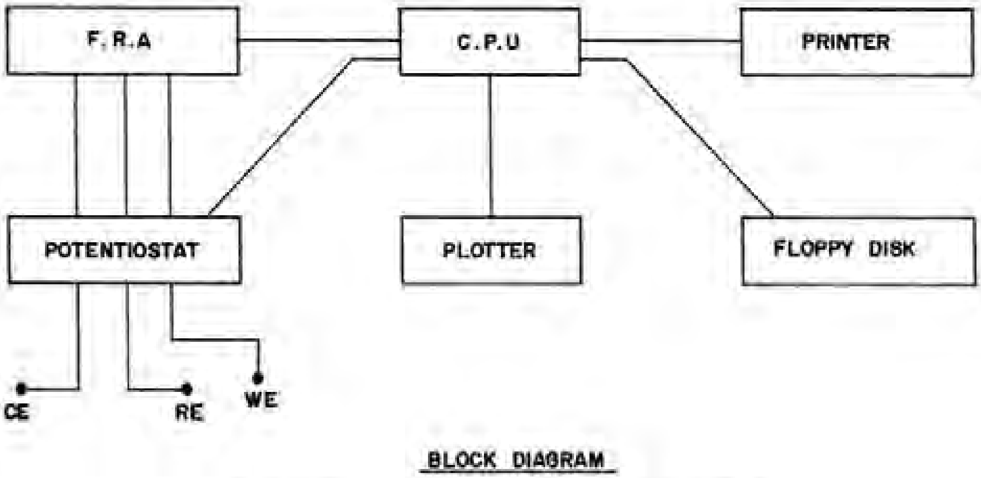

Fig. 4.15. Block Diagram of Impedance Instrumentation

4.1.7.5. Interpretation of impedance plots: Different types of curves obtainable for steel embedded in concrete are shown in Fig. 4.16. The intersection of the impedance data with the Z' axis yields values for

- R_s - the concrete solution resistance
- R_f - the resistance associated with a film at the concrete-steel interface
- R_t - charge transfer resistance associated with the mass transfer of electrons across the double layer (This R_t is equivalent to usual polarisation resistance R_p)

Diffusion rates of electroactive ions can be identified from a diffusion parameter, the Warburg coefficient which is determined from the low frequency behaviour.

Capacitance of the double layer and interfacial film can be determined from the peak magnitudes of the high and low frequency semi-circles.

4.1.7.6. Experimental studies by various research workers: Dawson and Co-workers[2] have reported that two electrode system can detect the on-set of corrosion. Wenger et. al.[3] have also used this technique to monitor corrosion of steel rebars in beams. The constant K was found to lie between 13 and 26 mV in all their studies. K was found to depend on the number of elementary steps in the anodic and cathodic reactions. A model was developed for equivalent circuit taking into account the geometrical aspects.

Elsener and Bohni[4] performed impedance measurements on steel embedded in mortar using saturated calomel reference electrode and cylindrical stainless steel counter electrode. R_t was correlated with the corroded area of steel.

Matsuoka et. al.[5] showed that utilisation of both A.C. impedance and rest potential data can be very useful. They proposed a modified equivalent circuit incorporating concrete resistance R_c, double layer capacitance C_{dl}, anodic polarisation resistance R_a, cathodic polarisation resistance R_c

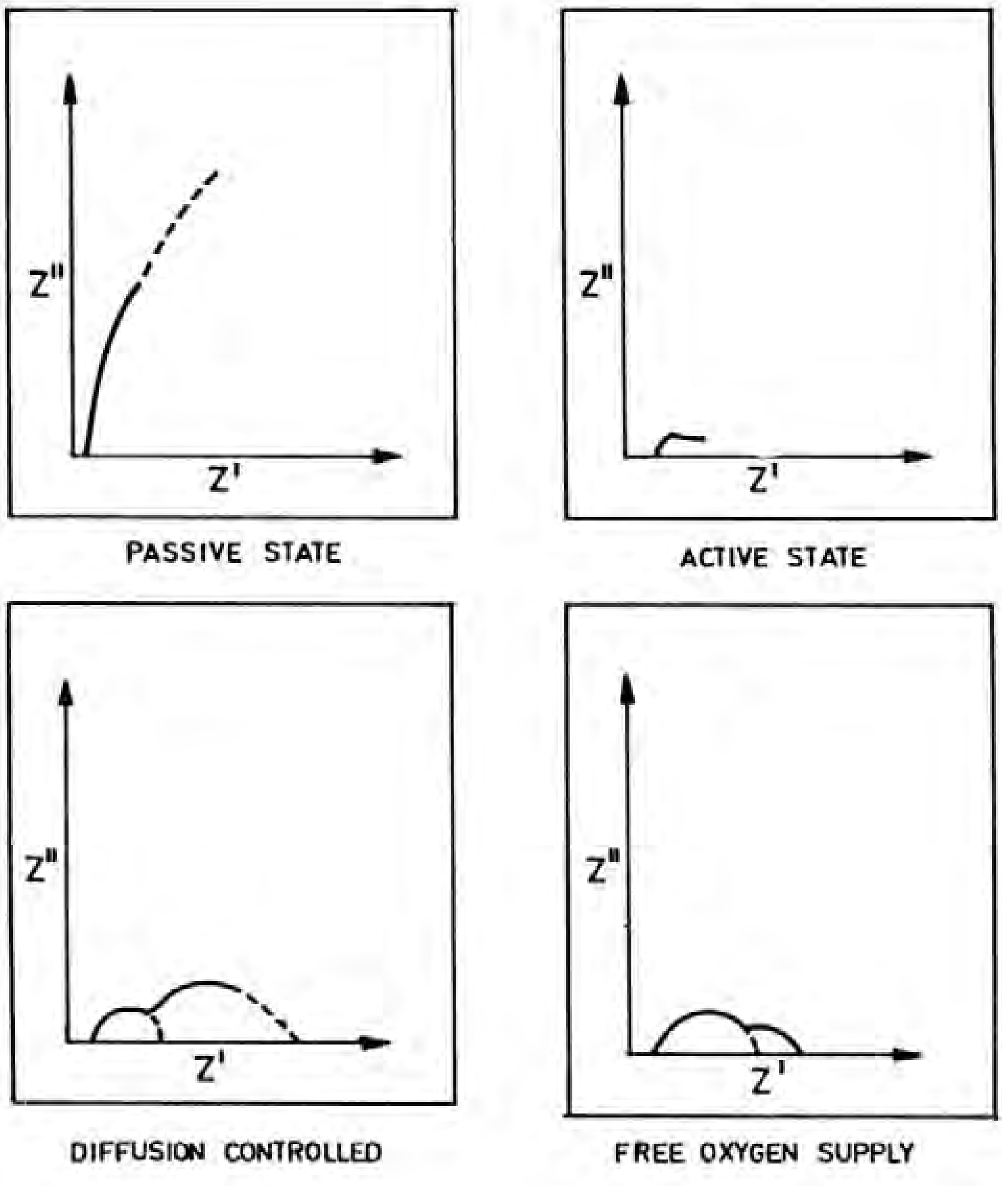

Fig. 4.16. Typical Impedance Plots

and Warburg resistance W. Their study showed that at the initial stage when the rebar may be in uncorroded passive state, rest potential will be noble and a capacitance type impedance curve will be obtained. When corrosion sets in, potential becomes less noble and two types of impedance curves are obtained depending on wet or dry condition of concrete surface (oxygen diffusion process). When severe corrosion sets, a comparatively nobler potential and a small imperfect semi-circle impedance plots are obtained.

Work carried out at CECRI

Laboratory experiments have been carried out at the Central Electrochemical Research Institute, Karaikudi on steel specimen embedded in concrete with different amounts of chloride[6]. Specimen were kept exposed to open atmosphere and impedance behaviour was periodically monitored upto 550 days. Corrosion rate was found to increase with time in the case of steel embedded in concrete containing 5 per cent chloride. Steel coated with inhibited cement slurry showed capacitive type (passive) behaviour even in presence of 4 per cent chloride. Immersion in 3 per cent NaCl solution did not yield any semi-circular plots over 20 months indicating that corrosion had not set in. On the other hand, salt spray produced the expected corrosion effect after a certain period. Thus, it was found that impedance technique can be an useful monitoring technique.

Przyluski et. al.[7] studied the corrosion of steel in mortar specimen with $CaCl_2$ admixture and found impedance spectroscopy to be quite useful in offering more univocal information. A good correlation was found between the values of R_t and a parameter 'β' which is equal to $\frac{2\alpha}{\pi} - 1$, where α is the depression angle.

Qiu and Escalante[8] showed that corrosion of steel in concrete is controlled not only by charge transfer processes but also by diffusion processes as well as other processes. They suggested that instead of charge transfer resistance, impedance modulus could be used. The smaller the modulus higher the corrosion rate. A typical Bode diagram is shown in Fig. 4.17.

4.1.7.7. Limitations in field use: When the impedance study is transferred from laboratory model studies to in-situ field monitoring, some obvious difficulties could be usually visualised. One has to deal with larger area of reinforcement network. Accessibility of rebar network and interference effects may lead to practical problems. The most difficult part is obtaining the true R_t value at each location of the probe sensor since the degree of polarisation induced on the rebar gradually decrease with the distance from the position of the counter electrode. Thus, the main problem is the irregular distribution of the electrical signal applied with a counter electrode of much smaller dimension compared to that of the structure.

To obviate the current distribution problem, a double counter electrode method has recently been proposed[9]. The main concept is simultaneously polarising the rebars by a central counter electrode (CE) together with another surrounding counter electrode (SE). Double electrode system can more effectively confine polarising current flow into a constant area compared to a single electrode system. Distribution of polarising current is found to vary not only with the geometry and the dimension of the reinforcement but also with the resistivity of concrete and the characteristics of surface film. The possibilities and limitations of using such an arrangement have been analysed and it is shown that the confining capacity of double electrode is only effective for high corrosion rates. Andrade et. al.[10] have also analysed the possibilities of confining the electrical signal using a guard ring.

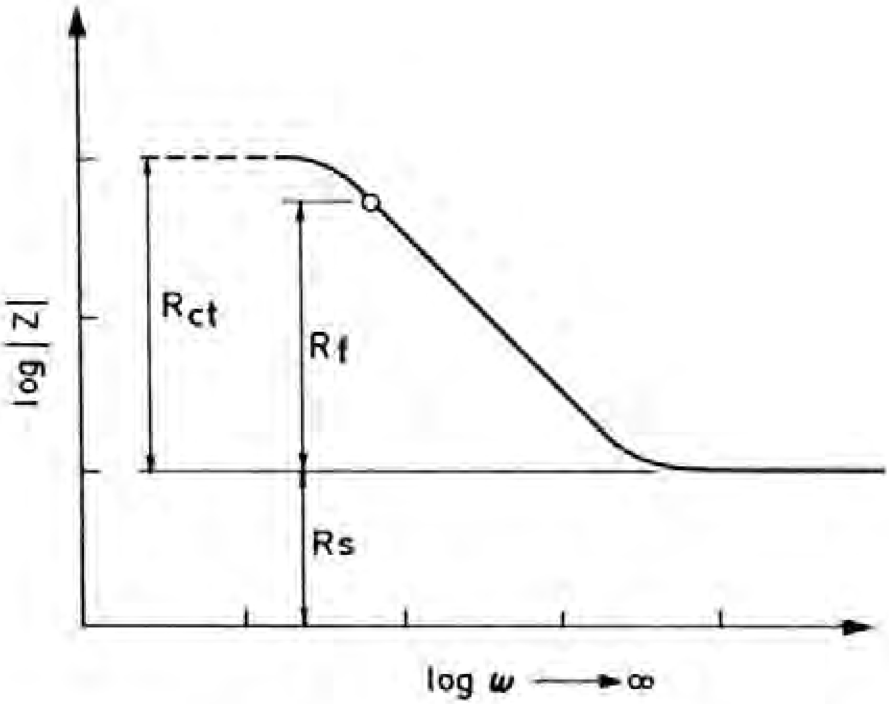

Fig. 4.17. Typical Bode Diagram

4.1.7.8. Simulation models: The applicability of A.C. impedance technique on reinforced concrete structures has been theoretically explained recently[11]. Corroding rebar is simulated as a one-dimensional transmission line of twenty-one meter segments embedded in a uniform concrete matrix. Simulations indicated that corrosion can only be detected at very low frequencies (<1 mHz). Data at best can give only an average corrosion resistance R_t in the whole measured area that may change in a wide area.

4.1.7.9. Conclusion: Laboratory scale studies have shown that A.C. impedance spectroscopy is an useful technique for monitoring rebar corrosion. However, for in-situ monitoring of large scale concrete structures, the problem of irregular distribution of the electrical signal applied with a small probe sensor is to be solved. More simulation studies are needed to accurately interpret the impedance data obtained in field measurements. Rugged and battery instruments tuned to the field requirements are yet to be developed.

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4.1.8. Noise analysis: Electrochemical Potential Noise Technique provides a new method of monitoring corrosion of reinforced concrete structures[1]. This technique enables information on the mechanism and rate of corrosion processes at areas identified in concrete structures. As the noise analysis estimates the instantaneous corrosion rate, it is an added advantage among the already established potential mapping technique.

4.1.8.1. Principle: A low amplitude, variation of the corrosion potential (range of 1μV to 10mV, 10μHz to 1Hz) of steel in concrete is measured to obtain a noise data as a record of potential fluctuations in the form of power spectra.

4.1.8.2. Equipments needed: Since fluctuations are in microvoltage, a highly sensitive equipment is necessary.

1. Solartron 7055
2. Digital voltmeter

3. Data Logger (Potential time recorder)
4. Hewlett Packard HP-85 Micro computer or an equivalent unit "Solartron 1200 Signal Processor" which combines the above 4 individual units.

Address of supliers

Solartron 7055

Solartron Instruments,
Victoria Road, Farrburrough,
Hampshire, GU147, PW, England

HCL Ltd., Instruments Division,
41, Deepak Building, 13, Nehru Place,
New Delhi-110019

Digital Voltmeter

Hindustan Instruments Ltd.,
704, Vishal Bhavan,
95, Nehru Place, New Delhi-110019

MECO Instruments (P) Ltd.,
Bharat Industrial Estate,
T.J. Road, Sevree,
Bombay-400015

4.1.8.3. Technique: A schematic of experimental set-up used in the investigation of noise studies is shown in Fig 4.18. Noise source is located within the probable corroding area. A time record of sufficient interval is monitored over the frequency range (10μHz to 1Hz). Noise data as a record of potential fluctuation is obtained. Noise signal is transformed from time domain to frequency domain displaying in the form of amplitude and frequency based on either Fast Fourier Transform or Maximum Entropy Method of Spectral Analysis[3]. The measurement interval is usually between 2-10 seconds depending upon the frequency range applied to the specimen.

4.1.8.4. Laboratory investigations: Electrochemical potential noise measurements was applied on concrete specimen for assessing the corrosion of rebar in concrete[2]. Studies revealed that noise amplitude increases with decreasing frequency for uncontaminated concrete. Noise measurements

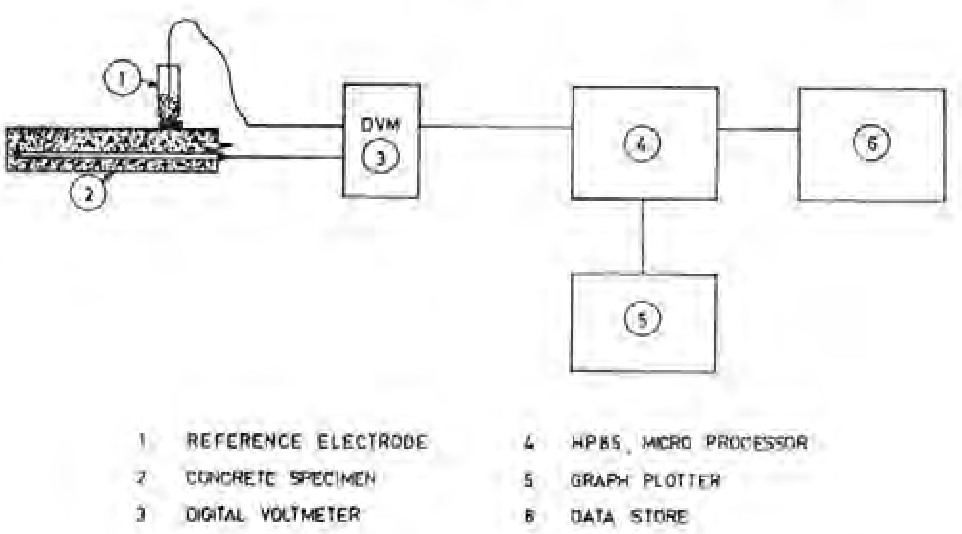

Fig. 4.18. Block Diagram of Noise Measuring Instrumentation

made on steel embedded in concrete contaminated with chloride revealed that series of fluctuations are obtained due to instantaneous potential drop which indicates that the system is undergoing pitting type of corrosion.

It has been reported that the exact nature of relationship between noise spectra of potential time records is still uncertain[4].

Corrosion potential noise measurements have been made on reinforced concrete structure to understand the behaviour of rebar embedded in concrete. The measurements are restricted to specific area of interest as previously specified by isopotential mapping. This technique is useful for identifying areas where active corrosion of pitting type is occurring[5]. From the spectral density plots, one can know the role of frequency. Then time constant of the noise transient is obtained from the slope of the plots indicating whether the system is under pitting (-10 to 20 dB per decade or undergoing general corrosion (-30 to -40dB per decade). Hence, this technique has been used to assess the level of the corrosion activity of the steel in concrete under laboratory conditions.

Polarisation resistance noise method which combined the effect of potential and current was employed in concrete beams/ blocks. Studies revealed that this technique cannot be used for large reinforced concrete structures as reinforcements are placed together. Refined work on the above technique revealed the usage of external polarising electrode for assessing corrosion rate. Studies showed that current mapping method combined with electrochemical noise enables in obtaining isopotential mapping by using external platinum electrode as source of current to polarize the steel rebar. The platinum electrode is maintained wet to ensure adequate conductance path to concrete. Mean current flow and current fluctuation identifies the area of corrosion and indicates level of corrosion activity including film breakdown and possibility of pitting type of corrosion. This technique is analogus to potential noise method.

4.1.8.5. Field investigations

(A) Application to Reinforced Concrete Structures

The usefulness and feasibility of this technique in monitoring the instantaneous corrosion rate for in-situ application on reinforced concrete structure has been investigated by some workers[2]. Noise measurements were made for assessing the condition of reinforced concrete swimming pools. This technique was also used on the retaining walls of a training pool and monitoring was done in areas where cracking had occurred and also in areas where calcareous deposits has appeared on the concrete surface. Measurement intervals usually were between 2-10 seconds. Noise data obtained on swimming pool wall was higher than those found in chloride contaminated concrete. Corrosion rate observed on-site coincided with those obtained in laboratory conditions.

(B) Application to Prestressed Concrete Structures

Electrical noise technique was applied to galvanized prestressed wires grouted in a retaining wall[2]. Noise data indicated that the attack was localized which may be due to breakdown of galvanized layer. This observation was consistent with the pitting attack visually examined.

4.1.8.6. Limitations: Since fluctuations are in microvolt range, a highly sensitive equipment is necessary.

4.1.8.7. Conclusion: Electrochemical noise monitoring is an emerging technique. It needs further development so as to satisfactorily relate the data to corrosion rates. The feasibility of using this technique for on-site inspection is to be established.

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4.2. Corrosion Monitoring of Prestressing Steel in Post-Tensioned Concrete

4.2.1. Corrosion monitoring of prestressing steel by electrical resistance technique: A critical examination of the available literature information shows that no fool proof non-destructive technique is available to monitor the condition of prestressing steel wires in prestressed concrete bridges. Attempts were made by CECRI to quantitatively assess the corrosion damage in the prestressing cable by making direct electrical resistance measurements. In this technique, the ends of the prestressing cables are made accessible for giving electrical connection.

The condition survey carried out on a few prestressed concrete bridges by the Central Electrochemical Research Institute has revealed that the prestressing steel has undergone severe rusting. In some cases, wires were found to have snapped. An electrical resistance technique has been adopted to assess the condition of prestressing steel with satisfactory results. Theoretical electrical resistance was calculated and actual values measured using this method was compared with theoretical values.

4.2.1.1. Principle: The electrical resistance 'R' of a single prestressing wire of length 'L' is given by the equation,

$$R=\rho\ l/A \qquad \text{... Eqn. 1}$$

Where, A is the area of cross-section of single-wire

ρ is the specific resistivity of prestressing steel

When the cross-section area 'A' reduces due to corrosion, resistance value 'R' increases. The resistance value measured can be compared with initial resistance which is calculated with the known initial diameter. By periodic measurements of resistance values, corrosion rate is calculated.

A typical prestressing cable consists of 12 wires of equal diameter and the wires through stressing operation are short-circuited at the anchorage ends. The wires can, therefore, be considered as parallelly connected. Theoretical electrical resistance of a cable R_T is given by,

$$R_T = R/12 \qquad \text{... Eqn. 2}$$

While calculating R_T, the possible short circuiting effect due to cables touching each other, cable sheath cross-section, etc. are also to be considered wherever necessary. Percentage reduction due to corrosion can be arrived at after actually measuring the resistance (R_m)

$$\text{per cent reduction in diameter} = 100\,(1 - \sqrt{R_t/R_m}) \qquad \text{... Eqn. 3}$$

4.2.1.2. Instrument: CECRI, Karaikudi has developed a portable battery powered instrument specifically for cable resistance measurement[1]. The instrument consists of a constant current source, an amplifier and a L C display.

4.2.1.3. Technique: The above said instrument is used for cable resistance measurements. The system uses four probe principle. It is a digital, mains operated, micro ohm meter working on this principle. A constant direct current of the order of 100 milli amps is impressed through current terminals of the meter and a potential drop across the ends of the cable is sensed, amplified and displayed resistace in milli ohms. In this method, contact resistance is eliminated and the resistance of the connecting wires are not included. Alternatively, any suitable resistance meter can be used.

4.2.1.4. Laboratory investigations

Applications to prestressed concrete structures

Work carried out by CECRI

In order to verify the usefulness of the technique, laboratory simulation studies were carried out at CECRI, Karaikudi. A bunch of 12 prestressing wires was subjected to corrosion in acidified chloride solution. Electrical resistance was monitored periodically. Actual diameter of each individual wire at a number of places was also measured using vernier caliper. Studies showed that inspite of wide variation in the reduction in diameter of individual wires due to corrosion, there is good correlation between the average reduction and measured resistance R_m, Fig. 4.19.

4.2.1.5. Field applications

Application to prestressed concrete structures

Work carried out by CECRI

This technique has been used with fair amount of satisfactory results in some prestressed concrete bridges in our country.

In bridge 'A' which was a new bridge under construction, the cable under measurement should have a theoretical resistance value R_T of 14.2 milli ohm. The measured resistance 'R_m' which was measured in that grider just after prestressing was found to be 14.7 milli ohm indicating that 'R_m' in a new grider is in good agreement with 'R_T'.

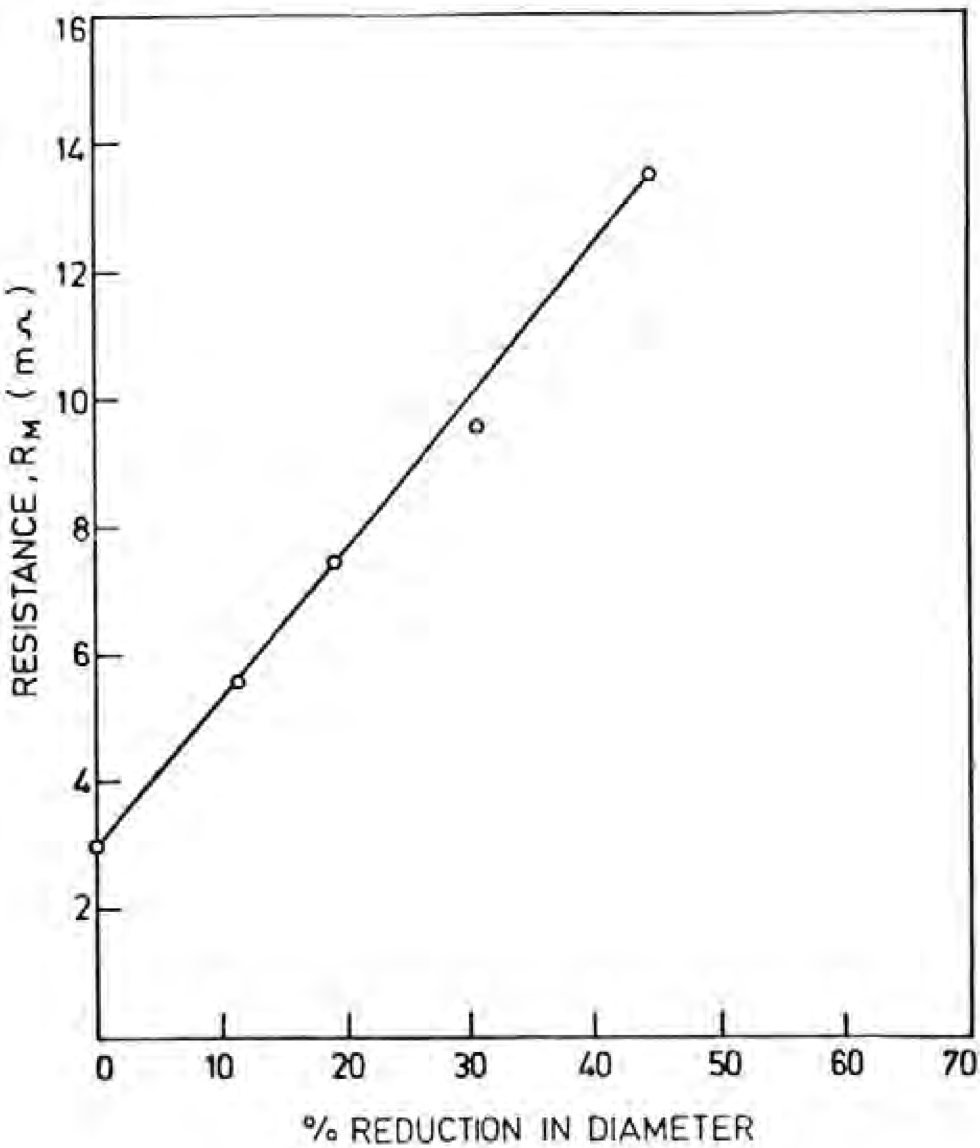

Fig. 4.19. Per cent Reduction in Diameter vs. Resistance (Rm)

In bridge 'B' which was worst affected, cable 1 with a R_T value of 33.9 milli ohms showed a R_m value of 43 milli ohm after 16 years. The actual diameter of the cable was measured to be 6 mm at a visible location instead of 7 mm. Similarly, cable 2 with a R_T value of 35.8 milli ohms showed a R_m value of 70 milli ohms after 16 years of construction and the actual diameter of this cable was measured to be 4.7 mm at a visible location instead of 7 mm.

Table 4.3 illustrates the progress of corrosion in cables with time measured in bridge 'C'. It can be clearly inferred that over a period of 4 years (from 11th year to 15th year), corrosion has progressed to an extent of about 7 to 10 per cent reduction in diameter, which is quite appreciable.

Table 4.3. Progress of Corrosion in Cables with Time Measured

Cable No.	Resistance (mill ohms)		Reduction in Dia	Age
	R_T	R_m	per cent	(years)
1	3.7	7.8	31	11
1	3.7	10.3	40	15
2	3.7	8.8	35	11
4	3.7	12.0	45	15
3	3.7	13.5	50	11
3	3.4	18.4	57	15

Quantitative assessment of corrosion damage in the prestressing cables of prestressed girders was made on a Bridge structure, in Maharashtra, using this technique[2]. The ends of the prestressing cables were exposed for giving electrical connection. Percentage reduction in diameter of individual cables which refers to the general extent of corrosion in cables and sheaths was worked out. It was found that there was a correlation between percentage reduction in values obtained from actual dia. measurements.

Electrical resistance of few cables was measured at different time intervals to get some idea about progress of corrosion in the prestressing wires with time. It was found that electrical resistance generally increased with time indicating progress of corrosion. In some cables, decrease in resistance values was also obtained. This may be due to short circuiting between adjacent cables due to snapping of wires. Reduction in diameter varied from 25 to 42 per cent in different spans of the bridge structure. Percentage corrosion varied between 6 to 76 per cent.

Electrical resistance measurements indicated the discontinuity in prestressing cables[3]. Estimation of reinforcement corrosion was also assessed by measuring the reduction in diameter due to corrosion in prestressing cables pier 'A' of a major bridge structure in India. Actual percentage reduction in diameter of cables ranged from 7 to 50 per cent. Survey made on the eastern side of the cable of pier 'B' revealed that reduction in diameter ranged from 0.9 per cent, whereas, cables at other end were found to be in tact. Electrical resistance measurements showed that the average percentage reduction in diameter of the prestressing wires due to corrosion works out to be about 15 per cent.

Conditions of prestressing cables in five different spans were ascertained by this technique where ends were made accessible for electrical connection[4]. Data revealed that percentage reduction in diameters of prestressing wires varied from 9 to 66 per cent. Measurements indicated out of 22 cases, eight cases indicated considerable reduction in diameter. In three positions, resistance value was greater than 2000 milli ohms which indicated discontinuity probably due to snapping of wires.

4.2.1.6. Limitations: Eventhough the technique looks very simple, lot of care should be taken while making the measurements. Cable ends are to be made accessible for making electrical contacts. In the existing bridges, this may be possible only in those cables which are anchored at the deck. Cable ends are to be cleaned thoroughly to ensure good electrical contact. Interpretation of the results is to be done cautiously because of unknown factors involved. Conducting corrosion products, such as, sulfides do carry appreciable current and affect the measurements. Pits and perforations produce erroneously high corrosion rates. Whole calculations are based on the assumption that the wires have undergone uniform corrosion throughout their length. If the corrosion is highly localised then this technique will not be able to indicate the same. In spite of all these limitations and uncertainty factors, his technique can still provide some information about the condition of prestressing steel. Instead of one time measurement, it is desirable to periodically take this measurement. An increase in resistance with time is indicative of corrosion in progress. It is to be emphasized that since it is not possible to identify and isolate the individual wires and all the wires and cable sheath may be short-circuited, the measured resistance values represent the overall condition of the prestressing system.

4.2.1.7. Conclusion: Electrical resistance technique, based on field survey appears to be an useful approach. If the uncertainty factors associated with this technique could be properly understood and eliminated by carrying out laboratory simulation studies, interpretation of field data can be made more precise and in that case these technique can become quite promising.

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3. Unpublished Report, CECRI, 1986

4. Unpublished Report, CECRI, 1988

4.3. Direct Examination of Cement Grout and Prestressing Steel

4.3.1. Radiography: Radiography technique is one of the non-destructive methods of testing concrete for obtaining information about the location, concrete quality and area of steel in reinforced concrete structures. Use of radioactive isotopes for concrete testing have been employed in ϒ-radiography studies[1,2]. Radiography technique is reported to be a reliable method of locating internal cracks, voids and variation in density of concrete.

Classification of Radiographic Mehods is shown below:

Radiographic methods

X-rays **ϒ-rays**

Application of radiographic methods using X-ray and ϒ-ray radiation for testing of concrete in detection of flaws, reinforcement location, etc. have been comparatively of recent origin for in-situ measurements. X-rays and ϒ-rays are invisible electromagnetic radiation which can penetrate concrete. They travel in straight lines. Rays attenuate depending on nature, density and thickness of concrete.

4.3.1.1. Principle: The principle of radioscopy is that the emission of Photons by the radiation generator is transformed in visible light by a fluorometallic converter for attaining maximum energy. By placing the radiation source on one side of the concrete and a photographic plate (recorder), on the other side, it is possible to determine the density or thickness of concrete provided one factor is known. Photograph of the concrete is produced from which reinforcements, cracks, voids, etc. are identified.

4.3.1.2. Equipments needed: Radiographic linear accelerator, Area Monitors/Survey Meter, Zone Monitors, Fluorometallic Converter, Mirror, Camera, Memory box, Video monitor, Image reproducer and Personnel Monitoring Badges.

Linear Accelerator

Linear accelerator emits X-rays with peak energy level of 8 MeV which are more penetrative than 1.17-1.33 MeV radiations from cobalt sources[4].

Advantages claimed in using Linac when compared to other sources are:

1. Linac Mini Accelerator of even 4 MeV allows radiology of concrete bridge of thickness upto 1m[4].
2. Total radiation (scattered and leakage) would not exceed 0.21 per cent of main beam of the source.
3. Greater safety as radiation is stopped when electric supply is cut.
4. Better picture quality.
5. Source always have same energy and same flow. In the case of radioactive sources, Activity decreases very rapidly.

Area Monitoring Instruments

Area monitoring instruments are essential for periodic monitoring of radiation emitted. The range of the meter should be 0.50 R/hr or 0-200 R/hr. (approx).

Zone Monitors

Zone monitor is used for continuous monitoring of radiation at the entry point during the exposure time. Range of the monitor is usually 0-200 mR/hr.

Fluorometallic Converter

Emerging radiation is transformed in visible light using this converter. The composition of this converter varies for the energy range used.

Camera

The image formed is taken up in real times by the Camera.

Video Monitor

The observation of transmitted image alongwith recording is done using this monitor system.

Reproducer

Areas of interest indicating voids, cracks, etc. are obtained in the photographic paper.

Personnel Monitoring Badges

Film/Thermo Luminiscence Dose meters badges (TLD) supplied by Division of Radiological Protection, Bombay are essential for monitoring the radiation doses received by the operators working in this technique.

Address of suppliers

Gamma Radiography Equipment

Rich Siefert & Co.,
Rontgen Work,
Bogenstr 41, 2070 Ahrensburg,
(Germany)

The Scientific Instrument Co. Ltd.,
480/3 Khivraj Complex,
Nandanam,
Madras 600 035

Scorpian System

Centre for the Study of Techniques of Equipment
CETE, Normanide Centre, LRPC, Blois, (FRANCE)

Linear Accelerator

Radiation Dynamics Ltd.,
U.K.

Siemens India Ltd.,
Worli, Bombay

Survey Meter and Zone Meter

Electronics Corporation of India Ltd.,
Instrumentation Group, Cherlapalli, Hyderabad-500 762

Personnel Monitoring Badges

The Officer-in-charge,
TLD Personnel Monitoring Services,
Division of Radiological Protection,
BARC, Bombay-400 085

4.3.1.3. Technique: The radiation source is kept on one side of the concrete and a photographic plate on the other side. The schematic and methodology diagram of this technique are shown in Figs. 4.20 and 4.21. The radographic linear accelerator delivers a dose rate of 1500 rad/min at 1m from the target in the main beam. Maximum energy leakage occurs at an angle of 105^0 to the main radiation beam. Studies showed that maximum scattered radiation from bridge road surface occurs at 180^0 to main beam direction[4]. The total scattered and leakage radiation should not exceed 0.21 per cent of the main beam at 1m from the assembly. Photogrammetric data analysis should be made and examination regarding location, size of reinforcements are made to know the condition of concrete for evaluating the extent of damage occurred.

4.3.1.4. Laboratory investigations

(A) Application to Reinforced Concrete Specimen

It has been reported that the application of Gamma Radiometry has been confined to laboratory conditions[5]. It measures the density of freshly placed concrete. The principle of operation is same as that of Υ radiography except the positioning of source and receiver which are held side by side perpendicular to concrete surface. Radiation is attenuated when passing through the concrete and scattered in all directions. The receiver detects the radiation and is then related to density of the concrete by empirical means.

Studies showed the possibility of following the evolution of corrosion products by X-ray radiography technique on rebar embedded in concrete[6]. Electrochemical corrosion of rebars was accelerated by impressing anodic charge of +850 mV. Concrete specimen were exposed in 3.5 per cent NaCl solution. Measurements of corrosion products were taken by passing X-rays of 0.01 nm wave length. Radiographs were taken on six specimen at interval of 5 days upto 55 days of exposure. It was found that concrete cracking under these conditions occurred after 27 days of exposure after the formation of 1000 gm/m^2 of corrosion products. Radiography techique was employed in concrete blocks of size 10 x 22.5 x 30 cm for understanding the corrosion of rebars in concrete blocks[7]. The condition of 3 bars was, such as, to simulate no corrosion, slight corrosion and severe pitting corrosion. Radiograph profile of the 3rd bar indicated the presence of pitting type

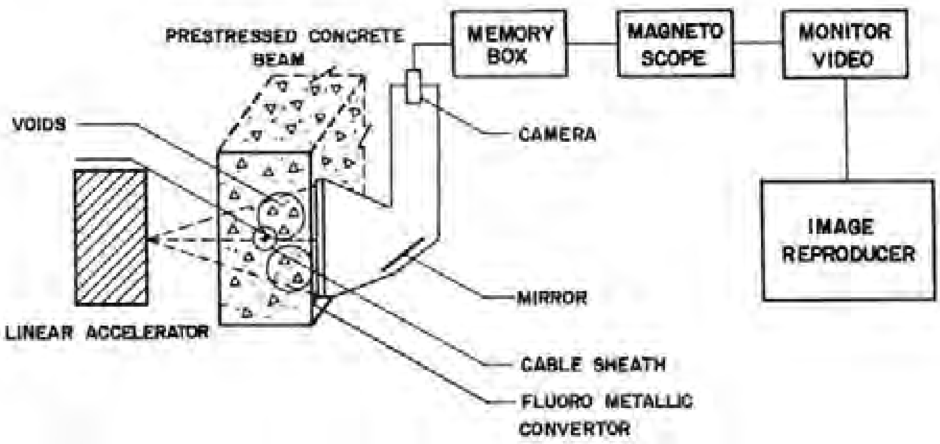

Fig. 4.20. Schematic Diagram of Radiography Technique

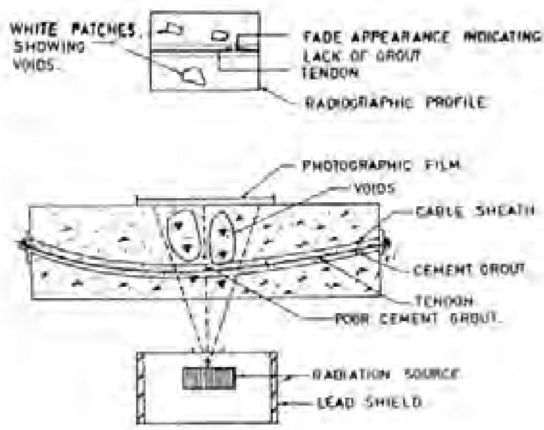

Fig. 4.21. Front Elevation of a Typical Prestressed Concrete Beam showing the Radiographic Methodology

of corrosion. The honey combing present in concrete specimen was distinguished as white patches on radiograph reproduced.

Radiography technique was employed in locating the position of steel reinforcing bars in two concrete slabs[8]. 100 millicurie radioactive source was used for the study. Slabs have been cut open and measurements have shown that rebars were precisely in the positions indicated by radiography work.

γ-ray radiation was used for determining the thickness of concrete slabs of known density[9].

Caesium 137 of 5 mC; was used as the source. The radiation through the slab was measured by a scintillation counter placed on the other side of slab. The Detector is moved until maximum count rate is obtained.

In 1966, Preiss[10] has calibrated a curve finding a correlation between count rate reading and slab thickness. If steel rebar is met with, count reading is reduced as steel is denser than concrete and hence numerous readings should be taken and highest reading is taken for determination of slab thickness.

It has been reported that internal structure of concrete and development of micro-cracking were studied by some workers using X-ray radiography. X-ray radiography was used for detecting the amount of corrosion (not the rate) of rebar in concrete containing various amounts of chloride ions[11]. Density of X-ray film decreased with increasing corrosion of steel.

Some workers have reported the assessment criteria for ϒ-ray Back Scatter test for quantifying the concrete quality[12].

Back Scatter Reading (mm steel)	Density	Concrete quality
8.5	> 2300	Good
7.5-8.5	2000-2300	Below average
7.5	< 2000	Poor

(B) Application to Prestressed Concrete Specimen

Literature survey revealed that damage to grouted cables in which ducts were partly grouted or where grouting materials remain in fluid conditions could be identified using Radiographic analysis[13].

Radiographic examination of the ends of prestressed beam demonstrated the possibility of identifying the type of reinforcements[7]. The reinforcing bars and prestressing wires were identified and light shading (fade colour) obtained below the prestressing wires showed lack of grouting in the particular area of concrete beam. Radiography technique using ϒ-rays was used for identifying the presence of honeycombing, cracking and for understanding the performance of injected grout in prestressed concrete structures.

Imperfection, like, air and water pockets entrapped in grouts in the ducts containing prestressing wires was examined using this technique[14]. Extent of voids in grouting present in the ducts was also investigated.

Literature survey showed that X-ray radiographic examination was carried out for studying the bond stress existing in prestressed beams[15]. Measurement of strains in prestressed wires without disturbing the concrete was done using this technique. Small lead markers were placed in slots in the reinforicng steel while casting concrete. Positions of markers before and after straining was recorded using X-ray photography.

4.3.1.5. Field applications

(A) Application to Reinforced Concrete Structures

Investigation of Swaythling Bridge, Southampton was carried out using Radiography technique[4]. Adequate exposure geometry was obtained when the source film distance (SFD) was twice the object film distance. Defects, such as, voids, cracks, position of rods, were identified using this technique. Exposure time for radiation was 28 seconds for a concrete beam of 870 mm in depth. Eight vertical exposures using Linac on the bridge deck was made. Radiographic examinations were also carried out on number of concrete beams of thickness ranging from 610 to 1575 mm. Five horizontal exposures under the bridge deck through the support beam was made using 10 Ci Iridium 192; or 2 Ci Cobalt 60 source for which the exposure time was 15 minutes or 12 minutes per exposure.

Reinforcement corrosion was detected using X-ray and γ-ray radiations when the thickness of corrosion was greater than 0.2 mm[16].

(B) Application to Prestressed Concrete Structures

Quality of grouting in prestressed concrete structures has been verified in France since 1968 by Υ-radiography technique. From 1985 onwards, examination was carried out using LRPC of BLOIS with miniature Linear Accelerator[4].

Radioscopic examination of prestressed concrete bridges has been done using 'SCORPION' system through a linear mini accelerator of 4 MeV which allows radiology of concrete bridges of thickness upto 1 metre. Better picture quality, greater safety are the added advantages claimed in using linear mini accelerator when compared to cobalt 60 sources.

According to FIP, radiography technique using X-ray or Υ-ray was used for checking the location and condition of prestressing tendons for errors in positioning, steel failure and fractures, corrosion or lack bond[17]. This technique was also used for checking the quality of grouting in prestressed concrete works.

4.3.1.6. Safety measures: Proper safety measures are to be taken for the protection of operators from dangers of exposure to radioactivity.

The radiation safety procedures with regard to Radiography technique are as follows:

1. The road is to be closed for vehicular traffic and road barriers are to be erected.
2. Lead shielding walls should be placed between linac and public house.
3. Flashing Amber lights should be erected 90 seconds prior to commencing of operation.
4. Quartz fibre electroscopes must be worn by all operators.
5. Total absence of public movements should be ensured.

4.3.1.7. Limitations: Following are the limitations with regard to application of radiography technique:

1. Radiations being dangerous, extra precautionary measures are to be taken.

2. Two opposite faces of the structure must be accessible.

3. Radiography technique can be used to examine concrete upto about 450 mm thick beyond which exposure time becomes more[5].

4.3.1.8. Conclusion: Inspection by radiogrphy technique using LINAC Source was found to be useful in assessing the quality of grouting in prestressed concrete bridges as it delivers maximum energy with minimum losses when compared to other radioactive sources used in Y-radiography.

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4.3.2. Endoscopic technique

4.3.2.1. Introduction: Close visual inspection of the critical internal surfaces of tubes, pressure and other large vessels and structures can now be made faster and more economically by using endoscopes. Endoscopes are designed specifically for inspection operations where the instrument itself must be very long in order to reach the areas which need to be surveyed and where high levels of illuminations are also required. This optical technique has been tried in post-tensioned prestressed concrete structures to check the condition of prestressing tendons as well as the grouted duct. This is a semi-destructive type of inspection*.

4.3.2.2. Principle: Endoscope is a tube containing a series of optical lenses. By means of these lenses it is possible to redirect and refocus beam of light on the object to be examined. Now-a-days instead of optical lenses, optical fibres fabricated from glass or plastic are used, with sizeable resolution[1]. However, in the case of prestressed concrete structures holes are to be drilled through the concrete cover to insert the endoscope and reach the cable duct.

4.3.2.3. Equipments needed: Endoscope, extendable tubes with various length, generator or rechargeable battery pack, flexible fibre glass cables, tungsten filament lamp or quartz halogen lamp.

Address of supplier

Henke-Sass Wolf Ltd.,
4, Tannery Yard, Burford, Oxfordshire,
OX8 0DW, (UK), Telex: 0993b 82 2613/4

4.3.2.4. Technique: Simplest arrangement to carry out this inspection is shown in Fig. 4.22. For carrying out visual inspection in bridge structures, critical places, such as, change in section of the duct and similar special points of the duct are to be selected. The selected places of the observation should not be close to other cables and other non-prestressing reinforcement. In case of doubt, observations can be made by means of the endoscope during the drillings. Drillings of size 20 mm in diameter are made for endoscopes with a diameter of 8 mm. The depth of the drillings should not exceed the length of the endoscope. The length of endoscope system can be increased by the addition of extension tubes available in lengths of 0.5, 0.75, 1 and 1.5 metres. All sections are connected with each other by means of a positive, square section thread which inhibits cross threading and promotes fast, easy assembly at site. Now-a-days with a new range of endoscopes it is possible to observe the object from a distance of 30 metres from the eyepiece.

* In India, endoscopy technique has been used on bridges across river Narmada in Gujarat on NH-8 and Zuari in Goa on NH-17.

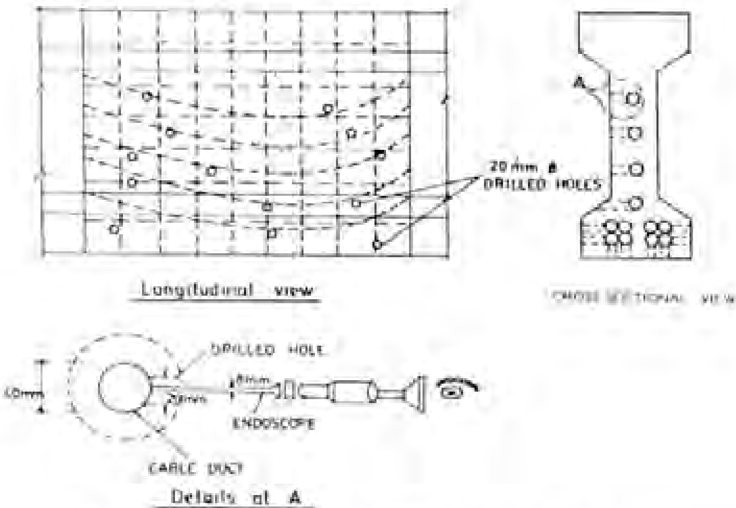

Fig. 4.22. Endoscopic Examination of Cable Ducts

Now-a-days quartz halogen lamps are built into the tip of the endoscopes to provide very high levels of illumination necessary for operation under arduous conditions. Having established a drilling hole to the cable duct the visual inspection of the grout, extent of corrosion in tendons, etc., is carried out by direct observation. To suit the variety of applications, five different types of objective heads and eyepieces are available. By changing the objective head, the angle of view can be adjusted to 0°, 45°, 90°, 110° and 0°-wide angle. There is also a choice of five different eyepieces - monocular, binocular, 45°, 90° and variable magnification zoom. Photographs can be made with the suitable camera at typical locations.

Estimation of ungrouted duct length

It is possible to estimate the ungrouted duct length, knowing the geometry of the tendon and the duct. By using either the law of Boyle-Mariotte or by means of a flow-through gas meter, the volume of the duct can be estimated. From this, length of the cavity can be made. After these measurements, the necessary regrouting can be carried out by suitable vacuum pump and cement mortar container connected with hoses.

4.3.2.5. Field applications: Philip Mohr has performed this technique on Sallingsund prestressed concrete bridge in Denmark[2]. He used this technique to find the ungrouted duct length. The inspection was carried out by cutting a hole of size 15 x 30 cm into the tendon using effective site system. Regrouting operation was carried out successfully. From the studies he concluded that the re-establishment of the drilled holes including securing of water proofing was unsatisfactory.

4.3.2.6. Limitations: Holes are to be drilled through the concrete at several places and hole drilling is a specialised job. Drilling should be carried out carefully so as to avoid cutting non-prestressing steels. Inspection is possible only at drilled locations and hence, the entire cable profile can't be covered by this technique. Quantification of corrosion is not possible.

4.3.2.7 Conclusion: Endoscopic technique, if properly applied, can be quite useful in identifying ungrouted portions and regrouting them. However, this technique will not be useful to quantify the corrosion damage.

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4.4. Detection of Defects in Concrete

4.4.1. Ultrasonic pulse velocity technique: The ultrasonic pulse velocity technique is a non-destructive method of testing concrete. It is used for,

(a) Assessing the homogenity of concrete

(b) Determining the location of possible defects, such as, foreign substances, cracks (internal) and areas of low density

(c) Determining the thickness of concrete members where there is access to only one face

(d) Determining dynamic modulus of elasticity

(e) Assessing concrete strength.

This can be used on both precast and cast in-situ elements, and is essentially comparative. It gives the difference in properties of two apparently similar pieces of concrete, or changes in properties that a piece of concrete has undergone[1].

4.4.1.1. Principle: Sound energy above the audible frequency of 16,000 Hz is designated as ultrasonics. It is a form of mechanical energy and propagates through the material as stress waves by direct and intimate mass contacts without any bodily movement of the material. Propagation of these waves through the material is controlled by elastic properties of the material[2]. These ultrasonic waves also termed as 'pulses' are generated by combining mechanicial vibration with sound waves of same frequency of vibration. Pulses are emitted by a transducer are transmitted through the material and received by another transducer which is located at a distance of 'L' from the transmitting transducer. The transit time 'T' in microseconds of the first pulse arriving at the receiver is precisely measured by electronic means. From these physical parameters pulse velocity can be calculated as follows:

Pulse velocity = L/T

Ultrasonic measurement on concrete structures with two accessible sides in which a transducer is mounted on each side is a well-established technique[3].

4.4.1.2. Equipments needed: PUNDIT (Portable Ultrasonic digital indicating tester) (or)

'V' - meter

Address of supplier

Ultra Image International
Two Show's Cove, Suite 101, New London CT 06320(203)442-0100
Telex: 643412; Fax : 203 442-2389

4.4.1.3. Technique: Schematic diagram of pulse velocity testing circuit on concrete is shown in Fig. 4.23. The transducers are placed over concrete surface with an appropriate coupling agents, such as oil, water, grease or other viscous materials. These coupling agents enable to avoid air between the contact surface of the diaphragm of the transducers and the surface of the concrete[4]. If the concrete surface is rough and uneven, it is necessary to smooth the level, by grinding the area at which the transducer is to be placed. There are three alternate arrangements for placing the transducers in ultrasonic testing.

(i) Direct Transmission: Transducers are placed on opposite concrete surfaces.

(ii) Semi-direct Transmission: Transducers are placed on adjacent concrete surfaces meeting at an angle of 90°.

(iii) Indirect or Surface Transmission: Transducers are placed on same concrete surface.

Steps Involved

(i) Pressing the faces of the transducers against the surfaces of the concrete, (ii) measuring the transit time, (iii) measuring the length of the shortest direct path between the centres of the diaphragms and (iv) calculating the pulse velocity.

In the case of tests through badly cracked or deteriorated concrete, variation of the results substantially increases to as large as 20 per cent. In such cases, calculated velocities will be sufficiently low as to indicate clearly the presence of distress in the concrete tested[4].

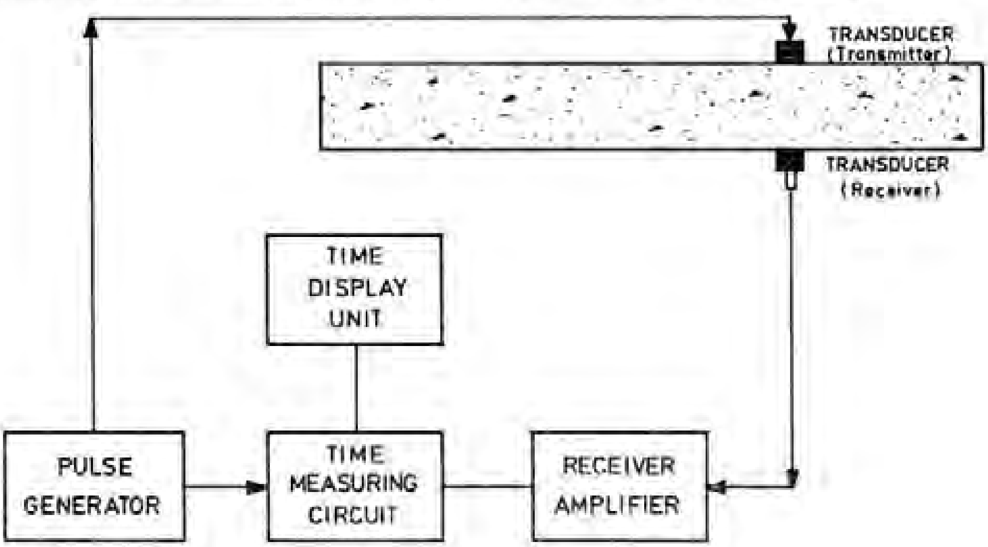

Fig. 4.23. Schematic Diagram of Pulse Velocity Testing Circuit

Now-a-days ultrasonic pulse echo technique is used in which one Piezoelectric transducer is being used and it acts as the transmitter and receiver[5]. But this technique is mostly applied to determine the thickness of concrete pavements where only one face is accessible.

Effect of Reinforcement Bars

Ultrasonic testing of a reinforced concrete member is affected by steel bars embedded in concrete[6]. It is found that the steel rebar appears to act together with the surrounding concrete in transmitting ultrasonic pulses. So, the effective pulse velocity (V_e) measured lies somewhere between the pulse velocities in the two separate media and varies with the diameter (d) of the bar and the pulse velocity in concrete (V_c). The effective pulse velocity has been derived as follows:

$$V_e = 5.90\text{-}10.4\ (5.90 - V_c)/d \text{ for } d \not< 10 \text{ mm}$$

4.4.1.4. Interpretation of results

(A) Histograms

Variations in the concrete within a member cause variations in pulse velocity. So, measurements of pulse velocity provide a means of studying the uniformity of the concrete. The test results in terms of pulse velocity may be plotted as a 'histogram'. Histogram gives a picture of the distribution of results. This is shown in Fig. 4.24. A narrow distribution indicates uniformity while a broad distribution indicates non-uniform quality. A narrow distribution with some exceptionally low velocity values indicate probable sign of honeycombing.

Alternatively, uniformity may be examined by the coefficient of variation of the results. For good concrete coefficient of variation should be less than 5 per cent. Coefficient of variation being more than this becomes a matter of concern[7].

(B) Contours

A crack of void lying between transducers prevents direct passage of ultrasonic pulses. The pulses reaching the receiving transducer is diffracted around the periphery of the void, thus the transit time is increased. The presence of voids or cracks can be detected by scanning the volume of concrete at various grid points. Orthogonal grid pattern is preferable and the results are plotted as contours. This is shown in Fig. 4.25. A rapid change of pulse velocity over a short distance as evidenced by the closeness of the pulse velocity contours, indicates the presence of void. The pulse velocities obtained in the void are substantially lower than the mean.

(C) Determination of Compressive Strength from Pulse Velocity Measurements

By using semi-empirical method a correlation has been established between compressive strength and pulse velocity[8]. The coefficient of correlation is 0.98 which indicates that the correlation is very significant.

Correlation analysis has led to the following empirical equation:

$$f_c' = 4.776\, e^{0.558\, V_p}$$

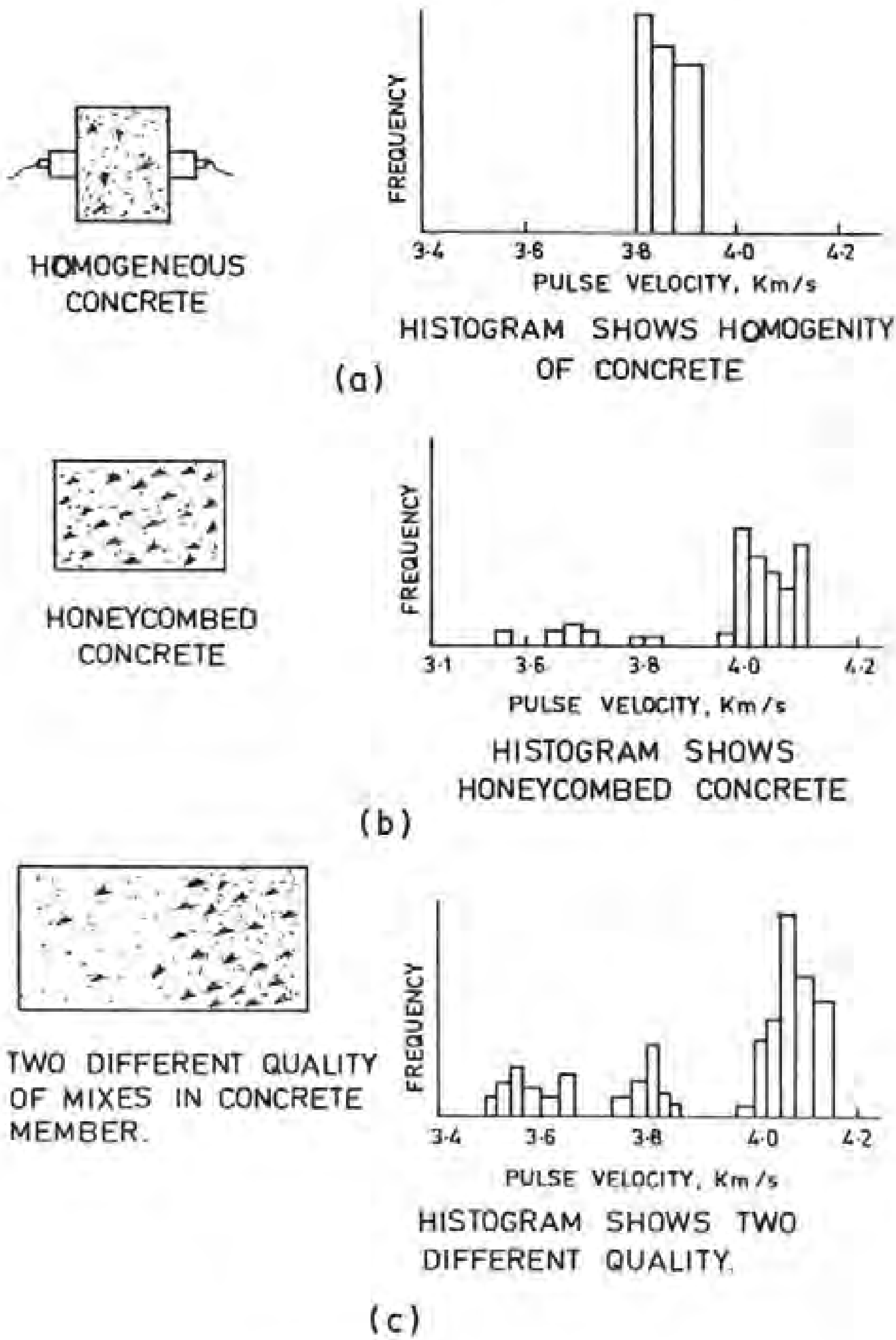

Fig. 4.24. Typical Histograms of Pulse Velocity

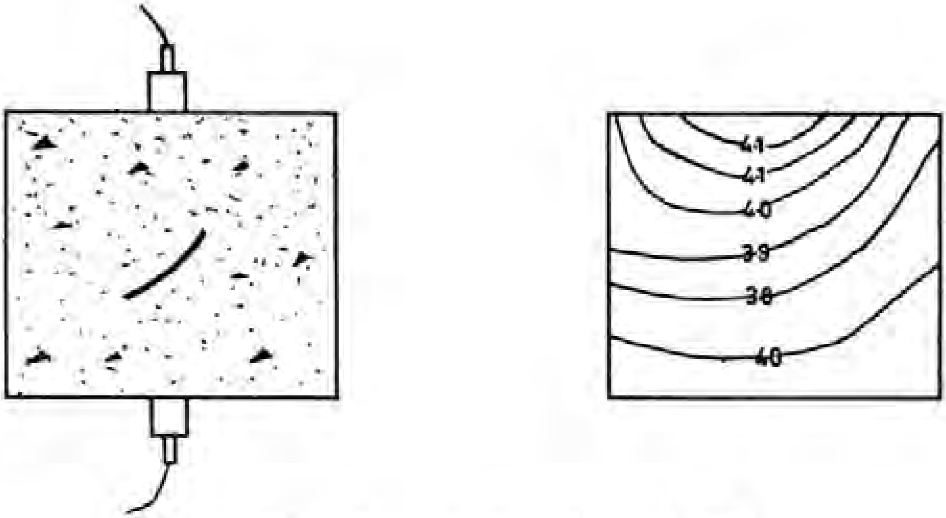

Fig. 4.25. Pulse Velocity Contours in Cracked Concrete

Where

V_p = pulse velocity measured in concrete
f'_c = compressive strength

(D) Estimation of Elastic Modulus from Pulse Velocity Measurements

The pulse velocity is related to the physical properties of the concrete by the equation[4].

$V^2 = (K)(E/D)$

K = a constant

E = Modulus of Elasticity

D = the density

4.4.1.5. Laboratory investigations: Initially, pulse velocity measurements were used to measure the thickness of the concrete pavement using two transducers [9,10]. By modifying the material present in the transducer the maximum thickness that can be measured is 178 mm. Claytor and Ellingson have measured the thickness of a refractory concrete slab of 300 mm using two 250 KHz pZd - 5A transducers[11].

Elvery and Din have performed UPV measurements on singly reinforced beams to arrive at a relation between pulse velocity and cube compressive strength[12]. From the studies a curve linear relationship was obtained.

Chung and Law also attempted this UPV measurements on concrete prisms of size 150x150x750 mm to arrive at a correlation between pulse velocity and compressive strength[7]. They also obtained curve linear relationship between them. The above results were also confirmed by Kaplan and Varghese[13,14].

Alexander and Thornton have observed that pitch catch measurements rather than pulse-echo should be made to gain full efficiency. Both transmitter and receiver be tuned electrically with inductors to improve signal to noise ratio (SNR) and sensitivity.

National Council for Cement and Building Materials carried out UPV measurements for monitoring sulphate resistance of concrete[16]. From the studies it was found that the 10 per cent decrease in pulse velocity, approximately correspond to about 20 per cent decrease in the dynamic modulus of concrete.

Chung and Law have observed that it is advisable to choose pulse path which avoid the influence of the reinforcement[6]. They derived an expression to find the zone of steel influence as follows:

$$a/l < 1/2 \sqrt{\frac{1-\vartheta}{1+\vartheta}}$$

a = prependicular distance from centre line of steel rebar to nearest edge of transducer

l = shortest distance between transducers

ϑ = pulse velocity ration = V_c/V_e

V_c = pulse velocity in concrete

V_e = effective pulse velocity in concrete-steel medium

If the pulse path inevitably falls within the zone of steel influence, the measured pulse velocity should be multiplied by a correction factor (K) given by:

$$K = \vartheta + 2\left(\frac{a}{l}\right)\sqrt{1-\vartheta^2}$$

Prakash Rao is performed UPV measurements on reinforced concrete blocks and found that the pulse velocity values are influenced by rebar present in the concrete[17]. He also conducted these tests in prestressed girder casted at laboratory, at compression as well as tension zone at loaded conditions. In tension zone, the pulse velocity decreased as the load was increased and fell suddenly due to formation and propagation of cracks. But in compression zone, the pulse velocity decreased slightly with load and thereafter remained constant. He also concluded from the studies that the UPV cannot be directly related to the strength of concrete without prior knowledge of its composition. This technique is more helpful in locating defects, such as, cracks, voids than in estimating precise strength levels.

Elvery has reviewed that the specification, such as, ASTM, British Standards should state the type of the test, the limiting values of the test, method of interpreting the results[18]. Because, so far the standards do not specify clearly the limits of condition of the test, such as, moisture content and temperature.

Mather has reported that the reproducibility of results obtained on uncracked concrete specimen using UPV technique was within one per cent[19]. The advantages of this technique over the resonance testing are (i) limitations regarding the size and type of test specimen are negligible and (ii) localized damages or localized differences caused in compaction can be easily determined.

4.4.1.6. Field Applications

(A) Applications to RCC Structures

Whitehurst has performed ultrasonic pulse velocity technique on 13 bridges, 14 dams and 15 highway pavements in-situ, to study the quality of the concrete[20]. From the studies it was found that sufficiently large number of measurements were required to arrive at conclusions without any error.

Jones has reported certain minimum pulse velocity for acceptance of concrete in prestressed and reinforced concrete works[21]. These are given in Table 4.4.

Table 4.4. Minimum Pulse Velocity for a Few Specific Structures

Type of work	Minimum value of pulse velocity for acceptance, km/sec.
Prestressed concrete - T sections	4.572
Prestressed concrete - Anchor units	4.359
Reinforced concrete - Framed building	4.115
Suspended floor slab	4.724

Leslie and Cheesman used this pulse velocity technique for periodic crack surveys of hydro-electric gravity dams[22]. In their investigation it was found that some cracks which were as deep as 61 cm initially became shallower and reduced to about 10 cm in depth after 5 years.

Jones has also applied this technique to measure the depth of cracks on concrete pavements[24]. A crack of full depth produced a large attenuation of the wave and hardly any signal was received across the crack and the differences in the oscillograph display resulting from superficial and full depth cracks could be immediately detected.

Chung and Law have performed a field test on two reinforced concrete deep beams[7]. Honeycombing was suspected at the region of lapping of reinforcing bars. A grid of measuring points was marked in the suspect region of each beam. The results were plotted as contours. The pulse velocities in one beam were fairly consistant, giving a mean value of 3.97 KM per sec. ± 0.08 KM/sec. Significantly, no low readings were obtained. The results are plotted as Contours and shown in Fig. 4.26. The contours do not show any particular pattern, indicating that no sizeable voids were detected in this region, whereas, in another beam where the pulse velocities were less consistant, significantly low readings were obtained. This is shown in Fig. 4.27. From this, it is suspected that voids or honeycombs occur in this region. The extent of honeycombing was subsequently confirmed when the defective concrete was chipped off.

Leslie, Cheesman and Whitehurst have conducted these measurements on large structures, in particular on concrete beams and found that there are considerable differences in the magnitudes of the pulse velocity propagated through very good and very poor concretes[20,22]. They have suggested a broad division based on the pulse velocity which is given in Table 4.5. This is confirmed later on by Brunarski[24]. He arrived at the same results and related the pulse velocity to the approximate compressive strength.

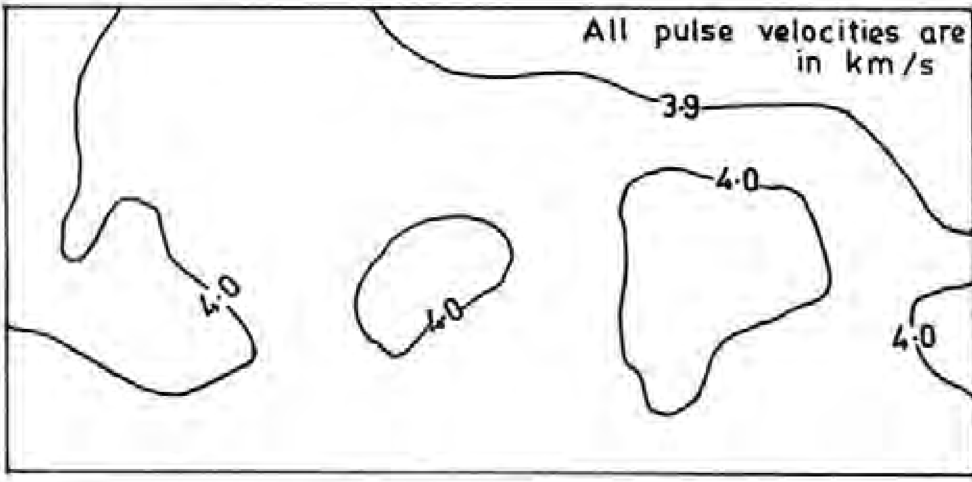

Fig. 4.26. Pulse Velocity Contours in SOFFIT of Uncracked Beam

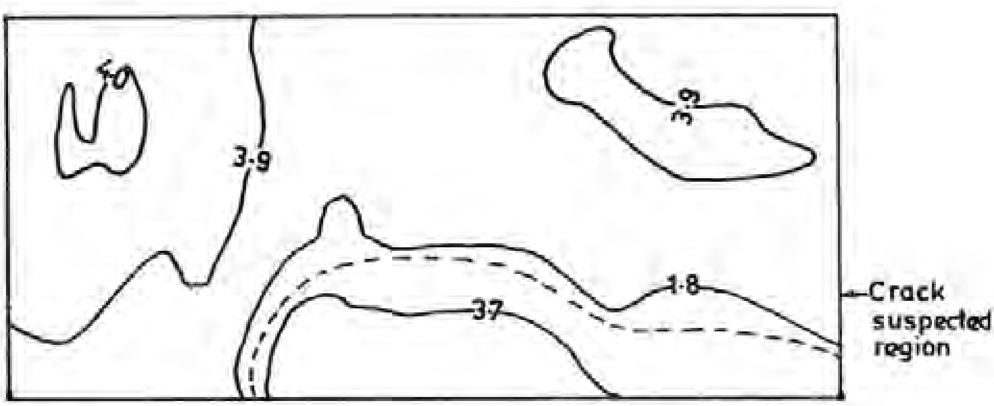

Fig. 4.27. Pulse Velocity Contours in SOFFIT of Cracked Beam

Core strength vs. Pulse velocity

Chung and Law also conducted pulse velocity measurements on concrete cores which were cut from the structural members[7]. The compressive strength of the core were determined destructively after pulse velocity measurements. The estimated compressive strength using empirical formulae, $f_c' = 4.776\, e^{0.558\, V_c}$ from pulse velocity value lies within 20 per cent of the measured compressive strength.

Tomsett has tried pulse velocity survey on the wall of sewage works where signs of 'bleeding' were evident[25]. Pulse velocity survey gave a co-efficient of variation of 5.7 per cent. This low variation indicates good workmanship. Air entrainment had been suggested to overcome the problem of 'bleeding'. An ultrasonic pulse velocity survey of a subsequent wall cast using air-entrained concrete showed a very much lower variation in pulse velocity than the original wall.

(B) Applications to Prestressed Concrete Structures

Tibor Javour used this pulse velocity survey for checking the quality of concrete in prestressed bridge construction[26]. The results were compared with the result measured by hammer pulse apparatus. Prestressing of deck slabs was permitted after an ultrasonic pulse velocity of 4.5 Km/s was recorded and this correspond to a strength of about 40 N/mm². For removal of formwork where the strength development for the whole span was to be checked, the hammer pulse apparatus was found more suited.

Table 4.5. Relationship between the Quality of the Concrete and the Ultrasonic Pulse Velocity

Quality of concrete	Longitudinal pulse velocity	Approximate compressive strength
	Km/sec.	N/mm²
Very poor	Below 2.0	—
Poor	2.0-3.0	4.0
Fairly good	3.0-3.5	Upto 10
Good	3.5-4.0	Upto 25
Very good	4.0-4.5	Upto 40
Excellent	Above 4.5	Above 40

Jones and Wettern fixed a pulse velocity of 4.511 Km/sec, which corresponds to a cube strength of 35.15 N/mm² when using ultrasonic testing for the acceptance of precast section of prestressed concrete beams[27].

Tomsett has reported that a precast, prestressed beam which showed a 11 per cent coefficient of variation in pulse velocity survey failed in routine load test[25].

4.4.1.7. Limitations: Pulse velocity measurements are affected by the cement type, nature and grading of aggregate, moisture content and ageing of the concrete structures. Sufficient number of readings are to be taken to obtain reliable results and haphazard interpretation of these may lead to wrong conclusions.

4.4.1.8. Conclusion: Ultrasonic pulse velocity technique if properly applied, can be quiet useful in identifying internal cracks, voids and homegenity of the concrete*.

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* Ultrasonic Pulse Velocity technique has been used in India for bridges across river Yamuna in Delhi on NH-24 and Narmada in Gujarat on NH-8.

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4.4.2. Ground penetrating radar technique: Ground penetrating radar is a remote sensing technique which has been applied to evaluate non-destructively the conditions of existing concrete bridge decks. This technique provides a rapid method of identifying thin and weakend areas in concrete structures. This technique also determines the position of reinforcing steel lying within the bridge deck.

Advantages claimed with this technique are:

* More area covered
* Faster data collection
* Identifies deteriorated areas in concrete structures
* Measures slab thickness
* Measures depth to rebar embedded in concrete
* Voids are also identified

4.4.2.1. Principle: High frequency electromagnetic pulses are sent into the concrete by a transducer and the reflected pulses are recorded graphically. The location of rebar and depth of voids from the concrete surface are identified.

4.4.2.2. Equipments needed: Main equipment used for Ground Penetrating Radar investigations on bridges is SIRSYSTEM - 8

(i) GPR DATA collecting mobile vehicle

(ii) Sirsystem '8' : This system consists of control unit, transducer, graphic chart recorder and magnetic tape recorder. This equipment operates on 12 Volts DC.

(iii) Transducer : Consists of radar transmitter, receiver and antenna. It is used with the system for transmitting electrographic energy with desired frequency into concrete and receive, and process the reflected energy for analysis.

SIR System 8

SIR System '8' is manufactured by Geophysical Survey System Inc., USA.

4.4.2.3. Technique: High frequency electromagnetic pulses are sent into the concrete by radar transducers. Transducers operating at lower frequencies will yield greater depth of penetration of radar signal while higher frequencies, although not able to penetrate deeply, gives greater resolution[1]. It has been reported that a brief pulse of 0.8 nano seconds long is directed into bridge deck. If the energy is encountered by an interface, a portion of energy is reflected back to transducer, processed in control unit placed in data collection van, amplified and time differential between initial transmission and reflected waves is determined. Taking into considerations of Dielectric characteristics of the concrete, time differential is converted into depth. The electromagnetic pulse is repeated at rate of 50 kHz (50×10^3 cycles/sec). The transducer is moved along the concrete surface which enables a automatic graphic recording identifying the locations of voids and rebars. Ground penetrating radar technique yields data along series of longitudinal lines over the bridge deck. Number of scans and distance between them depends on degree of coverage desired.

4.4.2.4. Field applications

(A) Application to Reinforced Concrete Structures

It has been reported that deteriorated areas of pavement of bridge deck showed distinctive strip chart and was characterized by scattering of energy[1]. Presence of numerous reflections obtained on recorded graph confirmed the deteriorated portion of the bridge deck. This technique was evaluated on overlaid bridge decks by State of Virginia and was found to be effective in locating concrete delaminations on overlaid covered decks[2].

(B) Application to Prestressed Concrete Structures

No literature information could be collected with regard to use of this technique in prestressed concrete structures.

4.4.2.5. Limitations: The application of this technique requires elaborate site arrangements. It can't indicate either corrosion or rate of corrosion of steel in concrete. Although, radar technique offers good radiation and precision, the enormous volume of data produced makes interpretation difficult[3].

4.4.2.6. Conclusion: This technique identifies the deteriorated portions persent in the concrete structures and also determines the position of rebar embedded in bridge decks. However, radar technique cannot be used for assessing the corrosion of reinforcements or tendons.

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4.4.3. Infrared thermography technique: Delaminations are usually examined by performing sounding techniques, like, dragging chain, hammers, iron rods, etc. Delaminated areas produce dull sound when struck compared to good concrete and is effective only on exposed concrete decks[1]. Delaminations occur when layers of concrete separates from bridge decks near the reinforcements. It has been reported that the main cause of formation of delaminations leading to potholes were due to corrosion of reinforcing steel due to ingress of chloride ions exposed in Marine Environments[2]. Infrared thermography technique can be used to identify delamination on exposed concrete decks and decks overlaid with asphalt. This technique identifies delaminations in bridge deck by observing temperature differential between delaminated and sound concrete under specific environmental conditions.

Advantages claimed with this technique are:

* Faster data collection
* More accurate results than conventional sounding procedures
* Less operation
* Portable and provides a permanent record

4.4.3.1. Principle: Measures infrared radiations and is used to determine surface temperature differentials of concrete structures during heating or cooling. Presence of thin air film at interface between bottom of delaminated area and top portion of deck surface acts as insulator causing delamination to absorb and reflect more radiant energy than sound portion of bridge deck as shown in Fig. 4.28. Hence, a temperature differential between delaminated and sound portion of the bridge deck is obtained.

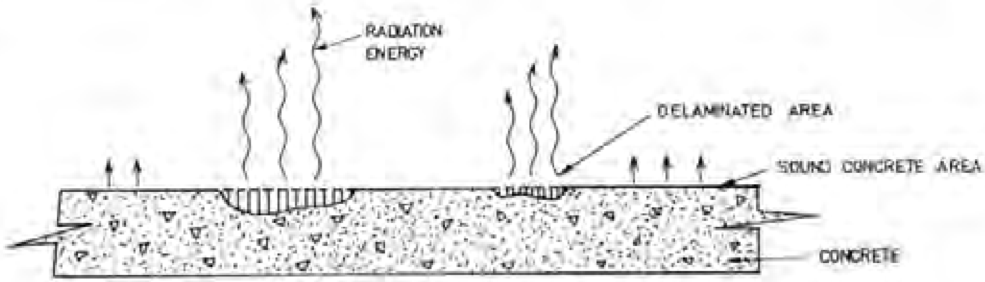

Fig. 4.28. Principle of Delamination Detection

4.4.3.2. Equipments needed: Infrared Data Collection Van consisting of

Infrared data recorder and

Infrared Camera

OR

Agatronic Thermovision 750 System with a 7° lens

Infrared Camera

Infrared Camera is mounted on a framework and raised to approx. 5.1m above the pavement of bridge Deck.

Infrared camera is used for detecting emitted thermal radiations. It produces video signal and records thermal imagery on video tape capable of measuring temperature differences as low as 0.9°F (0.5°C). Camera uses mercury cadmium telluride detector. A 45° expander lens is provided for viewing full pavement width of the bridge structure at one time.

4.4.3.3. Technique: Infrared camera is used for identifying temperature differentials corresponding to delaminated and sound portions of bridge deck. Internal features, such as, voids or delaminations will influence the rate of heating and cooling and may be identified from the study of temperature contours. The concrete surface temperature changes throughout day due to sunlight and cooling at night. Black and white video produced by infrared camera is displayed for identifying delaminated areas. Analysis of data is done by using video tapes recorded. Delaminated areas show white areas on black and white video. Non-delaminated areas show dark areas. Isotherms are drawn. White dots corresponding to points of equal temperature are super-imposed on infrared image and identifies the delaminated areas in concrete structures.

4.4.3.4. Field applications

(A) Application to Reinforced Concrete Structures

Infrared thermographic inspection was employed on top side of 11 mile, 8 lane Dan Ryan

Expressway Bridge Deck in Chicago in 1982 for evaluating the presence and extent of delamination, cracking and efflorescence[3]. Temperature ranged from 60°F (15.6°C) to 80°F (26°C). Pavement temperature differential of 1.8°F (1°C) to 2.7°F(1.5°C) was consistently obtained during data collection.

Infrared data obtained was converted to hard copy photograph or strip charts. Size, shape and location of delaminated areas was identified. Computer Aided Design and Drafting System (CADD) was used for recording the Delaminated datas. It has also been reported that results of thermographic inspection and visual inspection were confirmed by pavement coring. 40 pavement cores were taken to confirm infrare data. Core obtained in each area identified as delamination by infrared technique confirmed the presence of delaminations when visually examined.

A van mounted infrared thermography system was employed in detecting variation of delamination in six Bridge Decks[4]. Infrared and colour video cameras were used for viewing the full pavement width. Delaminated and non-delaminated areas identified by Thermography had good correlation with the core sample taken from the respective areas.

Ontario Ministry of Transportation and Virginia Highway and Transportation Research Council have recommended the infrared Thermography technique as a promising method of detecting delaminations on bare concrete bridge decks[5]. However, this technique was not found to be a promising method in detecting delaminations over asphalt overlaid bridge decks.

(B) Application to Prestressed Concrete Structures

No literature information could be collected with regard to use of this technique in Prestressed Concrete Structures.

4.4.3.5. Liminations: This technique when employed at ground level identifies the delaminated areas with great accuracy. However, its accuracy reduces when the equipment is mounted on an Airborne platform. The operation of this technique is restricted to certain hours of day or night. Weather conditions must also be favourable. Studies made by State of Wisconsin showed that this technique cannot measure the thickness of delaminations[4].

4.4.3.6. Conclusion: This technique identifies only the delaminated areas on reinforced concrete decks. Its application with regard to prestressed concrete structures is not established.

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4.4.4. Holography: Development of internal micro-cracking and detection of such fine cracks in concrete is difficult. Holography is a sensitive, non-destructive technique to detect such micro-cracks. By this technique, cracks with accuracy of the order of 0.1 microns can be detected[1].

4.4.4.1. Principle: Holography is a method of recording phase shift on photographic emulsion, using the principle of optical interference [2,3]. Light from a single source called 'Laser Source' (Helium-Neon laser) split into two beams, one illuminating the object, the other called 'reference beam' is directed on to a high resolution photographic plate. Light scattered by the object called 'Signal beam' is also recorded on the same plate. The constant phase relation between the rays in the reference beam and those in the signal beam produces a complex pattern of interference fringes on the photographic plate. This plate is called 'hologram'. The plate has to be highly sensitive and able to resolve more than 2000 lines/mm at a wave length of 632.8 mm to record the phase modulations accurately. Looking through the hologram it can be possible to see the original object superimposed on the holographic image. If the object is distorted or displaced by fractions of a micrometer, the amplitudes and phases of its reflected rays change, and interfere with the rays produced by the hologram. Now the image through the hologram is seen to be crossed by a series of black interference fringes, which are related to the distortion of the object.

The same principle applies to the detection of cracks on a concrete surface. Small loads will cause the cracks to open, and interference between holograms before and after loading will show a fringe pattern with discontinuities corresponding to the displacement discontinuities that occur at crack positions. Typical fringe patterns are shown in Fig. 4.29.

4.4.4.2 Technique: Simplified optical layout of holographic interferometry setup is shown in Fig. 4.30. There are three methods for observing fringes on holographic plate.

(i) Real-time Holographic Interferometry

In this technique, a concrete structure is gradually deformed with increasing load and its holographic image on the hologram is simultaneously viewed. This technique allows to study the gradual change in crack and deformation.

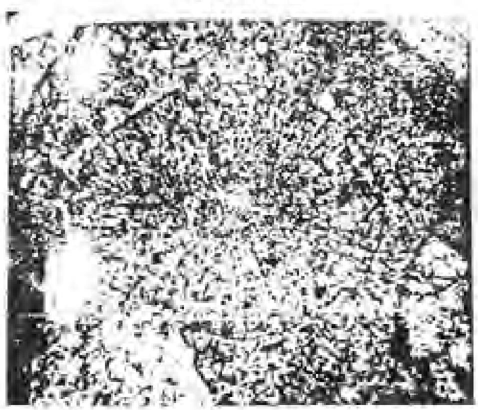
(ii) Double Exposure Holographic Interferometry

In this technique, fringe pattern is obtained on the hologram at the initial and then at the subsequent loading stage. Thus, two holograms are made on the same plate. When this plate is developed and illuminated with the reference light, it contains a fringe pattern corresponding to the displacement between the two loading stages. Thus, this technique allows permanent recording of each loading effect.

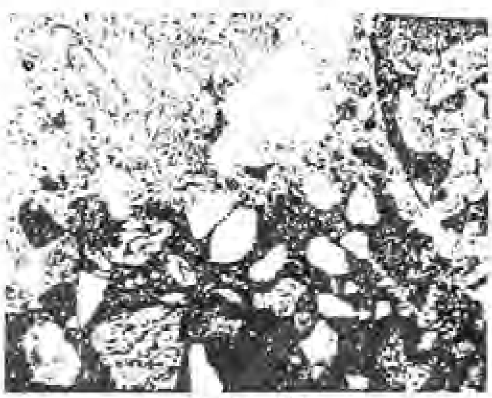
(iii) Speckle Interferometry

In this technique, the object surface is illuminated with a laser light and a photographic camera is used to focus the image of the object on to a photographic plate. The illuminated object surface seems to contain numerous tiny black dots called 'speckles'. These dots are caused by diffraction of the monochromatic laser light from the irregularities on the object surface. When the object is deformed slightly, these speckles move corresponding to displacements on the object surface as though they were imprinted on the surface itself. The mode of crack opening is studied

a) Leaching



b) Sulphate attack



c) Carbonation

Fig. 4.29. Typical Fringe Patterns

by passing the laser through points on each side of the crack and quantifying the difference in displacements. Displacements measured are of the order of 25 microns with an accuracy of 0.1 micron.

4.4.4.3. Instruments needed. For holographic examination following instruments are needed:

Nickelson Interferometer, Holomatic-6000 (Instant holographic camera system with microprocessor control) and other optical instruments as shown in Fig. 4.30.

4.4.4.4. Laboratory investigations: Maji and Shah have performed holographic studies on cement mortar specimen of size 9.4 x 15 x 2.5 cm to know the effect of inclusions, bond cracks, matrix cracks, diagonal cracks, etc. while the specimen was loaded axially[1]. They used this technique to understand the fracture mechanics of cement mortar. From the studies they concluded that the crack initiated at the aggregate matrix bond leading to non-linearity in the load-deformation relationship. They also concluded that bond cracks seemed to initiate at sites where shear stresses are expected to occur, whereas, matrix cracks originate from top and bottom of the aggregate and propagate in the direction of applied load. Diagonal cracks are probably a result of end-constraint and they join the vertical cracks when nearing the ultimate load.

Another important conclusion is that the aggregate dimension and void dimension have differing effects on the overall response of the concrete.

Luxmoore has carried out holographic technique on 15 cm size cubes and cylinders to

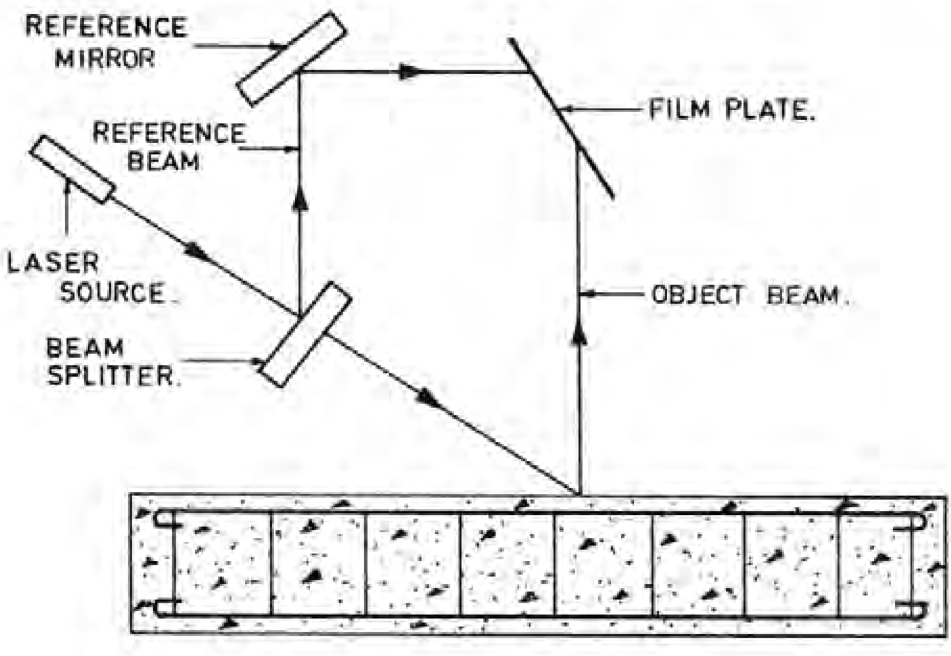


Fig. 4.30. Holographic Examination of R.C.C. Beam for Crack Detection

understand the basic failure mechanisms in concrete[4]. Specimen were subjected to compression test and splitting tensile test to study crack initiation, location and its length. From the studies he found that the first fine crack was detected at around one quarter to one third of the ultimate load. As the load increases, the crack branched out to the right and left, and other cracks started and joined the initial crack system. At around three quarters of the ultimate load, a vertical crack propagated from the base and connected with the cracks propagating vertically downwards from the top just prior to the ultimate load. Failure occurred catastrophically along the vertical diameter, with some spalling along the edges, but in all cases, the inter connection of the crack system occurred just prior to failure. These results were compared with X-ray radiography, eddy current testing, penetrating dyes and found to be just as reliable.

4.4.4.5. Field applications: It appears that this holographic technique has not been applied so far as in-situ method to detect the microcracks present in the structures.

4.4.4.6. Limitations: Applications of holography technique in fieldwork is limited because of its sensitivity to small extraneous vibrations and displacements. It is a very slow photographic process. This technique is ideal for testing the central and flat surface of the structure and is not suitable for corners, joints and curved portions.

4.4.4.7. Conclusion: This technique is still in an experimental stage and not applied on large scale structures.

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4.4.5. Petrography

4.4.5.1. Introduction: Physical condition or homogeneity of concrete is one of the important parameter influencing the durability of concrete structures. Petrography is the examination of thin section of hardened concrete core under a suitable microscope.

Petrography can be used to examine the hardened concrete for the following purposes[1]:

(i) To find the amount, distribution, type and condition of cement paste

(ii) To identify the possible destruction of the protective passivation zone around steel reinforcements which results from the carbonation of cement paste

(iii) To determine the quantity and distribution of air voids, whether particular concrete is air entrained or not

(iv) To examine the distribution, types and amount of coarse and fine aggregates

(v) To deduce the causes of deterioration and future serviceability of the concrete

4.4.5.2. Principle: The sample of thin section either cement paste, mortar or concrete is examined under the microscope at a wide range of magnification. A camera equipped with such microscope to take photographs and photomicrographs of features of interest on test sample is used. Then these photographs are analysed for each phase to arrive at suitable conclusion.

4.4.5.3. Technique

(i) Binocular Stereo Microscopes

This type of microscope is employed mainly on aggregates and sliced concrete cores at magnifications up to about x100.

(ii) Polarizing Microscopes

These are used to study thin sections of about 30 microns by transmitted light method at magnifications typically between x20 and x500. Such thin concrete sections are prepared by grinding samples on laps using progressively finer abrasive.

(iii) Metallographic Microscopes

These are employed for the identification of cement type in concrete matrix by incident light method at magnifications upto x1000.

4.4.5.4. Instruments needed: For petrographic examination, following instruments are needed[2]. Stereo microscope, polarizing microscope, metallographic microscope, microscopic slides, eyepiece micrometer, microscope lamps, free abrasive machine, hot plate or oven. Diamond saw, abrasives, lens paper, refractometer and immersion media, cover glasses. Dollies (specimen holder), stage micrometer, cutting lubricant for diamond saw.

4.4.5.5. Interpretation of photomicrographs

(A) Alkali-aggregate Reaction

Photomicrographs are examined for voids, cracks and presence of 'Rims' on aggregate particles. Rims indicate chemical reactions between the cement and the aggregate. Rims produced in the concrete on particles of sand and gravel are absent or relatively thin and faint at locations where the particle is in contact with an air void. Rimmed crushed stone in concrete usually indicates alteration in the concrete due to alkali-silica reaction or alkali-carbonate reaction [3,4]. Pale rims in mortar bordering coarse aggregate and pale areas in the mortar may be gel-soaked paste or highly carbonate paste [5,6,7].

In normal good quality concrete, the petromicrographs show, amorphous cement gel surrounded by aggregate with calcium hydroxide crystals quite evenly distributed but with slight concentrations under the coarse aggregates and fine aggregates. Quartz, calcium hydroxide and calcite, these three materials occur together, the quartz particles look like sand grains or rock fragments, while calcium hydroxide either as tablets tangential to aggregate or as poiklitic crystals in the paste enclosing residual cement or areas of gel. Calcite are approximately rhombic in shape.

(B) Type of Cement

Types of cement are differentiated on the basis of mineral phases present in the unhydrated clinker particles. In OPC the major phases are C_3S, C_2S, C_4AF and C_3A. In sulphate resisting cement, there is more C_4AF in lieu of C_3A, which is only present in small quantities. Similar studies can identify the other types of cement, such as, super-sulphated cement, portland blast furnace slag cement, high alumina cement, etc.

(C) Deterioration

(a) Leaching

When cement clinker particles hydrate, free lime is liberated to form calcium hydroxide [$Ca(OH)_2$] which crystalises as a mineral portlandite. As lime is slightly soluble in water forming carbonic acid with CO_2 present in the atmosphere, concrete is capable of being leached by this carbonic acid molecule. This leaching action can be seen in the photomicrographs given in Fig. 4.31(a) as a zone bordering the leached area and here coarse tabular crystals of portlandite [$Ca(OH)_2$] are found and these are so distinctive in nature.

(b) Sulphate Attack

Sulphate present in the water react with C_3A phase of the cement clinker, produce calcium sulphoaluminate which crystalises as the mineral known as Ettringite. This forms fine acicular crystals in the cement paste as shown in Fig. 4.31(b). In concrete ettringite appears to be in active state and in some others it is in passive state. In its passive role it is found in filling air voids and in active state it is often found along with cracks.

(c) Carbonation

When carbon-dioxide in the air reacts with the free-lime present in the cement paste, calcium carbonate crystals are formed. Under the microscope, the high order interference colours produced by the calcite crystals considerably highlights the carbonated zone. Typical petromicrograph is shown in Fig. 4.31(c). The iron in the cement paste in the carbonated zone is also highly oxidised. Comparative studies have shown a close correlation between the phenolphthalein staining test and microscopical analysis in determining the extent of carbonation.

(d) Air-void Content

Air-voids present in the concrete can be detected by petrographic examination[8]. Air-voids

a) Uncracked

b) Cracked

Fig. 4.31. Typical Photomicrographs

are almost invariably larger than 2 μm in diameter expressed as the proportional volume of air voids in concrete to the volume present of the hardened concrete. This ratio can be determined by either linear traverse method or pointcount method. Practically, air void determination being applied most frequently in determining whether a particular concrete is or not air entrained.

4.4.5.6. Field applications

(A) Applications to Reinforced Concrete Structures

Power and Hammersely have done petrographical examination on a dam in southern Scotland[9]. From the petrographical analysis it was found that the dam has suffered microcracking as a result of aggregate shrinkage. The coarse aggregate being greywacke had shrunk causing a diagnostic peripheral micro-cracking in the cement paste surrounding the aggregate particles.

A concrete sample taken from a ten-year old 150 mm thick suspended floor slab which had contracted inwards was subjected to microscopical analysis. From the analysis, it was revealed that considerable shapeless crystalline masses of $Ca(OH)_2$ were present in the cement paste throughout the concrete slab. This is due to the addition of high water-cement ratio causing high drying shrinkage.

Samples taken from the ground slab which was removed from the demolished building were subjected to petrographical analysis. The base of the concrete slab was very soft and extensive amount of cement paste had been removed. From the microscopical analysis*, it was found that the aggressive leaching of the lime was considered to be the main cause of the deterioration and this view was further supported by the fact that the water at the base of the concrete was very alkaline, having a pH in excess of 9.

(B) Applications to Prestressed Concrete Structures

This petrographic examination has been carried out on the samples taken from the O'Hare Airport Leads Bridge, Desplaines[10]. This bridge consists of I shaped precast pretensioned girders and cast-in-place concrete deck slab. Petrographic analysis showed that concrete in prestressed girders are generally of good quality. Depth of carbonation of cement paste was in the range of 1.5 to 6.4 mm and in some places it was 12.7 mm. Secondary deposits in aggregate voids and sockets, indicative of possible adverse reactions were not detected. Aid-void content appeared to be low varying from 3 to 4.5 per cent.

4.4.5.7. Limitations: Petrography is a destructive type of test. So, large scale examination of the structure may not be feasible. Petromicrographs need experienced petrographer to interpret the results. Special surface preparations are necessary which are time consuming.

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* Microscopic and chemical tests on lumps of spalled concrete and grout were carried out for the first Thane Creek Bridge.

4.4.5.8. Conclusion: Petrography is not a corrosion monitoring tool. At best it can be used to examine the quality of the concrete core.

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4.5. Monitoring the Structural Response

4.5.1. Vibration analysis

4.5.1.1. Introduction: Monitoring of dynamic properties of structures periodically will indicate the degree of deterioration and structural integrity. Vibration analysis is used to measure the dynamic properties of the structures. This is a non-destructive testing technique in the field of testing bridges in-situ.

4.5.1.2. Principle: All structures vibrate in accordance with the type of structure and have several natural frequencies, specific damping and dynamic factor. Significant changes in these parameters indicate a modification in the mechanical and physical conditions of the structures caused by degradation due to time-dependent material properties, environmental aggressive elements or alterations in the structure or its supports. Hence, the vibration test can help to assess the time-dependent behaviour of the bridge structures.

4.5.1.3. Technique: Natural frequency is the frequency which is the characteristic of the body. Modification of the natural frequency of a bridge structure can be caused by: cracking in the concrete, change of temperature, change of the condition of the supports, change of prestressing; creep and shrinkage. The damping factor is used to explain the dissipation of energy in a vibration structure and this can be used to study the internal microcracks by measuring the change in acoustic emission of pulses. The dynamic factor is the quotient between the dynamic increase of amplitude and amplitude under corresponding static load. This will not provide a suitable index to assess the time dependent behaviour, but its determination will provide control over the assumed impact factor in the design.

By using the suitable excitation procedure at the bridge site, the dynamic properties, such as, displacement, acceleration of vibrations, modes of vibration, coherence, phase difference are measured with the aid of suitable instruments. Fig. 4.32 shows the arrangement for measurement of vibration response on bridge deck.

4.5.1.4. Equipments needed: Cantilever deflection gauge, Accelerometer, Oscillograph, Oscilloscope, Analogue-digital trace analyser, Transfer function analyser, Digital profilometer, Traffic detectors, Vibration generator, Dynamic wheel load recorder.

4.5.1.5. Interpretation of test results

Analysis of frequency spectrum

The frequency spectrum generated in the few seconds immediately after the application of the impulse indicates the physical characteristics of the bridges. In the long term repeated observations, carried out in similar environmental conditions, it has hence to be ensured that the geometric and mass parameters of the bridge are unchanged, the system oscillates freely in its natural modes and the exciting source and its location are the same. Changes in any of these parameters, if detected, should be kept in consideration while analysing the frequency response. A peak must appear at the same frequency in the spectra in all measured locations.

(a) Cracks

Cracking in the bridge structure decreases the moment of inertia and consequently the natural frequency is decreased. Cracking produces an increase in damping[1].

(b) Change in prestressing force

Any change in prestressing force can change the EI stiffness of the structure and consequently the natural frequency can be changed.

(c) Continuity in multi-span bridges

When the continuity in multispan bridge exists then the measurements made in any span show the natural frequency relative to this span and the frequencies relative to the neighbouring spans. The phase angle difference is close to 0° or 180°, with high coherence. If the continuity is disturbed by cracks or change of prestressing, then the perturbation in frequency spectrum is present. The phase angle is unstable with low coherence.

4.5.1.6. Field applications

Applications in monitoring prestressed concrete structures*

This vibration analysis has been carried out in one of the post-tensioned bridges in India.

* Vibration studies were carried out for Bassein Creek Bridge by I.I.T., Bombay, with similar indications.

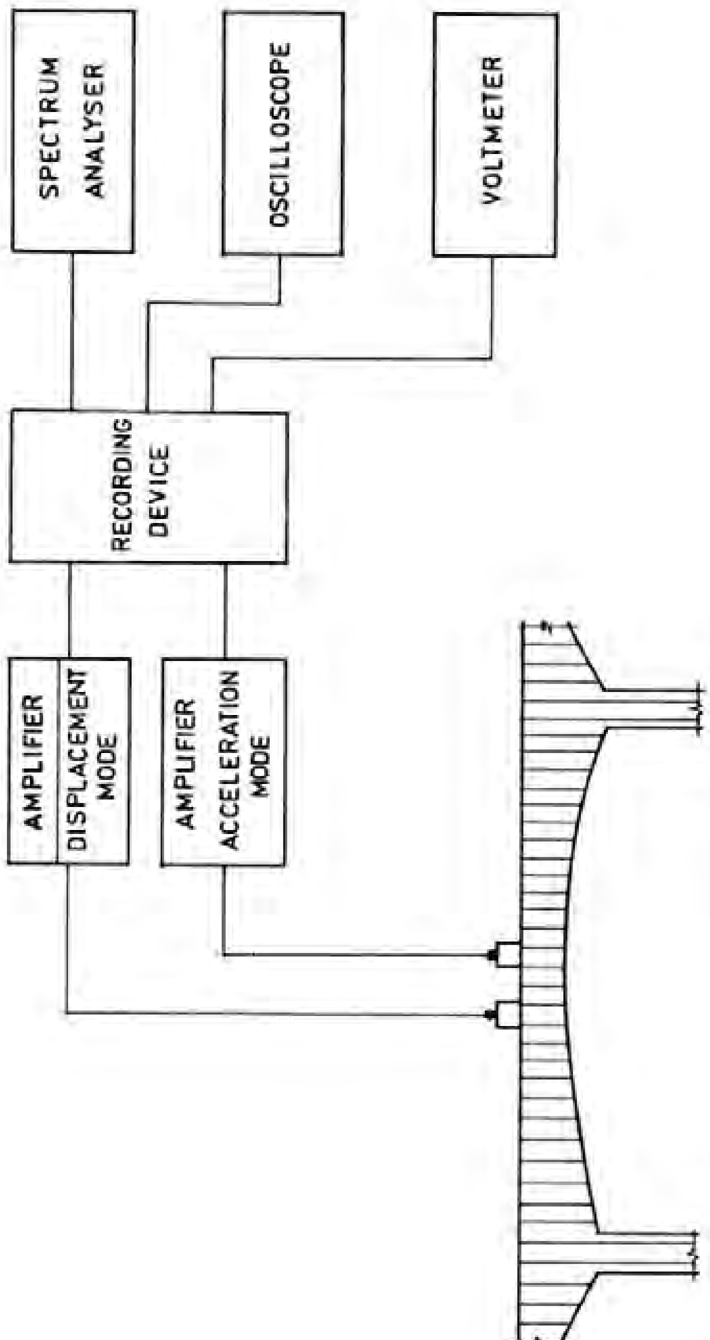

Fig. 4.32. Measurement of Vibration Response on Bridge Deck

Following were the observations in this bridge. Since the traffic conditions are quite varying, strict comparison of acceleration levels could not be made. It was also found difficult to assess the loss of strength from the data obtained. There was also a problem regarding the natural frequency of the structure. From the vibration analysis it was observed that the structural stiffness was not very much affected. The results of response levels measured didn't indicate any correlation to the loss of prestress due to corrosion. The vibration levels measured didn't reveal any abnormal features in the response.

4.5.1.7. Limitations: Due to the presence of complex parameters in this test, the vibration analysis can only help to corroborate the findings from other techniques usually employed to monitor the performance. Change in temperature, bridge bearing and thickness of the bituminous road surface can affect the measurement of dynamic properties. This method measures the loss of stiffness and not the loss of strength. A measurable loss of stiffness generally implies a loss of strength, for concrete bridges. But sometimes serious loss of strength can occur without producing a measurable loss of stiffness. Since the vehicular loading on any bridge is influenced by speed, interval, deck surface conditions, etc., it is extremely difficult to arrive at the actual vehicle frequency and hence the response of the structure. Thus, it is very difficult to arrive at the exact fundamental frequency of the bridge structure.

4.5.1.8. Conclusion: Vibration analysis can throw some useful information on the stiffness of the bridge structure provided it is possible to accurately monitor the frequency response and also to estimate fundamental frequency of the bridge structure.

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4.5.2. Strain analysis

4.5.2.1. Principle: Long term observations of bridges are carried out by comparing dynamic characteristics of the structure with static analysis. The importance of any differences with regard to the safety and serviceability of the bridge is to be assessed. The determination of dynamic characteristic is done by measuring strains during the passage of vehicles on the bridge. For large span bridges over 50 m strains are measured at the centre as well as at the 1/4th and 3/4th of the span. Both surface and internal strains are measured.

4.5.2.2. Equipments needed

Deformometer,

Built-in-vibrating wire-strain-gauges.

Address of suppliers

Telemac, 17, rue Alfred-Roll,
75011 Paris, (France)

Strainstall Ltd.,
Cowes, Isle of weight, (U.K.)

TM Stress Measurements & Engineering Pvt. Ltd.,
Sterling Centre, 16.1, Dr. Annie Besant Road, Bombay 400 018

4.5.2.3. Technique

(a) Surface strains

Surface strains are measured by measuring bilateral displacements of attached metal marks or built-in-pins by deformometers[1]. By using deformometers with the base greater than 15 cm, it is possible to measure the strain values precisely to the value of 0.001 mm. Deformometers are generally fitted with dial gauges or inductive transducers.

(b) Internal strains

Internal strains in the bridge structure are measured by built-in vibrating-wire gauge[2]. The length of the vibrating wire gauge is mostly between 10 cm and 30 cm. This is fixed between the two limiting points of a known length and put into vibration by the use of an electromagnet. The wire vibrates at a frequency which is proportional to its tension. When the structure is subjected to loading, the change in length of the wire and thereby the frequency of vibration of the wire are measured. In this method strains are precisely measured to 5×10^{-6} of unit deformation.

4.5.2.4. Analysis of results: For analysis of results, the strains have to be separated into two components. One is anticipated strains and another one is non-anticipated strains. Anicipated strains are those which have been considered in the static analysis so that it is possible to take measures against their unfavourable consequences, (e.g.) the effects of anticipated load, concrete creep and shrinkage, temperatures, settlements of supports. Non-anticipated strains are those which have not been considered in the static analysis so that they can contribute to the depreciation of structure (e.g.) unexpected settlement of supports, the failure of adhesion between reinforcement and concrete, large decrease of material strength.

The anticipated strains can have two components: One is theoretical strains and another one is actual strains. Theoretical strains, which are the consequence of static analysis and which have been determinated on the basis of the requirements of standards and regulations. Actual strains, which are the consequence of static analysis in which the data obtained by long-term measurement of the bridge. Using these measurements it is possible to evaluate the bridge behaviour by comparing total strains (anticipated strains + non-anticipated strains) with theoretical anticipated strains.

If the total strain is equal to the theoretical strain, the behaviour of structure is normal. If the total strain is greater or smaller than theoretical strain, then measured actual strain is used in place of theoretical strain. Even after, if the total strain is greater or smaller than measured actual strain then the structure has to be subjected to further investigation.

4.5.2.5. Limitations: The sensitivity of the measurements depends upon the strain sensors. The sensors may get damaged during use. Interpretation depends upon the accuracy with which theoretical strain has been calculated. It is not possible to isolate the causative factor.

4.5.2.6. Conclusion: It can give some useful information about the overall structural performance.

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4.5.3. Deflection analysis

4.5.3.1 Introduction: Load bearing capacity of the bridge structures and their reliability and safety can be assessed by monitoring deflections periodically. The actual deflections measured at critical points have to be compared to that of the theoretical values. Wherever the values are differing by a large margin, the structure is to be analysed for further investigation.

4.5.3.2. Principle: The deflections due to vehicular loading are measured at critical places by using deflectometer. The deflection measurements at each measuring point may be plotted along with the permissible values as per standard code of practice.

4.5.3.3. Equipments needed: Deflectometer, Precision level, water levels, strain gauges of both mechanical and electrical type. The difficulty with the levels is that measurements have to be taken in the early morning and traffic has also to be stopped. The electrical strain gauges sometimes behave erratically because of altered resistance of leads due to corrosion. Direct reading mechanical strain gauges are better than manual ones. The deflection monitoring can be done only subsequent to the initial reference readings.

4.5.3.4. Technique: Critical locations for placing deflectometer are selected which depends on the particular type of structure. For bridges with simply supported beams the deflections are measured at the centre of the span and at the maximum positive bending moment[1]. For long span bridges over 50 m and statically indeterminate structures, the deflections are measured at the centre and at the 1/4th and 3/4th of the span. Measurements should be made at least twice every year, preferably after the winter and summer months. During the initial 3 years of service the observations are to be executed preferably four times every year.

4.5.3.5. Laboratory investigation carried out at CECRI: CECRI has carried out some studies on simply supported beam to know the effect of corrosion on deflection behaviour. From the studies, it was observed that when the diameter reduced from 9.6 mm to 8.4 mm by corrosion, the deflection correspondingly increased from 0.06 mm to 0.08 mm. When the diameter reduced from 9.6 mm to 7.2 mm by corrosion the deflection correspondingly increased from 0.06 mm to 0.10 mm.

4.5.3.6. Limitations: Theoretical deflections are based on design factors, whereas, actual deflection is influenced by vehicular loading, age of the structure, temperature and so on. These may introduce some errors particularly when deflection monitoring is commenced some years after construction. In such cases, continuous monitoring over few years only can give some useful data.

4.5.3.7. Conclusion: It can give some useful information about the overall structural performance.

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4.5.4. Acoustic emission: Acoustic emissions are transient waves generated by rapid release of energy within the concrete. Due to low amplitude or high frequency of sound, these sounds are usually not audible. Acoustic emission technique has been used to study the formation, propagation and influence of microcracks in concrete structures[1,2]. This technique is a diagnostic method for evaluation of crack initiation and for evaluation of deterioration by testing of core specimen removed from concrete structures. Propagation of cracks in concrete structures is followed by detection of acoustic emission events.

4.5.4.1. Principle: This technique records the sounds emitted from concrete under load. Transducer is used for detecting the emissions. Analysis of acoustic emission wave motions is done by studying the energy and frequency of sound waves.

4.5.4.2. Equipments needed: Acoustic Emission Technique Model 5500 data acquisition systems is used for monitoring the crack initiation.

Address of supplier for AET 5500 Analyser

Hartford Steam Boiler Inspection Technologies
1600, Tribute Road, Sacramento, CA 95815-4400, (USA)

4.5.4.3. Technique: The sound emission test equipment records the sounds emitted from concrete under load. The acoustic energy is transformed to electrical energy by using a barium titanate transducer with a resonant frequency of 100 kilo cycles which is attached to the concrete surface with silicone grease and positioned using a special holder. Transducer detects the acoustic emission waves. The schematic diagram of the system is shown in Fig. 4.33. Transducer is connected to an amplifier and to a sensitive tape recorder set at highest speed. During failure of the specimen, the recorder synchronized with the loading records the sound emitted by the concrete. The load at which crack occurs can be clearly analysed by connecting the tape recorder to an oscillograph recorder with print the sound energy on a graph as function of load. The frequency of sound waves is measured using a storage oscilloscope. Pulse and pulse width are measured. This technique thus, monitors the initiation and propagation of cracks by detecting the acoustic emission events (energy frequency of sound waves).

4.5.4.4. Laboratory investigations; It has been reported that acoustic emission technique can monitor the formation and propagation of cracks even at low loads particularly for mixes containing high proportion of coarse aggregate[1]. This technique enables studying the loads at which internal disruptions occur in concrete specimen and appears to be a sensitive method of determining the load at which microcracks occur and giving an indication of energy change that accompanies cracking.

It has been reported that acoustic emission technique can be used for measuring the extent of damaged zone on concrete surface[3].

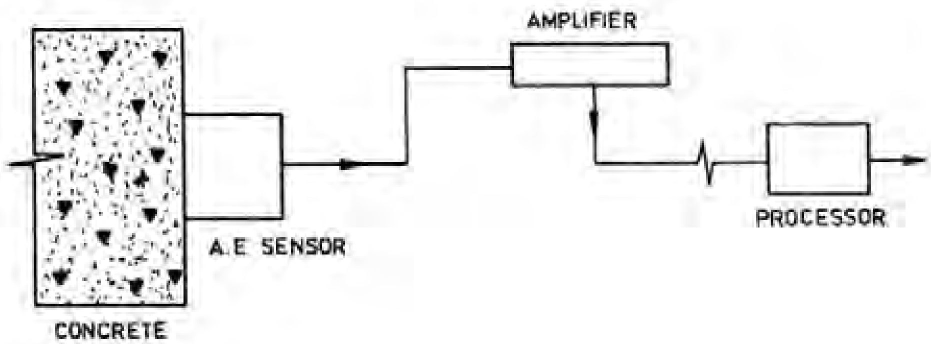

Fig. 4.33. Schematic Diagram of Acoustic Emission System

Acoustic emission activity becomes high under uniaxial loading in the concrete containing microcracks[4].

4.5.4.5. Field applications

(A) Application to Reinforced Concrete Structures

Acoustic emission technique was employed in detecting the deteriorated portion of a 30 year old concrete Silo[5]. Acoustic emission events were active and were observed upto 80 per cent of failure load. Concrete core specimen extracted from the structure contained considerable amount of critical microcracks.

It has been reported that the concrete core samples extracted from deck slabs in a bridge structure was subjected to this test for assessing the damaged portions in the structure[4]. Core specimen from left lane and right lane of the deck slab was tested. In-right-lane, numerous acoustic emission events were observed even from low loading level while in left-lane, few acoustic emission events were observed upto final loading. Hence, concrete from right-lane contained microcracks and was in deteriorated condition when compared to left-lane of the bridge structure.

(B) Application to Prestressed Concrete Structures

It has been reported that acoustic emission technique has been employed periodically in detecting cracks in bridge structures[6]. Battle Pacific North West Laboratories has applied this technique on a damaged prestressed concrete bridge structure and has collected relevant data.

According to FIP, acoustic emission technique can be used for detecting the failures of prestressing strands and wires in the prestressed concrete structures[7].

4.5.4.6. Limitations: This technique is only a diagnostic method for evaluation of cracks in concrete structures.

4.5.4.7. Conclusion: Since no direct correlation has been established acoustic technique is not useful as a corrosion monitoring technique.

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4.5.5. Optical fibre sensors

4.5.5.1. Introduction: Formation and propagation of cracks in concrete structures are detected now-a-days by optical fibres. Materials based on hydraulic binders, all mineral fibres, such as, glass or silica are used as optical fibres. This method is flexible and inexpensive to study the fracture behaviour of concretes and for the minitoring of reinforced or prestressed concrete structures.

4.5.5.2. Principle: A light emitting diode is used as a transducer to emit the luminous light. An optical fibre is used to transmit this signal. When this signal is intercepted by a defect in the system, it disappears. This is recorded by the receiver, which may be a photodiode.

4.5.5.3 Instruments needed: Following instruments are needed: Light emitting diodes, photodiodes, other accessories, such as, voltmeter, graphic recorder, threshold detector and alarm, optical fibres of diameter 133 to 200 μm and capable of detecting a crack opening of less than 50 μm.

4.5.5.4. Technique: This method can be used to detect, number of cracks and deformation by modifying the instruments present in the circuit. Fig. 4.34 shows simplest arrangement for crack detection using optical fibres.

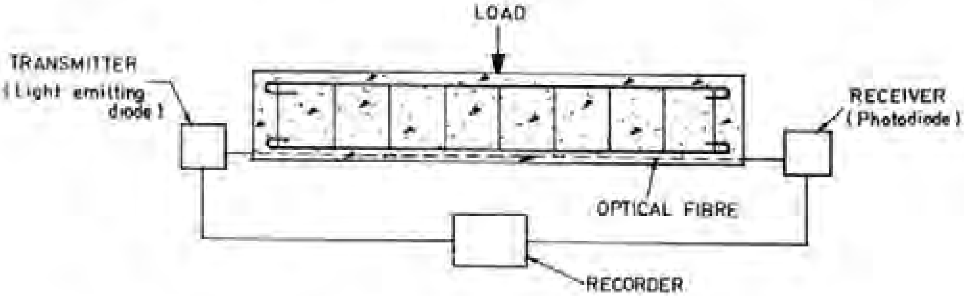

Fig. 4.34. Optical Fibre Technique for Crack Detection of Beam under Load

For Detection

One or few fibres are placed in the zone where the cracks are more probable to occur. These cracks are detected by suitable alarm or recording arrangements.

For Location

To obtain direction and number of cracks present in the concrete member, a multi-channel surveilance system should be incorporated in the circuit.

For Predicting Deformations

With the help of signal processing device by back-scattered signal method, the deformation of the concrete member along the length of the optical fibre can be detected.

Placement of Fibres

Tubes about 1 metre length are placed for positioning fibres in the concrete. These are held in place during concreting by providing holes in the moulds. These tubes are attached to the reinforcement bar in such a way that it can withdraw axially after final setting of concrete.

4.5.5.5. Laboratory investigations: Pierre Rossi has performed this technique on double cantilever beam to detect crack propagation[1]. For crack monitor he used a strip of silver lacquer perpendicular to the direction of the crack. Through this strip a small current was applied, when crack passes through this strip, a clear drop in voltage occurs. From the studies he plotted the approximate shape of the crack tip and this is very similar to the crack tip obtained by resin impregnation technique[2].

4.5.5.6. Field applications

(A) Applications to RCC Structures

Pierre Rossi Fabrice-Le Maou have added these optical fibres in one of the caissons in the Tunnel under the Morhe river near Paris. In submerged caisson, to monitor the cracks and their propagation optical fibres were placed in the cross section of one of the shell at various heights. This study is in the experimental stage now and the results are not given. But the Authors concluded that this technique may be used to detect the surface cracks where other means of detection is not possible.

(B) Applications to Prestressed Concrete Structures

No literature information is available with regard to the application of this technique in prestressed concrete bridges.

4.5.5.7. Limitations: This method is applicable only to a structure being built. In field, placing of fibres is very tricky, because fibres are fragile and may get damaged during concreting. Reliability and reproducibility of this method is yet to be established. Effectiveness of this method will depend essentially on an understanding of the various possible mechanisms of failure.

4.5.5.8. Conclusion: Eventhough not much data is available on the field application of this method, with sufficient experimentation, this may probably prove useful as a sensor.

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4.6. Field Inspection Techniques and Laboratory Chemical Analysis

The objective of field inspection is to obtain information concerning the susceptibility of rebar corrosion and to know the present state of environmental penetration and deterioration. This inspection is very vital and enables to know the type of repair needed and the area needing this repair so as to avoid catastrophic failure of the strucure. This field inspection and chemical analysis consist of,

1. Visual observation with photographs
2. Cover thickness measurements
3. Depth of carbonation
4. Detection of atmospheric pollutants
5. In-situ strength determination
6. Coring
7. Analysis of concrete samples

8. Analysis of reinforcement materials, grouting materials
9. Analysis of water
10. Structural details

Methodology

4.6.1. Visual survey of the structure with photographs: This is carried out in order to detect all symptoms of damage and defects visually using standard tools. This is to be carried out for the following aspects.

4.6.1.1. Cracks: Cracks are common in concrete because of its low tensile strength and relative large volume change. In making a condition survey, location, orientation, width and depth of cracks should be reported.

Detection of cracks

Cracks are detected either visually, if possible, or by the use of flourescent dyes. Ultraviolet light is a sensitive and well tried techniques[1]. Nowadays an acoustic crack detector or magnetic crack definer or magnetic particle technique is used[2].

Location

The location of the cracks is either shown on a scale plan or described.

Orientation

Longitudinal, transverse, diagonal and random are the terms used to describe the orientation of a crack with respect to the major axis of the member.

Crack width

Crack width is measured according to the following scale[2].

Narrow	-	less than 0.3 mm
Medium	-	between 0.3 mm and 1.0 mm
Wide	-	greater than 1.0 mm

Depth of cracks

Depth of crack can be determined either by coring or some other NDT technique, such as, ultrsonic pulse velocity technique.

Some more types of cracks usually occurring on concrete surface are listed below:

(i) Hairline cracks

Small cracks or random pattern in an exposed concrete surface

(ii) Pattern cracks

Fine openings on whole concrete surfaces follow a particular pattern. This is resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface or both.

(iii) Checked cracks

Development of shallow cracks at closely spaced but irregular intervals on the surface of the concrete.

4.6.1.2. Scaling: Scaling is the flaking of the surface mortar of the concrete. As the scalling progresses, coarse aggregate is exposed and eventually loosened. Scaling result from poor finishing and curing practices, presence of salts and inadequate air entrainment. The areas of scaling, which are either described or shown on a scale plan are classified as follows[3].

Severity of scaling	Depth (mm)	Characteristic appearance
Light	0 to 5	Coarse aggregate not exposed
Medium	6 to 10	Coarse aggregate exposed
Heavy	11 to 25	Coarse aggregate projecting from the surface
Severe	Over 25	Loss of coarse aggregate particles

4.6.1.3. Surface Deposits: The most common type of surface deposits are efflorescence, rust stains, exudation, dampness and incrustation.

(i) Efflorescence

It is deposit of salts, usually white, which result from the flow of a solution from within the concrete to the surface where the water evaporates.

(ii) Rust stains

Brown colour stains are observed in the cracks and the concrete surface in which rebars are exposed to the atmosphere.

(iii) Exudation

It is the solid-gel, like, material discharged through the cracks present in the concrete surface. This results from the alkali aggregate reaction[4].

(iv) Incrustation

A crust which is white in colour generally hard formed on the surface of concrete. This is due to leaching of lime from cement[4].

(v) Dampness

The extent of water on the surface due to improper drainage in the structures.

4.6.1.4. Some other defects

(i) Pop-outs

The breaking away of small portions of concrete surface due to internal pressure which leaves a shallow, typical conical depressions on concrete surface. Diameter of this depression is usually around 50 mm[5].

(ii) Drummy area

Area of concrete surface which gives off a hollow sound when struck with suitable hammer or dragged chain method.

(iii) Discoloration

Departure of colour from that which is normal or desired.

(iv) Honey combing and air pockets

These defects originating at the time of construction may be present in formed surfaces. Air pockets results from insufficient vibration and honeycombing which results from incomplete consolidation and leakage of the mortar at a joint in the formwork.

4.6.1.5. Photographs: Photographs are taken where extensive deterioration is evident, typical areas, general view of the structure and cores with significant deterioration.

4.6.2. Cover thickness measurements: A covermeter or pachometer or profometer is used for measuring concrete cover. By means of this it can be able to detect rebar size, direction and position.

Principle

Measurements are based on the damping of a parallel resonant circuit. An alternating current with a given frequency flows through the proble coil, thus creating an alternating magnetic field. Metal objects within the range of this field alter coil voltage as a function of cover and bar diameter.

Profometer or covermeter

It comprises a probe and the indicator unit. The electronic system, controls, indicator instruments are assembled on the indicator joint front panel. Eleven different bar diameter may be set in a rotary selector switch with a range from 8 to 34 mm. By means of this, the maximum cover thickness can able to be measured is 120 mm.

Position and direction of rebar

The position and alignment of reinforcement bars embedded in concrete is detemined by systematic scanning of the probe on the surface. The probe is positioned directly over a rebar when maximum deflection is indicated on the neutral scale of the instrument.

Cover thickness

After position and direction of the reinforcement bars have been ascertained, thickness of the concrete cover can be directly read, if the rebar diameter is pre-set.

4.6.3. Carbonation: Carbon-di-oxide present in the atmosphere react with the calcium hydroxide present in the concrete forming calcium carbonate and additional water. This carbonation process leads to the neutralization of the concrete pore solution ($pH < 9$) and causing the breakdown of the passivity of the rebars. The progress of the carbonation front into the concrete can take place through the following three processes.

(i) By means of chemical process, i.e., the reaction of CO_2 with $Ca(OH)_2$ of the pore-water which result in the formation of additional water.

(ii) By means of diffusion process, i.e., the diffusion of CO_2 through the concrete which is already carbonated.

(iii) Diffusion of water formed in the chemical reaction.

Since diffusion process must take place through the pores in the concrete and porosity has decisive influence on the carbonation process.

Thus, the removal of calcium hydroxide from concrete reduces CaO content and leads to the decomposition of other concrete constituents viz., hydrosilicates, hydroaluminates and hydroferrites, resulting in loss of strength and deterioration of concrete. This process of carbonation, although progressing slowly, penetrate into the concrete to a considerable depth.

Method of determining depth of carbonation

The depth of carbonation of the concrete in various parts of the structure is ascertained by using bromo-cresol purple as an indicator[6]. The indicator turns yellow in acidic medium and violet in alkaline medium. The depth of yellow colour gives the depth of carbonation. Phenolphthalein indicator is also used to detect the depth of carbonation. The indicator turns purple in alkaline medium and colourless in acidic medium[7].

4.6.4. Detection of atmospheric pollutants: The atmospheric pollutants present in the atmosphere, such as, chloride, sulphur-di-oxide and carbon-di-oxide are determined by exposing salinity candle and sulphur-di-oxide candle respectively at the field.

Chlorides

The chloride in the atmosphere is estimated by 'salinity candle method'. This method is based on the absorption of chlorides present in the atmosphere by the wet gauze, which is then digested with hot distilled water to remove all chlorides. The chloride is estimated volumetrically by Mohr's method[8]. Atmospheric salinity prevailing at the site is independent of the distance from the sea. A chlorde content of more than 100 mg/m^2/day should be considered as quite aggressive.

Sulphur-di-oxide

The sulphur-di-oxide candle is covered by means of gauze cloth containing lead peroxide paste. The sulphur-di-oxide atmosphere reacts with lead peroxide forming lead sulphate. The lead sulphate is made to react with sodium carbonate with the formation of sodium sulphate. The sulphate is precipitated as barium sulphate by the addition of barium chloride.

Carbon-di-oxide

CO_2 present in the atmosphere is estimated by passing atmospheric air through a known volume of barium hydroxide solution of known normality kept in a glass container for a particular period of time. This barium hydroxide kept in the container is converted into barium carbonate. The solution is titrated with standard oxalic acid for getting the volume of unreacted barium hydroxide. From these values CO_2 present in the atmosphere can be obtained in mg/m^3.

4.6.5. Coring: This is the method of collecting the concrete samples from the structure. This is destructive in nature and consists of drilling the hole perpendicular to the horizontal surface using a penumatically operated coring machine[9]. The core samples taken from the various locations can be carefully analysed for strength determination, chloride permeation and other NDT techniques such as petrographic examination. The diameter of the core varies from 50 mm to 100 mm.

4.6.5.1. Strength determination from core samples: The diameter of the core for the strength determination should be minimum of 100 mm. These core samples are capped before making the compression test and tested under standard compression testing machine[10].

4.6.5.2. Rapid chloride permeability: Determination of the rate of chloride permeability on core samples can be carried out by a method proposed by FHWA[11]. In this test concrete core sample of 50 mm length is placed in the test cell and voltage of 60 V is applied between two electrodes on each face of the core. Chloride ions present in the left chamber of the cell, which is filled with NaCl solution move through the core sample to the right chamber of the cell, which is filled with an alkali solution. The amount of current transmitted during 6 hour is a measure of the chloride permeability of the concrete. If the current transmitted during 6 hr. is in the range of 1823 to 1919 coulombs then it will indicate that the rate of permeability for chloride is low.

4.6.6 Chemical analysis of concrete samples: Penetrated chlorides and sulphates from the atmosphere as well as polluted water surrounding the structure cause deterioration. All these chlorides and sulphates do not take part in the corrosion process. Only residual chloride and sulphate are responsible for promoting corrosion and remainings react with the constituents of cement and become inactive[8]. It is necessary to analyse the concrete samples taken from the different part of the structure for free chloride, free sulphate and alkalinity in the laboratory.

Alkalinity

100 g of powdered concrete is shaken with 100 cm^3 of distilled water in a conical flask in a microid flask shaker for 1 hour. The extract is then filtered through a whatman filter paper. Next, 10 cm^3 of filtered solution is titrated against N/10 standard acid solution using methyl orange indicator. The normality of the solution is calculated from the titration. For good quality concrete the normality should be around 0.04N. The fall in alkalinity indicates the passive environment close to the reinforcement ceases due to the entry of corrosive consituents.

Free chloride

Extract prepared from the powdered samples is analysed for free chloride. A 50 cc of filtered solution is taken and the chloride is estimated by silver nitrate titration using potassium chromate as indicator[6]. The tolerable limit for chloride is 0.1 per cent by weight of concrete[6]. Potentiometric titration technique is also used to detect free chloride present in the concrete sample[7].

Free sulphate

A 50 cm^3 of filtered solution is taken and the sulphate estimated as SO_3 by barium sulphate precipitation mehtod[6].

4.6.7. Analysis of reinforcement materials, grouting materials

4.6.7.1. Quantitative assessment of corrosion in mild steel reinforcements: This is done by in-situ measurement of the diameter of the exposed rods using calipers. Before making the measurements the rust and loose scale are removed from the surface. The original diameter of the rebars used at the time of construction is taken from the original drawings. From the measurements the reduction in diameter is calculated. This can be expressed as the corrosion rate in mmpy. This gives quantitatively the extent of corrosion in mild steel reinforcements[13]. Inductive magnetic measurement is used for detecting steel failures and for checking the corrosion and deterioration of reinforcing and prestressing steel[8].

4.6.7.2. Composition analysis of prestressing wire: Samples taken from the fractured tip of the strand* can be analysed for their chemical composition such as manganese, chromium, nickel and copper. This can be carried out using the Atomic Absorption Spectrophotometer[14]. But elements such as carbon, sulphur, phosphorous and silicon need specific elemental analysis. A technique of acoustic spying, where acoustic sounds are received and recorded is used for the detection of failures in cables, ropes, prestresing wires[2]. Samples of prestressing wires from fractured end are cleaned off and then subjected to metallographic examination to identify the mode of failure such as brittle fracture, stress corrosion cracking, cleavage, etc.

4.6.7.3. Analysis of cement grout samples: Cement grout samples taken from the rusted cable sheath are analysed for loss of ignition, pH, and free chloride. Loss on ignition test can be carried out as per I.S. specification[15]. If the loss on ignition is greater than 4 per cent it indicates the presence of some carbonaceous material or some pozzolana admixture, like, burnt clay powder or fly ash[14]. pH and free chloride can be estimated by the same method as described for chemical analysis of concrete samples.

4.6.8. Analysis of water: The water surrounding the structure is collected and analysed for corrosive constituents such as chloride and sulphate.

4.6.9. Structural details: Structural details of the bridge to be surveyed has to be collected for the following aspects. Location of the bridge from the sea, year of construction, year in which distress was noticed, length and width of the span, type of foundation, substructure details, superstructure details, mix ratio, cover used, etc.

* Chemical composition tests were carried out on prestressing steel of first Thane Creek Bridge, after corrosion was noticed.

4.6.10. Field inspection of submerged structure: At present, there is no quantitative information available for underwater inspection of concrete structures, therefore, it is diffcult to estimate the corrosion rate of reinforcing steel. This rebar corrosion problem is a major risk factor in offshore concrete design. A new underwater inspection technique has been developed by combining measurement of electrical potential differences caused by a corrosion current in sea water, with acoustic inspection to determine crack width and crack depth[17]. Visual observation* is carried out by means of Remote Controlled Television equipments (RCT) by drivers or by the use of submarines[18].

4.6.11. Load Tests: The last word in field survey is the load test. Load tests are usually made only after the results of other tests have not given a clear picture of the condition of the structure[19]. The test consists of applying a load equal to or greater than the design live load to see if the structure can support it without collapsing, cracking severely or deflecting excessively.

4.6.12. (a) Field survey carried out on various structures in USA: Ministry of Transportation and Communication of Ontario, USA has carried out a detailed bridge deck condition survey on two bridges. One is prestressed post-tensioned concrete bridge and another one is four span continuous reinforced concrete 'T' beam with asphalt water proof membrane[20]. From the survey the following conclusions were arrived.

(i) Top of the deck surface exhibits numerous longitudinal cracks coinciding with the voids in the deck slab.

(ii) The chloride present in the concrete sample is above the threshold value.

(iii) The bond of the asphalt membrane to the concrete is very poor.

(iv) The paint is peeling from the concrete causing severe corrosion.

(v) Concrete was not properly air-entrained causing severe delaminations and spallings.

4.6.12. (b) Field survey on parking structures in USA: Parking structures are subjected in varying degrees to ambient weather conditions causing severe deterioration in USA[21]. Unlike a bridge deck, the interior or a parking facility is not frequently rinsed with precipitation. Its exposure to chlorides may be worsened due to poor drainage of the slab surface. Deterioration usually result in spalls and delaminations in the driving surface, leakage of water through joints, cracks, spalling of the top surface and spalling of concrete on beams and superstructures.

4.6.13. Field survey on prestressed concrete bridges: Vladimir has identified the causes for corrosion of the prestressing concrete bridges[22]. Out of three bridges two of them are precast pretensioned and one is post tensioned type of bridge. Extent of distress, delamination, cover thickness measurements, chloride and petrographic analysis from core sample, metallographic examination of fractured tip of strands had been carried out for each bridge. From the studies following conclusions were arrived.

* Visual observation through under water videos or photography poses problems if water is murky. To overcome this, the habitat technique was developed and used successfully on Thane Creek Bridge. It essentially consists or a small pressure-air chamber in which a diver can sit and inspect portion by portion of substructure or foundation. The steel box with gaskets can be tailored to fit the shape of member. Even repairs can be carried out in the dry.

(i) Corrosion of prestressing steel prior to placing in concrete is caused mainly by defects in steel or by improper handling or a combination of both.

(ii) Corrosion of steel and deterioration of concrete occurred due to the presence of chlorides.

(iii) Failure of prestressing strands occurred as a result of a ductile overload due to reduction in the effective cross section caused by oxidation corrosion.

(iv) Inadequate concrete cover, cracks in the concrete overlay, improper drainage system are the major factors contributing to corrosion of reinforcement and subsequent concrete detetioration.

(v) Petrographic and permeability analysis of the concrete samples showed good quality and low permeability, hence proved these are not major factors promoting corrosion of prestressing strands.

4.6.14. Field survey carried out on various structures by CECRI, Karaikudi: The Central Electrochemical Research Institute, Karaikudi has carried out a national survey on the corrosion susceptibility of various structures such as concrete bridges, buildings, shipyard and fertilizer industries[13,14]. In addition to the methodology prescribed here, electrical and electrochemical techniques are also used to detect susceptibility of rebar corrosion. From the extensive studies, the following conclusions were arrived.

(i) Except one or two, other bridges surveyed have shown distress within about 10 years.

(ii) Atmospheric salinity prevailing at the site is independent of the distance from the sea.

(iii) The chloride and sulphate contents of the water flowing below the bridge also contribute to the aggressivity of the atmosphere.

(iv) The actual cover thickness provided at different bridge sites are very much lower than the values specified in the Standard Code of Practices. This inadequate cover thickness is the most important factor contributing to rapid penetration of corrosive constituents.

(v) Loss of alkalinity seems to be another important factor contributing to the breakdown of passivity.

4.6.15. Conclusion: Predicition of life concrete structures is essential for design, planning of maintenance and repair action. Probablistic assessment of the residual life of the structure can be made with periodic condition surveys[23]. Therefore, the use of the field survey is to know the present condition and extrapolate it to arrive at the remaining life of the structure.

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5. PROTECTIVE ASPECTS

5.1. Coating to Steel Reinforcement (Non-Prestressing Steel)

5.1.1. Introduction: Use of rusted reinforcement rods coupled with exposure to corrosive environment lead to premature failure of RCC structures. The instances of failures indicate that within 20 to 30 years, the durability of marine structure is adversely affected; i.e., the actual trouble free life of structures in aggressive environment is only about one fifth of the design life, unless some effective protective measures are taken at the initial stage itself. India with its nearly 5600 km of coastal line has to maintain many strategic reinforced and prestressed concrete structures such as bridges, offshore structures, etc. which are affected by saline action. A protective anticorrosive treatment to steel reinforcement before it is laid in concrete can guard against this chloride corrosion of steel. Currently three different types of coatings are being used. They are, galvanizing, epoxy coatings and cement based coatings.

5.1.2. Metallic coatings: There are number of metallic coating systems such as zinc, nickel, cadmium, copper, lead and tin. Metals like zinc, that are more active than iron will give sacrificial protection. This protective ability is of much value where the coating material is, likely, to be damaged and the base metal exposed[1]. Under these conditions, a galvanic couple is formed in which the more active metal dissolves and the base metal stays protected. When there is no breakdown of zinc coating, the coating behaves like a barrier to protect the underlying metal. Metallic coating like nickel coating is non-sacrificial as nickel is nobler than iron[2]. Zinc Institute had advocated the use of galvanizing and have given the specification for the same[3]. In recent years, there has been an increasing use of galvanizing as a protective method against corrosion of reinforcing steel, particularly in U.K. and U.S.A[4]. A series of evaluations conducted by the Portland Cement Association (PCA) on bridges containing galvanised reinforcement were undertaken[5,6]. For salt contaminated bridges, in Bermuda, which had been in service for 20 years, no corrosion damage was noted[7]. A large series of test results have been presented on metallic coated reinforced concrete prisms exposed at different levels in the Rance Estuary, France[8]. Galvanized reinforcing was observed to provide corrosion protection at different covers compared to black steel, although the results were not qualitative.

The performance of metallic coated steels in concrete under various conditions have been investigated[9]. It has been concluded that for application in new concrete structures both zinc and nickel coatings appeared suitable. In the case of 21 years old, "Long Bird Bridge", Bermuda, the galvanized coating had approximately 60 to 75 per cent of the original coating remaining[10]. Inspection of 3 years old, "Boca Chica Bridge" revealed that the span containing galvanized reinforcement showed no evidence of corrosion, although the chloride level was high, while the span containing black steel showed evidence of corrosion. The performance of zinc-coated reinforcing steel was examined using precracked beams exposed to intermittent salt-spray conditions[11]. The results of these experiments, which compared the performance of black and galvanized bar suggested that a zinc coating was measurably beneficial in arresting corrosion in concrete containing cracks of upto 0.3mm width in comparison with plain bar.

After a critical appraisal of the technical literature dealing with thin wall reinforced concrete exposed in marine conditions for the period covering the past 75 years, it was concluded that the use of galvanized reinforcement for this wall reinforced concrete floating piers would be beneficial[12]. Tests were run to determine, changes experienced in the zinc layer on reinforcing steel in concrete and whether galvanizing leads to improved protection against corrosion in carbonated concrete[13]. Concrete prisms reinforced with galvanized steel bar were exposed to various relative humidities for different periods. It was found that zinc was consumed mainly in the first seven days, but at 100 per cent relative humidity for 2 years exposure, there was additional zinc removal. To determine the effect of carbonation, series of tests were run using concrete prisms with reinforced deformed bars. Test specimen were stored for one month in the laboratory, then carbonated with a 3 per cent CO_2 atmosphere. It has been reported that immediately after carbonation of the concrete, the ungalvanized bars began to rust, and by two years had firmly adhering rust crusts, but the galvanized bars showed no signs of rust after two years of exposure.

Corrosion characteristics of galvanized reinforcing bars has been studied[14]. Concrete beams with galvanized reinforcement bars were subjected to accelerated corrosion tests in a 3 per cent NaCl solution at a constant current density of 100mA/m² using impressed current. Experiment result showed that the galvanized steel bar improved the corrosion resistance of reinforcement embedded in concrete at depths of cover from 10mm and 20mm when subjected to impressed currents. The corrosion resistance of the galvanized bar in a natural marine environment was greater than that of the black steel. It has been reported that hot dip galvanizing did not affect the mechanical properties of the hot rolled bars.

Laboratory and exposure site trials have been carried out on a number of metallic coating systems for reinforcing steel[15]. It has been reported that the zinc coatings on steel delayed the on-set of corrosion. The corrosion of zinc in hydroxide/chloride solutions were examined using potentiodynamic polarization techniques[16]. It has been concluded that chloride attack on zinc was considerably reduced in the presence of hydroxide.

5.1.2.1. Performance reports on galvanizing: The behaviour of zinc and galvanized steel in chloride-bearing calcium hydroxide solutions have been examined using full and partial immersion techniques[17]. The results of these tests and similar ones on mild steel were compared and it was concluded that under conditions of inhomogeneous embedment in concrete, the performance of galvanized steel was questionable. A series of long-term exposure programme reported that galvanized coating delay the onset of corrosion in marine environments, but do not prevent it completely[18].

Galvanized reinforcements from section of Bermuda Air Terminal torn down for remodelling showed no rusting after 13 years of service[19]. However, occasional cracking of galvanizing was observed when rod was bent too sharply. 15 years duration exposure tests at the Building Research Station showed that within the mortar, galvanized steel was only slightly attacked but loss of zinc was observed at mortar-steel-air interface and additional protection with an organic coating was recommended[20]. It has been stated that galvanized steel showed considerable resistance to pitting corrosion, but much of the zinc had itself corroded in the most corrosive marine conditions[21]. Three years exposure data collected by Building Research Establishment, U.K. showed significant loss of zinc on the galvanized steel at both 10mm and 20mm covers.

A eleven year exposure programme in marine environment revealed that the zinc coating suffered corrosion by 2 to 3 mil. loss in thickness of the original zinc layer[22]. It has been observed that the unfavourable influence of chloride and carbonation is less on galvanized steel but cautioned that being a complex problem[23]. A three year corrosion research programme on protective systems for "New prestressed and substructure concrete" has been carried out[24]. One objective of this programme was to establish the performance of galvanized bars used throughout the concrete and to establish the difference in corrosion resistance, when galvanized bars and normal bars are used in same construction. This has been done by two laboratory studies, which included a 48-week cyclic wet and dry salt water exposure and a year long study with cyclic salt water exposure on full size concrete specimen. It has been concluded that galvanized bars, when used exclusively within these various specimen, developed very low corrosion currents. Zinc corrosion by-products and extremely small amounts of steel corrosion by-products eventually developed on the galvanized bars. It has also been reported that galvanized bars and grey bars, when used in the same specimen developed very large and sustained corrosion currents. The measured corrosion activity was similar to specimen containing only unprotected grey bars. Zinc corrosion by-products and small amounts of steel corrosion by-products were observed on the galvanized bars, which indicated that the zinc coating was corroded away in small, localized areas.

The behaviour of galvanized versus black steel reinforcement in lollipop specimen partially immersed in saturated NaCl solution has been investigated[25]. Data from this series indicated that there was no benefit from galvanizing and that corrosion began at roughly the same time for both types of steel. Further, for low water/cement ratio concrete, galvanized steel specimen cracked earlier than black steel specimen. The effect of "tidal flows" of seawater has been examined on concrete prisms containing both black and galvanized steel under laborator/ conditons[26]. The results showed that both the black steel and galvanized steel reinforcement initiated cracking at the same time after immersion, but that following initiation more severe cracking was observed on the galvanized steel. There was a tendency for the galvanized steel to generally retard the cracking, but once the cracking occurred it tended to be more severe on the galvanized steel specimen. The behaviour of zinc-coated and nickel-coated steel in simulated concrete (calcium-hydroxide/sodium-chloride environments has been investigated[2]. It has been found that zinc showed quite severe corrosion in these tests, whereas nickel did not.

Two major exposure site programmes were undertaken to provide comparative information on the corrosion susceptibility of steel in concrete[27]. The main objectives of these two programmes have been to provide information on galvanized bar in good quality as well as poor quality chloride-free concrete and to evaluate the effect of chloride on the performance of galvanized bar. In the first series, concrete specimen which were made originally without deliberate addition of chloride has been under test for 14 years. The second series (made up of concrete beam and prisms) has been under test for five years, but in these specimen a range of calcium chloride additions has been made to accelerate the corrosion effect. From these tests, it has been concluded that at high chloride levels (1.9 per cent and above), serious corrosion occurred on both galvanized and plain steel. Galvanizing provides some delay in the onset of corrosion-induced cracking of the cover, but the delay ranging between six months and one year. In the carbonated chloride free concrete, the application of a galvanizing coating to the steel measurably delaying the onset of cracking. It has been concluded that under certain circumstances the use of galvanized reinforcement can considerably inhibit the corrosion process.

The corrosion preventive effect of galvanized steel reinforcement has been described. Precracked concrete specimen with galvanized reinforcement were subjected to cyclic environmental tests for 160 days in the laboratory. It has been found that in all the galvanized steels white rust of zinc and spot rusts of steel were noted on the steel bars at the cracked location. It has been reported that the steels having the chromate treatment and the amalgam plating seemed to be ineffective. It has been concluded that galvanizing is not always satisfactory at the splash zone[28].

5.1.2.2. Findings of BRE, UK: Building Research Establishment (BRE) has studied the performance of galvanized steel[29]. It was found that when concrete contained high levels of chloride, 2 per cent or more, cracking was not significantly different whether galvanized or bare steel reinforcements were used. It was considered that the small delay before the onset of cracking could make the use of galvanizing steel not economically worthwhile.

Corrosion performance of different concrete reinforcement products such as mild steel, galvanized steel, stainless steel, fusion bonded epoxy coated reinforcements have been investigated[30]. In that study, concrete specimen with galvanized bars were exposed to the atmosphere for five years. After exposure period, a comprehensive examination was undertaken by visual and electrochemical methods. It has been concluded that galvanized steel offers increased corrosion protection in carbonated concrete uncontaminated by chloride. But in the chloride contaminated concrete severe corrosion was observed. It has been reported that, corrosion was observed to be more severe in concrete contaminated with NaCl than in those with $CaCl_2$.

5.1.2.3. Findings of FHWA: The corrosion resistance of galvanized rebars in concrete has been studied by Federal Highway Administration (FHWA)[31]. It has been reported that, the galvanized rebar in concrete containing chloride was subjected to the same type of macroscopic corrosion as black steel. Both the long-term exposure data and the rate of corrosion data indicated that no benefit and perhaps a slight detriment, when all the rebar was galvanized and a 0.40 water-cement ratio concrete was used. Studies on 0.50 water-cement ratio concrete indicated that the combination of chloride contaminated concrete and black steel in chloride free concrete (electrically coupled) was particularly bad. The rate of corrosion was more than twice as high for situation than for the equal concrete in which all black reinforcing steel was used.

But in the case of 0.50 water-cement ratio, both the long-term exposure data and the rate of corrosion data indicated a benefit when all the rebars were galvanized. Significant corrosion-induced cracking occurred on the black steel slabs with concrete at this 0.50 water-cement ratio, but not on the galvanised rebar slabs. Rate of corrosion data indicated that about a 34 per cent reduction in macrocell corrosion current and a 22 per cent reduction in metal loss. But in the 0.40 water/cement ratio concrete an average of 86 per cent reduction in both corrosion current and metal loss has been obtained.

A panel of three experts surveyed the state-of-the-art on the use and performance of galvanized steel in an adverse environment. It has been concluded that there was not a high probability that the use of galvanizing reinforcing steel as the protective system and normal construction practice will provide long-term protection to concrete members constructed in areas where they were exposed to deicing salt or a coastal environment. It has also been confirmed that the galvanizing delays the onset of corrosion-induced distress, but not prevent it.

5.1.2.4. Bond strength: Conflicting reports have been presented on the bond strength property of the galvanized steel. It was found that the bond strength was reduced when rusted reinforcement was replaced by galvanised reinforcement[32]. Loss of bond strength with galvanized reinforcement has been reported[33]. As a contrary, it has been reported that the bond strength improved slightly when galvanized steel reinforcement is used[34]. It has also been reported that for plain bars, the bond strength of galvanised reinforcement was 30 to 50 per cent greater than for similar ungalvanized reinforcement. It has also been concluded that for deformed bars, the bond strength of galvanized bars was the same as that for similar ungalvanized bars.

When zinc corrodes at a very high alkaline solutions, the hydrogen gas is liberated. This liberated gas reduces the bonding between zinc and steel. To improve the bond strength, chromate treatment to galvanized bars has been suggested to raise the hydrogen over voltage and decreasing the gas evaluation[35]. A study on passivation of galvanised reinforcement by inhibitor anions has been made[36]. Zinc in saturated calcium hydroxide solution was found to passivate in the presence of either sodium chromate or chromic oxide. In both cases, chromate ions were reduced instead of water so that there was no evolution of hydrogen. The passive film consisted of zinc chromate and chromic oxide, but calcium hydroxozincate was also produced, which assisted in passivating the zinc surface. In the case of chromic oxide a smaller concentration of chromate ions was present and passivated the zinc to a lesser extent. It has been concluded that 70 ppm of sodium chromate was sufficient to passivate the zinc, whereas, atleast 300 ppm of chromic oxide was necessary to achieve the same degree of passivation.

Metallic coatings, generally of zinc, have been used with satisfactory results to provide sacrificial protection to steel reinforcement under normal conditions of atmospheric corrosion, but in the presence of chlorides, the zinc suffers increased attack[37]. Although reports on the ability of galvanized zinc coatings to protect steel against chloride corrosion are conflicting, the consensus of opinion is that if conditions are sufficiently severe for the steel to be corroded, the zinc will rapidly be consumed sacrificially, and deterioration of the concrete will only be delayed. But it will not be prevented.

5.1.2.5. Work carried out by CECRI, Karaikudi: The passivity of zinc in 0.04N NaOH chloride system has been investigated by different electrochemical techniques, such as, peak potential technique, potentiostatic technique and galvanost:.tic technique[38]. It has been concluded that zinc is passive in 0.04N NaOH solutions only upto a concentration of 200 ppm of chloride and zinc has the lowest tolerable limit. It was also found that under aggressive marine conditions, the concentration of chloride ions inside concrete surrounding the steel reinforcement is quite considerable and hence the passivity of zinc may be destroyed quickly. Once the passivity is broken, the galvanic effect accelerates the dissolution of zinc. It is seen from the cost evaluation of CECRI that cost of zinc coating is cheaper when compared to epoxy coating, but it is costlier than inhibited cement slurry coating developed by CECRI[39].

5.1.3. Non-metallic coating: Earlier researchers evaluated a few non-metallic coatings and suggested that an epoxy coal tar type of coating could be considered as a protective coating for reinforcing steel[40,41]. A study was undertaken to ascertain the feasibility of using organic coatings, especially epoxies, to protect steel reinforcing bars embedded in concrete of bridge decks from rapid corrosion[42]. 47 different coatings were evaluated, of which 36 were epoxy coatings. Both powder and liquid epoxy systems have been studied. Different tests, such as, chemical resistance, film integrity, physical tests, electrochemical measurements and bond strength tests were carried out on the coated steel specimen. From these tests, it has been concluded that properly formulated and properly applied powder epoxy coatings should adequately protect the steel reinforcement of concrete bridge decks from rapid corrosion. It has been reported that the powder epoxies have better overall properties as barrier coatings for reinforcing bars than the tested liquid epoxies.

Both powder and liquid coated reinforcing bars, with film thicknesses below 10 mils have acceptable bond strengths in concrete. It has been stated that polyvinyl chloride coated bars should not be used as reinforcement in concrete bridge decks, because they do not develop acceptable bond strength in concrete.

Steel specimen with 8 kinds of epoxy resin (5 of powder type and 3 of liquid type) coatings in thickness of 80, 150 and 250 μm have been subjected to accelerated corrosion tests[43]. Precracked concrete specimen with epoxy coated reinforcing steel were subjected to cyclic environmental tests for 160 days in the laboratory. For field tests, the test specimen were exposed to marine environments. From this study, it has been concluded that powder epoxy gives the best protection. It has been reported that for effective protection, a coating thickness of 150 μm or greater is required. But for good bond to concrete, the thickness is to be preferably less than 150 μm. Hence, the coating thickness of 150 μm has been considered as optimum thickness. It has also been concluded that the liquid type tar epoxy coating is not satisfactory in its protective performance or for bond to concrete.

A detailed study on the long-term corrosion resistance of epoxy coated reinforcing bars compared with plain uncoated and galvanized bars has been presented[44]. Three different thickness of epoxy coatings varying from 100 to 300 μm were used in this study. The tests were carried out on centrally reinforced concrete prisms with variable cover and pre-formed cracks of maximum width of 0.10 to 0.25mm. The specimen under stress were subjected to accelerated corrosion tests and marine exposure tests in a tidal zone upto two years. From the test results, it has been concluded that the uncoated bar showed extensive corrosion even with a cover of 70mm. The galvanized bars have also shown signs of corrosion with 40 to 70mm cover. But the epoxy coated bars with 200 to 300 μm thickness remained unaffected and retained all the original properties of the coating even with as small a cover as 20 mm. It has been reported that although galvanized bars were superior to the untreated bars after 2 years exposure tests, the galvanizing did not provide complete protection against pitting. It has also been concluded that with a coating thickness of 200 μm, epoxy coating protected the bar substrate well against chloride attack and corrosion, and the film itself remained intact, irrespective of depth of concrete cover. It has been suggested that epoxy coating can provide effective protection against corrosion for a long time even when chloride ions directly reach the metal surface of the bar.

A three year corrosion project on "Protective systems for new prestressed and substructure concrete" has been carried out in order to study the corrosion resistance property of the epoxy-coated bars in precast bridge decks[23]. This has been done by two laboratory studies, which included a 48 week cyclic wet and dry salt water exposure and year-long study with cyclic salt water exposure on full size concrete specimen.

From this study, it has been concluded that fusion bonded epoxy-coated bars were corrosion free at all water-cement ratios and all clear covers. These results showed that when minimum clear cover is allowed, the epoxy coated steel provides for a corrosion resistant reinforcing material. However, based upon chloride ingress characteristics into concrete and the possibility of imperfections in the coatings, it has been recommended that water-cement ratio of concrete should be 0.44 or less to further reduce the risk factor. It has been suggested that the use of the effective concrete sealers can provide significant additional corrosion protection for members containing coated steels.

5.1.3.1. JSCE recommendations: Recommendations for design and construction of concrete structures using epoxy-coated reinforcing steel bars have been drawn by Japan Society of Civil Engineers (JSCE)[45]. The main comments are as follows:

(a) Holiday

Existence of holidays in a coated bar reduces its corrosion resistance and the number of such defects can be used as a criterion for the suitability of coated bars for use. Though, it is desirable that the number of such defects to be zero, it is recommended that the number of holidays per metre of a bar shall not exceed 5 for D19 or small bars and 8 for D22 or larger bars, when tested with a 1000V D.C. holiday detector.

(b) Handling, Storage and Transporting of Coated Bar at the Place of Manufacture

Since the coating can be easily damaged by sharp edges, etc. care shall be exercised when transporting and storing coated bars. Damage is likely to occur to the coating if coated bars are stacked on more than 6 ties.

Since the coating of the coated bars deteriorates when they are exposed to the ultraviolet rays, they should not be stored in a location where the sunshine falls directly upon them. If the coating deteriorates due to the direct sun shine, in many cases the bendability and the impact resistance will be reduced.

(c) Reinforcement Work

Coated bars are subject to coating damage caused by contact with other bars during transportation, impact during unloading abrasion of wire ropes, extreme bending deformation, etc. To prevent the coated bars from such damages, the bars shall be securely bound together using a buffer material and it is desirable that the bent bars be supplied with a canvas cover, etc. so as to prevent the coating of the bars from being damaged due to the contact with the other bars. When loading and unloading coated bars, it is preferable to lift them using nylon sling, at either two or three points to avoid extreme bending deformation. If damage occurs to the coating during transportation, it must be repaired before rusting occurs.

It is preferable to store the coated bars on buffer material placed at suitable intervals, instead of placing them directly on the ground, to avoid damaging the coating and to supply them with a canvas cover to avoid exposure to direct sunlight. It is not desirable to store coated bars by piling them on top of one another. If they are to be stored by piling, buffer materials, such as, wood, rubber, jute sack, etc. should be used to separate the layers. However, the number of layers allowed shall be limited to a maximum of five, even for large diameter bars.

(d) Cutting and Bending of Coated Bars

Care must be taken not to damage the coating when cutting and bending coated bars. For this reason, at the point of drive roll and back-up barrel of the bending machine, use of urethane roller, lined with material that will not damage the coating, is recommended. The coating of coated bars will resist cracking during bending above a temperature of 5°C. The properties of coated bars will not change up to about 200°C considering the manufacturing process. Therefore, it was determined that the cutting and bending are, in principle, to be performed at an ambient temperature of more than 5°C. If the cutting and bending must be performed at an ambient temperature of less than 5°C, it is desirable to take measures to raise the temperature of the coated bar. Since coated bars are coated after blasting steel bars, any damaged portion of the coating tends to rust. For this reason, it is suggested that the damaged portion be repaired by coating before rusting occurs.

(e) Assembly of Coated Bars

As the coating of the coated bar is susceptible to damage from impacts, when the bars are dragged or dropped, they may sustain damage that reaches the base metal. When being assembled, coated bars shall be handled with care. For the ties of the coated bars, use of annealed wire with diameter of larger than 0.9mm with vinyl insulation is recommended. When vinyl covered iron wire is used, solid assembly of coated bars may become difficult to achieve, due to the slippery surface of the coating. For this reason, it is suggested that the bars be tied in a cross or at a large number of places. In the case that the bars will be exposed to direct sun shine for more than 3 to 6 months after assembly, the bending workability of the coating may be reduced, When performing rebending, etc. the bars shall be inspected for damage to the coating near the bending location and repairs should be carried out.

(f) General Provisions

Since the rebars are epoxy-coated to protect them from corrosion, it is necessary that the placing of concrete be performed with attention paid to prevent the coating from being damaged. Concrete should be placed and thoroughly consolidated so as not to reduce the bond strength of the coated rebars. For this reason, the use of an internal vibrator coated with urethane is recommended.

(g) Construction Joint

If mortar or concrete has adhered to the coated bars at the construction joint, it shall be removed by water jets or wiping it with waste rags before it hardens, care is being taken not to damage the coating. Sand blasting or wire brushes should not be used to clear a very hardened mortar or concrete adhered to the coated bars.

5.1.3.2. Findings of FHWA: Federal Highway Administration has studied the corrosion performance of epoxy coated reinforcing steel[30,46-50]. Results of the reinforced beams studies (Partial immersion of over 100 beam containing rebars with the coating in various conditions, in saturated NaCl solution) initiated in early 1974 showed 100 per cent of the black steel reinforced slabs were cracked at the end of 34 months but only 49 per cent and 5 per cent were observed in cracked condition at the end of 80 months in epoxy coated (not to specs) and epoxy coated (meets specs) slabs respectively. The corrosion currents in the various specimen were non-destructively measured using the 3 electrode linear polarisation device. It has been reported that the average corrosion current in black steel, epoxy coated (not to specs) and epoxy coated (meets specs) were 399, 32.4 and 13 μA respectively.

The above study applies to the situation in which all the reinforcing steel is epoxy-coated or the case when epoxy-coated reinforcing steel is electrically isolated from any black steel in close proximity. Earlier, results showed a potential problem may exist when damaged epoxy-coated rebars in chloride-contaminated concrete are electrically coupled to large quantities of black steel in chloride-free concrete. In this case, a macro-corrosion cell with a large cathode (black steel) could develop and drive corrosion rapidly at damaged areas; whereas, for the cases of all bars epoxy coated or electrically isolated epoxy-coated bars, both the anode and the cathode must be set-up on the damaged area, and because of the cathode surface area is small, the rate c˜ corrosion will be low.

It has been stated that concrete structures constructed in salt environments using epoxy-coated reinforcing steel have many times more resistant to corrosion-induced concrete damage than those constructed with uncoated rebar and conventional concrete, eventhough the epoxy coated rebar did not meet either the holiday requirement or the bent test as specified by AASHTO M284 or ASTM D3963. It has been reported that epoxy-coated reinforcing steel apears to be effective because its use increases the electrical resistance between the macro anode and macro cathode. Also, when all the rebar is epoxy coated, macro-cathodic polarisation occurs due to the inability of the small cathodic areas to reduce significant oxygen, further minimizing the corrosion rate at bare areas in the salty concrete.

The effect of black/epoxy bar coupling has been examined by an outdoor slab study. The results indicated that electrical coupling of epoxy-coated top mat bars with black steel bottom mats can induce accelerated corrosion. But it appears to be a practical concern only when the total visibly bare area exceeds 0.24 per cent. The best situation recommended is one in which all the bars are epoxy coated or the coated bars are electrically isolated from other metal in the structure. But, even when non-specification epoxy coated bars in salty concrete were all electrically coupled to large amounts of uncoated steel in salt-free concrete, total metal consumed was not more than 1/12 that for the situation of all uncoated bars.

The performance of either protective system, non-specification epoxy-coated reinforcing steel or calcium nitrite were compared at a chloride content of about 9 kg/m^3. It has been concluded that calcium nitrite provides a level of protection closest to that provided by coating only the top mat rebar. Calcium nitrite appears to be effective primarily because it does not allow a large electrical potential difference to develop between areas of steel, which, without the nitrite, would be highly anodic and cathodic to each other.

5.1.3.3. Performance reports on epoxy coating. Data on half cell potential readings on epoxy coated and galvanised rebar bridge decks over a ten year period has been collected[51]. An increase of corrosion activity has been recorded over a ten year period, as presented in the cumulative frequency plots of the half-cell potentials in both the epoxy rebar and galvanized rebar bridge decks. It has been concluded that it is not known whether these increases in potential are real and whether corrosion is actually occurring in the epoxy rebars.

It has been reported that epoxy coated reinforcing steel embedded in concrete in Florida bridge, USA, has experienced severe corrosion[52]. The effect of environmental chemistry and electrode potential on coating disbondment has been investigated by laboratory exposures under potentiostatic conditions.

Corrosion behaviour of epoxy coated reinforcing steel in concrete exposed to a simulated marine environment has been studied[53]. The investigation was set to determine the effect of different surface and mechanical conditons on the corrosion behaviour of reinforcing steel; namely, the degree of bonding, epoxy damage surface conditions of the steel, presence of cracks in concrete and the manufacturing sequences. In this investigation, the experimental techniques, such as, electrochemical impedance, open-circuit potential monitoring and direct examination after exposure have been used. From this study, it has been concluded that the epoxy coated rebars can develop active corrosion conditons beginning at times comparable to those experienced in bare rebar. It has been reported that after 300 days of exposure, the corrosion rate in bent epoxy coated steel appeared to be an order of magnitude lower than bare bent steel. Fabrication and bending results in loss of adherence of the epoxy and corrosion was observed in the resulting debonded area. It has also been reported that cracking of the concrete cover appears to accelerate the initiation of active corrosion, but this is not yet enough as to its long term effect on the corrosion of epoxy coated bars.

5.1.3.4. Finding of BRE: The performance of fusion bonded epoxy coated steel reinforcement and its potential durability have been assessed in a natural exposure trial carried out by the Building Research Establishment (BRE)[29]. Concrete specimen, such as, prisms and slabs with epoxy coated specimen were cast in two different mixes containing different amount of chlorides. After five years atmospheric exposure a comprehensive examination of all of the samples was undertaken by visual and electrochemical methods.

It has been reported that small nodules of brown corrosion product were found on epoxy-coated bars embedded in concrete. On removal, a pit containing brown corrosion product was also found. When removing epoxy-coated reinforcement from the specimen, it was found that it debonded from the surrounding concrete readily and clearly. This suggests that the epoxy-concrete adhesion bond is weaker than that between the metal surfaces and concrete. Defects in the epoxy coating of the bent bars were observed in the slabs with this reinforcement. In chloride contaminated concrete extensive underfilm corrosion around the bends gave black corrosion products. Some small nodules were also found at an early stage of pitting of reinforcement embedded in the chloride contaminated concrete.

The potential monitoring study indicated that coated bars showed more negative potential values. This could be taken to indicate more severe corrosion. However, post exposure examination of the bars indicated that they are essentially in good condition, with corrosion limited to isolated pitting attack and some underfilm attack.

It has been concluded that epoxy coating of steel provides a significant reduction in the rate of deterioration of reinforced concrete, although in concretes containing the highest level (3.9 per cent chloride ions by weight of cement) underfilm corrosion spreads from damaged areas.

Bond strength tests on epoxy-coated bars have been reported[54]. These indicated that cold worked bars conforming to BS: 4461 with a spiral or screw rib pattern will give unacceptably low bond strengths. Hot rolled bars with a Chevron rib pattern satisfied the bond strength requirement of BS: 4449. It has been stated that care must be exercised when epoxy-coated reinforcement is used in structural applications.

5.1.3.5. Work carried out by CECRI: Comparative performance of powder epoxy coatings (barrier type) and inhibited cement slurry coating (passive type) systems have been evaluated as per ASTM Standards[55]. It was found that different powder epoxy formulations gave protection to different extents. However, most of the epoxy powder systems were found to have poor alkali resistance, which is very important from the point of view of their use in concrete structures.

Galvanic current measurements in OH-Cl system showed that in the case of any defects in the coating more galvanic corrosion is likely to take place in powder epoxy systems compared to cement slurry system. The performance of epoxy coating to steel has been examined using impressed voltage test as per ASTM A775/A775 M-84[56]. A potential of 2V was impressed in between the coated rods for a period of 60 minutes. After the test period, undercuttings and rust products were observed on some of the epoxy coated bars. It has been concluded that some of the epoxy coating systems fails in the impressed voltage test specified in ASTM A775/A775 M-84*.

Though, the powder epoxy coating offer considerable corrosion protection to steel rebars, there are several limitations associated with their applications[57]. As the coating is a factory process and involves high temperature, the process control becomes vital and extra costs arise due to transportation to factory and to site, power, equipment, etc. Handling the coated rebars also becomes crucial and has to be monitored. In the context of price structure in the USA, the epoxy coated bars cost about 50 per cent more than of the raw steel[58]. In India, it may be double.

5.1.4. Cement based coating: Since the steel reinforcement embedded in concrete is surrounded by an alkaline medium a coating based on cement is expected to be more compatible. Cement coating is a passivating type of coating and hence may have higher tolerance towards defects. Because of the surrounding concrete, which is again alkaline, galvanic effect is likely to be less pronounced.

It has been recommended that a priming coat of pure portland cement slurry over the reinforcement may be given immediately followed by the concrete[59-60]. This method, if carried out properly, will create a continuous cement skin which may do much to protect reinforcement from corrosion without affecting the cement-to-steel bond. It has been reported that this method will not interfere with the physical properties of concrete.

Various passivating treatment for reinforcing steel have been suggested, such as, picking in hydrochloric acid followed by treatment with phosphoric acid[61] and treatment with a hydrolysable silicate or hydrated silica[62]. Simple preliminary coating of steel with a dense mortar is recommended to counteract acid fumes[63]. The application of coatings of cement containing paints with water proofing admixtures[64] or of slurries of lime and cement with casein or bone glue has also been recommended as an anticorrosive measure[65,66].

Bitumen paints were said to protect reinforcement from calcium chloride in concrete, but some paints of this type prevent the formation of bond between the steel and concrete[67]. Combinations of inhibiting agents in cement slurries were also proposed, notably sodium dichromate[68], sodium carbonate, sodium phosphate[69], sodium benzoate[70,71] and also barium chromate[72]. But all these methods do not appear to have made much headway either because of their doubtful field performance or of their adverse effect on bonding.

5.1.4.1. Work carried out by CECRI: From the point of view of economy and efficiency, Central Electrochemical Research Institute, Karaikudi, India, has developed a coating based on portland cement slurry admixed with corrosion inhibitors[73]. The coating is made impermeable to salts by a sealing treatment. The various properties of this newly developed coating, viz., bonding strength, corrosion resistance, behaviour under load, etc. have been reported[74].

* Recently, the Bureau of Indian Standards (BIS) has published I.S. : 13620 - 1993 on Fusion Bonded Epoxy Coated Reinforcing Bars - Specification.

The bonding strength between the inhibited cement slurry coated reinforcement and concrete was tested by conducting standard pull out tests in a 30 tonne Avery testing machine. It has been concluded that the coating does not adversely affect the bonding.

The coated rebars were subjected to standard salt spray test, immersion tests, field exposure studies and steam curing test. It has been reported that this inhibiting and sealed portland cement coating is able to offer very satisfactory protection to steel reinforcement.

The potential-time behaviour of the cement coated rebar was compared with uncoated rebar in 3.5 per cent NaCl solution for a period of 30 days. The surface condition was also examined by visual observation study. It has been observed that the inhibited cement slurry coating keeps the steel nobler by more than 100 mV. Visual observations showed that because of the high alkalinity developed by the inhibitor admixture and because of the imperviousness imparted by the sealing treatment, the steel maintained its passive condition even in the presence of 3.5 per cent of NaCl solution.

The tolerable limit for chloride in 0.04N NaOH medium was studied by both anodic polarisation technique and peak potential technique (Fig. 5.1). These studies showed that the uncoated steel rebars have a low tolerable limit, whereas, the inhibited cement slurry coated rebars have tolerated more than 10,000 ppm of chloride in 0.04N NaOH solution.

Inhibited cement slurry has been evaluated using precracked cantilever model slab technique. It has been reported that the inhibited cement slurry coating is able to increase the durability by a factor of 25 to 35 even under this cracked model slab studies with a crack width of 0.3 mm.

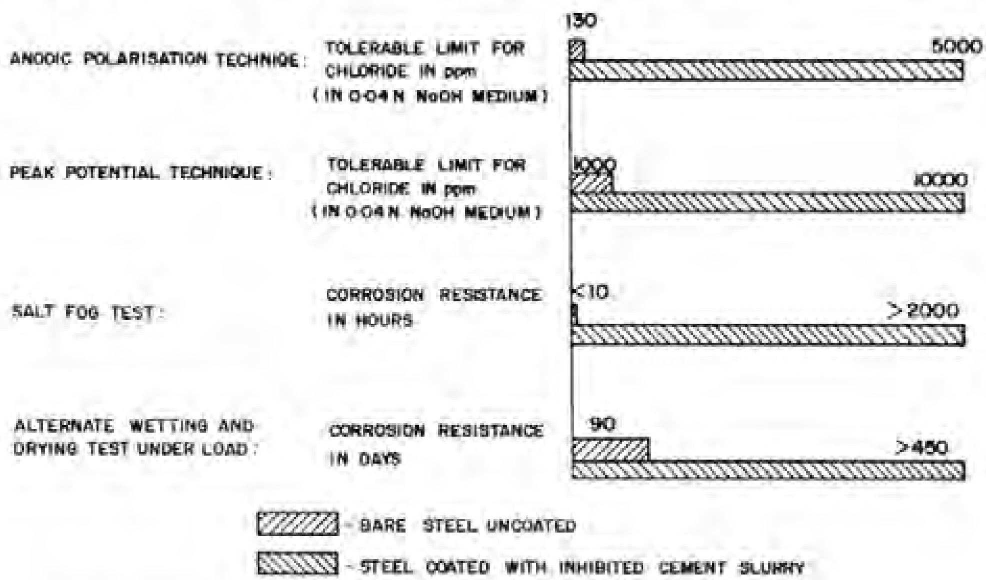

Fig. 5.1. Comparative Performance of Inhibited Cement Slurry Coating in Laboratory Accelerated Corrosion Tests

Concrete cubes with steel specimen were simultaneously subjected to alternate wetting and drying tests under uncracked conditions at the corrosion testing station, Mandapam Camp, Tamilnadu, one of the most corrosive locations in the world. These studies showed a corrosion rate of 0.271 mmpy for uncoated specimen against 0.004 mmpy for inhibited cement slurry coated specimen leading to a durability factor around 68. Thus, the durability factor is higher under uncracked condition compared to cracked condition.

Comparative performance of powder epoxy coatings (barrier type) and inhibited cement slurry coating (passive type) systems have been evaluated as per ASTM standards[36]. Cement slurry coating was found to have adequate chemical resistance against alkali, chloride and distilled water. Galvanic current measurements in OH-Cl system has been showed that in case of any defects in the coating, more galvanic corrosion is likely to take place in powder epoxy systems compared to cement slurry systems (Fig. 5.2). It has been concluded that the passivating cement slurry system appears to be preferable over powder epoxy system for specific use in concrete medium. This is particularly so, considering that the CECRI treatment is intended to be used mainly to protect the bars from corrosion till their embedment in concrete. Once the bars are in the passive concrete medium, the bars are protected by the quality and impermeability of properly compacted concrete.

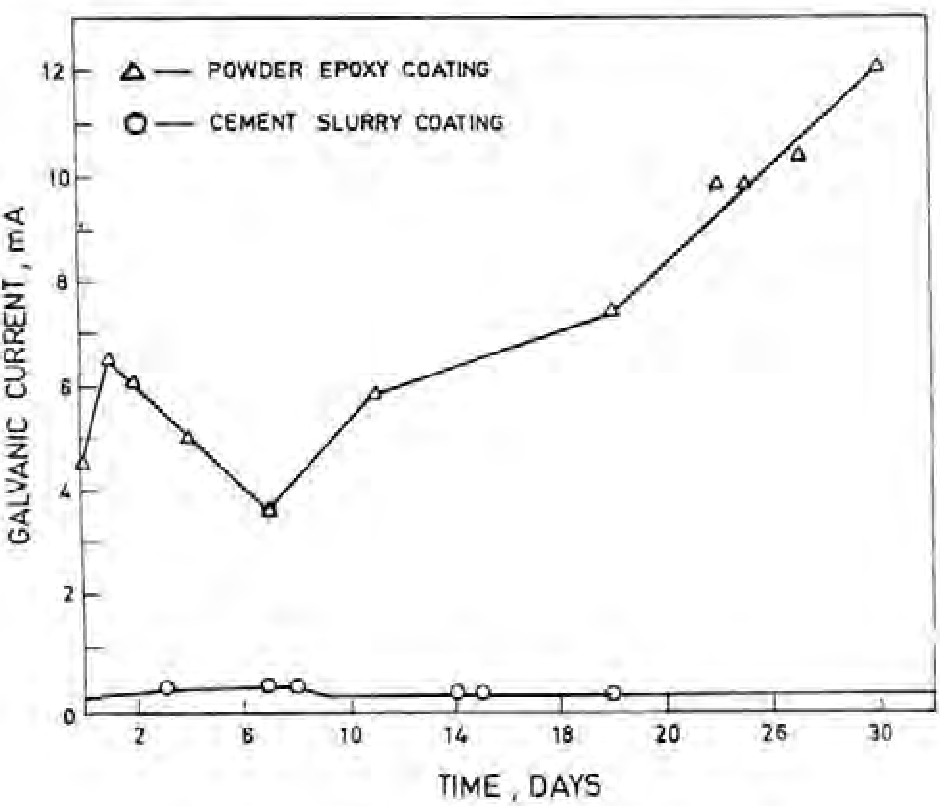

Fig. 5.2. Comparison of Galvanic Corrosion in Impressed Voltage Test

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5.2. Coating to Prestressing Steel

Corrosion resistance and bonding properties of two types of coatings have been investigated[1]. Cement slurry and coal tar modified epoxy (XP 3430 CIBA-GEIGY) were applied on different types of prestressing steels. Pre and post-tensioned beams and concrete prisms containing different types of high-strength wires and concrete qualities were prepared and subjected to the attack of high percentage of calcium chloride ($CaCl_2$) or a corrosive environment containing mainly sulfur-dioxide (SO_2).

It has been concluded from this study that the protective coatings having desirable anticorrosive qualities, might in certain circumstances affect the bonding strength of the prestressing strands to the concrete. It has been reported that the cement slurry provides an excellent bonding medium for straight wires. It has also been reported that cement slurry provides a good protective layer for the steel. In the post-tensioned system it has been concluded that resin coating provides a good protection against corrosion.

Recent applications and major properties of a newly developed epoxy coated seven-wire strand suitable for prestressed concrete in a corrosive environment has been described[2]. Two grades of corrosion resistant strands with 30 mil (0.76mm) coating were reported. Epoxy coated strand was designed for use with end anchors where bond to concrete was not required. When encased in concrete, this strand does not pull easily through the concrete, but it does not develop enough resistance to provide bond between strand and concrete.

During the production process of bond controlled epoxy coated steel, grit was thoroughly embedded into the outer surface of the coating, before the coating was subjected to the final treatment, which hardens it. Once the coating has been hardened, the grit provides an excellent bond between the coated strand and concrete.

In the concluding remarks, it has been reported that the seven wire strand for prestressed concrete covered with this epoxy coating makes it as corrosion resistance as epoxy coated reinforcing bars. The coating was thick enough and ductile enough to contnue to provide full protection, when the strand was held at the high tension normally applied to prestressing strand and when the strand was bent around a relatively sharp radius.

Galvanizing has been applied to prestressing wire and strand for corrossion protection[3,4]. However, zinc on the galvanized wire reacts with the alkali ingredients of portland cement and evolves hydrogen. Comparative tests for hydrogen susceptibility on bare wire, galvanized wire and redrawn galvanised wire used for prestressed concrete have been described[5]. From this study, it has been concluded that galvanized steel wire helps to inhibit hydrogen embrittlement of prestressing steel. It has also been concluded that the part of the hydrogen, generated as a result of the reaction of zinc on galvanized wire with portland cement is absorbed by zinc and does not penetrate into the steel. It has been reported that cold working process can cause cracks in the zinc layer of the galvanized steel and leads to the direct reaction of the base metal with the corrosion solution. Thus, hydrogen can enter into the steel lattice and cause hydrogen embrittlement problem.

A three year corrosion research project on "Protective systems for new prestressed and substructure concrete" has been carried out in order to study the corrosion resistance property of the epoxy-coated prestressing strands in precast concrete construction[6]. This has been done by two laboratory studies which included a 48-week cyclic wet and dry salt water exposure and year-long study with cyclic salt water exposure on full-size concrete specimen.

It has been concluded that the fusion bonded epoxy coated prestressing strands does not develop any corrosion activity even with 2.5 cm cover. The epoxy coated strands produces very high internal electrical resistance due to the properties of the epoxy-coating. It has been suggested that the use of effective concrete surface sealers can provide significant additional corrosion protection for members containing coated steels.

It has been reported that bituminous and metallic paints can be applied to prestressing tendons[7]. These coatings are unreliable for strands because of the difficulties involved in providing a uniform coating and subject to damage during handling. It has been recommended that they can be used only to inhibit the corrosion of stored tendons before use.

A tar-like coating was initially applied to the tendons in a floating bridge over seawater in USA[8,9]. It has been found that this coating did not provide satisfactory protection against corrosion.

In Canada, the tendons of the post-tensioned systems have been protected by asphalt protective coatings to prevent corrosion[10].

The application of phosphate paints on the tendons of the prestressed pressure vessel at Dungeness "B" Power Station in Kert, United Kingdom has been reported[11].

Numerous researchers have reported a marked increase in corrosion protection provided by the concrete cover, by applying a coat of "Neat cement slurry" to the prestressing steel after stressing process and before concrete is placed[12-15]. This technique has been highly recommended in cases where the environmental conditions require increased corrosion protection.

It has been found that out of eleven protection materials evaluated for new prestressed and substructure concrete components, epoxy-coated reinforcing bars and prestressing strands showed no corrossion, eventhough they were embedded in concrete with high chloride content[16].

Corrosion properties of polymer coated prestressed steel strands have been investigated in the laboratory using impressed current tests[17]. It has been concluded that polymer coated prestressed strands have excellent corrosion protective properties. Further research is needed to ascertain the feasibility of use in prestressed construction.

It has been reported that galvanized (or zinc coated) strand was used on the first prestressed concrete bridge built in the United States[18]. Research on galavanized prestressing steel concluded that a zinc coating appeared to be promising for corrosion protection of prestressing[19]. In france, investigations have been made on possible hydrogen embrittlement and mechanical properties of the zinc coated steel[20]. A reaction between the zinc coating and cement releases hydrogen. It has been reported that this undesirable reaction can be inhibited by addition of chromates to the cement or by dipping the galvanized steel in a chromate solution[21,22].

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5.3. Protection of Prestressing Steel in Post-Tensioned Concrete

5.3.1. Temporary protection: It has been reported that good results for temporary corrosion protection of post-tensioning steel can be achieved by blowing "Vapour Phase Inhibitor" powder into the closed ducts, until the final corrosion protection is applied[1,2]. Similar procedure has been recommended for cases, where early application of cement or grout is not compatible with the construction schedule*.

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* Vapour Phase Inhibitors have been used in India recently for the 2nd bridge across Thane Creek.

It has been stated that it is mandatory to protect the prestressing steel and anchorage (in the case of post-tensioning) from corrosion during shipping, storage, handling and placing[3,4]. It has been highly recommended that the use of water-soluble oils and a more durable type of packing for temporary protection of prestressing steel*.

It has been suggested that the temporary corrosion protection for steel can be taken in the form of special strand pack wrapping, Vapour Phase Inhibitors (VPI) or a combination of both[5,6]. A commonly used VPI consists of dicyclo-ammonium nitrite crystals, which sublime to create a vapour that minimizes ongoing corrosion by forming an insoluble ferric oxide coating on the steel[2].

It has been recommended that the temporary corrosion protection of the tendon should be carried out at the steel wire mill[7]. It has been suggested that the strands or wires should pass through a bath of dewatering oil immediately after the heat treatment and cooling operations, so as to remove any surplus cooling water and to form a temporary protective film.

It has been found that temporary protection of cables can be achieved by spraying soluble oil on the bundles of manufactured cables, whereby, the oil will cover the steel with a protective film[2,8-10]

Work Carried out by CECRI, Karaikudi

Central Electrochemical Research Institute, Karaikudi, has done experiments in the aspect of temporary protection of prestressing steels[11]. The efficiency of different solutions, like, lime solution, cement extract, carbonated solution, hydroxide solution and patented inhibitor solution were compared using various electrochemical techniques, such as, anodic polarisation technique, peak potential technique, immersion studies and stress corrosion cracking studies. From these studies, it has been concluded that the prestressing steels can be temporarily portected by keeping them in complete immersion in inhibitor solution covered by Indian Patent No.109784/67 during storage at site or while lying in conduit (before grouting).

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* It is common practice these days to thread the HTS cables after concreting and only immediately prior to stressing. The precautions for the storage, therefore, assume added importance. On the 2nd TCB, temperature controlled and humidity controlled storage facilities have been used. Temperature and humidity control is essential to prevent the water soluble oils applied to the HTS from degenerating or becoming unstable.

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5.3.2. Protection of cable sheathing: It has been reported that the main cause of corrosion in unbonded tendons is the penetration of moisture to the tendon, either through the sheathing and coating or at the anchorage[1]. Consequently, the sheathing should be of a type, which is water tight; strong and resistant to abrasion or damage during transportation, installation and concreting. And also, the sheathing should be free from pinholes and it should be continuous throughout its length, upto and including the anchorages. Although, various types of sheathing have been used in the past, it has been found that plastic materials are the most suitable[2].

While PVC and low density polyethylene have been extensively used as sheathing, they are not able to satisfy all the above conditions compared to higher density materials. The use of PVC is not recommended because chloride ions can be released under certain conditions.

It has been recommended that the plastic material should be either high density polyethylene or polypropylene. Both materials are tough, durable and non-reactive. High density polyethylene is more flexible and less liable to embrittlement at extremely low temperatures, while polypropylene is more stable at high temperatures. Both materials have high resistance to abrasion and creep, although polypropylene is slightly superior in these respects. For general applications, a thickness of 0.75 mm to 1 mm has been recommended so that the tube does not deform to the pattern of the strand.

It has been stated that for temporary and permanent anchorages, continuous-diffusion impermeable polypropylene or polyethylene sheaths are suitable[3]. The minimum wall thickness recommended is 1mm. It has been suggested that to resist degradation, plastics can be used with carbon black or ultraviolet inhibitors. It has been reported that heat shrinkable tubing, precoated with a controlled thickness of sealant can be used for sheathing.

It has been reported that light corrugated metal sheaths are not suitable since they are easily perforated by corrosion. It has been stated that any metal used as a sheathing, has to be compatible with the tendon, so as not to induce corrosion potentials between the different metals. Till a better material is tried and tested, it will be prudent to use VPI treatment to prevent corrosion of such corrugated metal sheaths, the usual thickness of which has been already increased to 0.5mm instead of the earlier 0.25mm. Their storage with ends sealed by plastic covers in temperature and humidity controlled sheds is also necessary.

In Netherlands, when button headed wire tendons of a post-tensioned bridge consisted of an aluminium trumpet attached to a steel sheaths, failures of wires had occurred at the junction of the aluminium trumpet and steel duct[4]. After extensive investigations and research, it has been concluded that the failures were as a result of hydrogen embrittlement.

It has been reported that corrugated ferrous-metal ducts with galvanizing can be used in bonded post-tensioning systems[5]. Ducts, such as, epoxy-coated metal or plastic, which resist the penetration of chloride or water have also been specified on several new bridges, In Europe, use of plastic ducts for post-tensioning applications have apparently been resolved satisfactorily.

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5.3.3. Protection by grouts

Protection by Cement Based Grout

Many reports indicate that the grout itself can prevent corrosion of the encased prestressing steel effectively and economically[1-6]. High quality, dense and impervious grout made of portland cement can provide lasting corrosion protection for prestressed bridges except in the case of extremely corrosive environments[7-9]. The protective mechanism of grout depends mostly on the presence of continuous cement gel[10,11]. This gel provides an alkaline film of liquid to the surface of prestressing steel, which keeps the prestressing steel in "Passive condition".

Other cements which may be applied to grouting includes slag cement, resin gypsum cement, luminite cement and combinations of the above with portland cement[8]. Certain additives, such as, Chemical Resin AM-9, Calcium Chloride, Silika-4, Embeco, Ferrogrout, Intraplast, Benotite, Plastiment, Lignasol can be used with cement in order to vary its characteristics with changing job conditions. Water soluble chemicals can also be used as additives in grouts. Carbohydrates can be employed to reduce the water-cement ratio and promote workability. In Israel, Methocel 65 HG 4000 Std cellulose has been utilised in grouting work[12]. Protective colloids, such as, gelatin, agar and ammonium stearate can be used as thickners in thickners in liquid grout mixtures.

It has been reported that the cement used in the grout should be portland cement ASTM C150 or air entraining portland cement ASTM C 175 of medium fineness[13,14]. It has been recommended that in addition to air-entraining agents, super plasticizers can be introduced into the grout[15,16]. The addition of about 0.01 to 0.03 per cent of coarse aluminium filings improves the bond strength and due to expansion, it is likely to force out the duct-water which has bled from the grout[17,18]. But the aluminium filing react with cement and produces hydrogen gas which might cause hydrogen embrittlement. And also, this gas lowers the compressive strength of the cement grout.

ASTM has specified that the minimum compressive strength of the grout for 7 and 28 days should be 1360 and 2040 kg respectively[15]. In aggressive environments, it has been recommended that prior to the application of the grout, the ducts should be flushed out with a "neat cement slurry" for additional protection. This provides additional hydroxides and minimize the possibility of voids in the protective cement gel[19,20]. For grouting larger ducts (more than 12 cm diameter), it has been considered good practice to regrout, a few hours after the first operation.

Standard Specifications

It has been found that many countries have emphasized the importance of the strict specifications and rigid enforcement of the specifications for grouting[21]. The French specification, do not permits the use of high alumina cement in the prestressed concrete*. German DIN specifications permit only the use of portland cement of medium fineness without any chloride content for grout[22]. In Germany, the use of post-tensioned structural elements without grout is not permitted. The Swiss code recommends the use of additive for grout, which increases its flowability, reduce bleeding and produces a volumetric expansion[23]. The French specifications forbid the use of cements and admixtures which contain chlorides. Furthermore, steel with surface defects should not be used. Berlin specifications recommend complete exclusion of air from the ducts and immediate grouting after prestressing. It prescribes the minimum cement content in the grout and forbids the use of cements and admixtures containing chlorides.

For protecting the tendons in prestressed concrete pressure vessels, the performance of a portland cement grout has been investigated by two different types of tests[24]. In the first test, the specimens after grouting were stressed to 60 per cent of the UTS while in contact with the corrosive solutions of 0.1M H_2S, 0.1M NaCl and 0.2M NH_4NO_3 and in the second test, the grouted specimen were first exposed to the solution for long times without an applied stress and were subsequently strained to failure. From this study, it has been concluded that the portland cement grout can provide complete protection to the steel in the aggressive environments, provided the grout remains intact.

Protection of Unbonded Tendons by Grease

ACI-ASCE has recommended that unbonded tendons of the prestressing steel can be permanently protected against corrosion by a proper applied coating[25]. While grease, wax, plastics,

* In India, use of aluminium powder in grout was in vogue 2-3 decades ago. This has since been discontinued as it is known to promote corrosion. The specifications for grout and the method of execution were not codified. This has since been done in the latest IRC: 18-1991.

bitumen and other materials have been used, it recommends that the protective compound should be in the form of a grease which can assist free movement of the tendon during stressing. Various types of grease are commercially available. However, most of the grease are not impervious to moisture and test results have shown that aggressive substances can migrate through the grease and come into contact with the steel, unless the outer covering is completely water-proof[26].

From the documentary evidences, it has been strongly recommended that a grease can provide extended protection to stressed high tensile steel tendons[27]. It has been reported that grease can be utilized to protect the tendons of the nuclear reactor vessels or naval marine applications. In West Germany, U.K. and U.S.A, petroleum jelly, either pure or enriched, with dissolved inhibitors and lithium-based grease have been used for protection of prestressing tendons.

The prestressing tendons at Wylfa Power Station in United Kingdom was protected by a grease containing a proprietory corrosion inhibitor[28]. Eight years after initiation of construction, a statutory inspection of prestressing tendons revealed extensive pitting of exposed hoop tendons with some pits as deep as 0.3 mm. Laboratory studies have concluded that pitting was due to the combined action of contaminating salt from the sea and moisture from the air.

It has been reported that despite the application of heavy grease, a number of tendons installed in the bottom cap of the prestressed pressure vessel under construction at Dungeness 'B' Power Station in Kent, United Kingdom, corroded badly after approximately nine months of unstressed storage in conduits[29]. It was later found that water had entered the tendon ducts and emulsified the grease protecting the tendon wires.

In U.K., the post-tensioned unbonded tendons in a elevated building slab over a parking area, were protected by a coating of corrosion inhibiting grease with specially wound kraft fibre paper[30]. It has been reported that after 40 days, some tendons failed after stressing and additional sporadic failures continued to occur at reducing frequencies. Unbonded tendons in the roof of a hotel structure in U.K., was covered by grease and plastic sheath[30]. It has been reported that two tendons have failed in the structure, when one of the strands projected about 1m in the adjacent room.

Performance of two different organic coatings for protecting the tendons in prestressed concrete pressure vessels has been investigated by two types of tests[24]. The two organic coatings used in these tests were commercially available petroleum based grease containing corrosion inhibitors. In the first test, the specimen after coating were stressed to 60 per cent of the UTS while in contact with the corrosive solutions of 0.1M H_2S, 0.1M NaCl and 0.2M NH_4NO_3 and in the second test, the coated specimen were first exposed to the solution for long time without applying any stress and later, they were subsequently strained to failure. From this study, it has been concluded that both the organic coating materials provide complete protection to the steel in the aggressive environment, provided the coating remains intact.

It has been reported that carbon steel wires contained in the tendons of the prestressed concrete vessel coated, at the time of installation, with a hygroscopic grease were found to be in severely corroded and broken condition when the tendons were pulled for inspection[31]. The high viscosity grease was found in some areas transformed into a material, consisting of heavy weight oil, significant amount of water and also about 1 per cent by weight of formic and acetic acids. Numerous organisms were detected using fluorescein isothiocyanate staining/epiflouor escence microbial population were or had been active in the materials of the tendon. It has been concluded that bacterial contamination had occurred on the wires during installation. Entry of air and water vapour was possible near the points of exit of the wires from tendon. These conditions allowed a community of aerobic and facultatively anaerobic bacteria or microbes to break down the grease forming the volatile organic acids, carbon-di-oxide and hydrogen with a decrease in the oxygen level in the tendon. Hence, it has been emphasized that the grease should be stable against water and oxygen and should not separate into soap and oil. It has been recommended that hydrophobic greases are preferable for corrosion protection of prestressing tendons[27].

Protection by Other Materials

It has been reported that instead of cement, or epoxy polyester resins can be used as a grouting material[15]. Concrete can be placed around the cable and epoxy resin paste, without the use of cable ducts. The cables in a 176ft span prestressed concrete foot bridge at Ranchi, were protected against corrosion by encasing them with a mixture of bitumen and sand and wrapping them round with bitumen - impregnated paper.

Work Carried out by CECRI

CECRI has identified two inhibitor systems for use in concrete grout and evaluated their efficiency by the following methods.

I. CORROSION EVALUATION TESTS

(a) Potential-time studies

Potentials were monitored for 35 days by adding 2 to 4 per cent chloride for both the inhibitor systems. Uninhibited cement grout showed comparatively more negative potential than the inhibited cement grout. This showed that the systems could effectively passivate the steel.

(b) Anodic Polarization Technique

In this technique, specimen of cylindrical size with centrally embedded and prestressing wires were used for evaluating the inhibition efficiency. Specimen were kept immersed in various alkali-chloride solutions and subjected to a constant current of 290 μ A/cm^2 for five minutes. In this technique inhibited cement grout has tolerated upto 14 per cent chloride, whereas, uninhibited cement grout tolerated only upto 3 per cent chloride[32].

(c) Gravimetric method

By adding 2-4 per cent chloride with inhibited cement grout, the gravimetric weight loss was measured after 30 days. Both systems had 100 per cent inhibition efficiency, whereas, uninhibited cement grout had corrosion rate ranging from 0.0175 to 0.0365 mmpy.

II. COMPRESSIVE AND TENSILE STRENGTH TESTS

Change in compressive and tensile strength due to the addition of inhibitor was evaluated as per ASTM Standard[33,34]. Obtained strengths were compared with resin grouts as specified in ASTM Standards for both inhibitor systems[35].

It has been concluded that the compressive and tensile strengths of both the systems were almost equal when compared to the resin grouts.

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5.3.4. Anchorage protection: The prestressing anchorage is normally embedded in concrete, at such a depth that the concrete provides adequate corrosion protection[1]. At the stressing end of the tendon this is usually achieved by means of a local pocket, which is filled with mortar after the tendon has been stressed and trimmed to length. Test have shown that the anchored end of a tendon is the most vulnerable location for corrosion attack, especially as the interstices between wires or the wires forming a strand tendon, form capillaries which allow moisture to gain assess to the most highly stressed parts of the tendon. It has also been shown that the sealing the stressing pocket with mortar does not always completely seal the anchorage and capillaries are sometimes left because of imperfect bond.

Hence, it has been recommended that the exposed end of the tendon and the gripping part of the anchorage must be completely sealed against moisture. Two methods have been reported. (i) The exposed portion of the tendon and gripping of the anchorage should be coated with some material, which will give permanent protection against the entry of the moisture. Epoxy-resin compound has been recommended as suitable compound for this purpose. (ii) The vulnerable parts of the anchorage may be coated with the same corrosion protective materials which are being used elsewhere on the tendon. In this case, it has been suggested that a specially made covering of rigid metal or plastic should be fixed to the end of the anchorage so as to completely encapsulate the tendon and grips.

It has been suggested that after sealing the end of the tendon, the stressing pocket should be filled with a low-shrink chloride free mortar[2]. It has also been suggested that a resin bond agent should be painted on the sides of the pocket to improve the adhesion.

In a precast post-tensioned building in U.K., the anchorage plates were covered with a heavy bituminous material and the wedge portions were sealed to avoid infiltration of water[3].

Anchor head zone has also been protected by a flexible grease-based or bituminous material, which allows the relative movements to take place without destroying the efficiency of the corrosion protection system[4].

It has been reported that fixed end of the tendon in an anchorage can be protected by the use of an epoxy or polyester resin encapsulated system[4]. This system provides a double protection, such as, a crack free chemical barrier and an impervious plastic barrier on the outside of the anchorage.

It has been suggested that the free length of the tendon in the anchorage can be protected by covering each component of the tendon individually in a close-fitting polypropylene sheath with the annular space between them filled with a protective grease[4]. Another method reported is to surround the complete tendon with a continuous large diameter tube of either polypropylene or polyethylene which again extends from the top of the fixed end capsule to the underground of the anchor head. The internal annular space can be filled with cement grout, grease or bitumen compounds.

In the Air India Hanger, Bombay Air Port, the anchored portion of the cable was coated with special epoxy resin in order to prevent stress corrosion. The unbonded portion was embedded in PVC pipes filled with anticorrosive grease[4].

It has been emphasized that the bearing plate and other essential exposed steel components at the anchor head should be painted with bitumastic[5]. Non-corrodable materials, like, plastics or plastic coated materials have been recommended for permanent anchorages in aggressive ground.

It has been reported that the anchor assembly was protected against corrosion by a coating of red-oxide, structural steel primer paint[6]. It has also been reported that this coating has performed well during the nearly one year of outdoor storage.

The possibility of ingress of moisture in the anchorages can be substantially reduced by reducing the number of anchorages by providing continuous members, as well as, by providing leak-proof expansion joints at their ends.

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5.3.5. Admixtures: Calcium nitrite has been reported as an anodic corrosion inhibiting admixture for concrete[1-6]. First, it was used in Japan to facilitate the use of salt bearing beach sands in reinforced concrete. It has been revealed that aside from its corrosion inhibiting characteristics, calcium nitrite can be used as an accelerating admixture (ASTM C-494, Type C). It has been reported that it increases compressive and flexural strengths, decreases shrinkage and has no effect on freeze-thaw durability. It has been concluded that calcium nitrite can be used as a viable corrosion inhibitor for concrete.

A three year corrosion research project on "Protective systems for new prestressed and substructure concrete" has been carried out to evaluate the performance of calcium nitrite corrosion inhibiting admixture for both reinforced as well as prestressed concrete structures[7]. Calcium nitrite admixture has been evaluated by two laboratory studies that included a 48-week cyclic wet and dry salt water exposure and year-long study with cyclic salt water exposure on full size sections of reinforced columns, beams, prestressed piles and stay-in-place bridge deck panels. From these laboratory studies, it has been concluded that the calcium nitrite corrosion-inhibiting admixture did not significantly delay the initiation of corrosion when compared to conventional concrete specimen in these studies. However, the severity of the subsequent corrosion process (total ampere hours of corrosion current) on grey bars and prestressing strands was reduced significantly when compared to conventional concrete specimen. The amount of corrosion by products on the grey bars was about one-tenth that measured on bars in conventional concrete specimen. In the moist-cured calcium nitrite concrete, it has been found that chloride permeability was similar to conventional concrete, but it was greater in the case of heat-cured calcium nitrite concrete specimen.

A three-year corrosion research project on "Protective systems for new prestressed and substructure concrete", has been conducted to know the performance of silica fume admixture for concrete[7]. From this study, it has been concluded that the concrete with high strength silica fume has shown very high corrosion protection capabilities. Chloride penetration was found extremely low. It has been stated that the very high electrical resistivity of this concrete mixture can drastically reduce the corrosion currents, eventhough the chloride level at the reinforcement steel becomes high enough to initiate corrosion.

It has been reported that microsilica can be used as an admixture for concrete[27]. It has been stated that microsilica reduces (in some cases) the rate of carbonation, decreases permeability to chloride ions, imparts high electrical resistivity and has little effect on oxygen transport. It has also been reported that microsilica binds the potassium and sodium oxide alkalies present in cement, and thus reducing detrimental effects with alkali-reactive aggregates. It has been concluded that microsilica admixed concrete can protect the reinforcement against corrosion.

To accelerate the setting of the cement and the hardening of the concrete or grout, calcium chloride ($CaCl_2$) had been used as an active ingredient in the reinforced as well as prestressed concrete structures[8-11]. Field experience and laboratory experiments have shown that 2 to 5 per cent of calcium chloride in concrete or grout containing prestressing steel leads to serious corrosion[12-16]. Based on the works of numerous researchers, the use of calcium chloride as frost protection agent has already been prohibited in many countries[17-18].

The American Concrete Institute Code for Corrosion Prevention Requirements states that the admixture should not contain calcium chloride[19]. The Swiss code for prestressed concrete emphasizes that the concrete or grout should not contain calcium chloride, nitrates or other chemically aggressive substances[20]. It has been suggested that the cement containing more than a trace of calcium chloride should not be permitted in prestressed concrete structures[21].

It has been reported that in a post-tensioned bridge in Netherlands, 1 per cent sodium carbonate was added to the portland cement grout to speed up the hardening process in the winter season[21].

It has been found that course aluminum filing can be used as an admixture to grout[22,23]. But it can react with cement and produce hydrogen gas which might cause hydrogen embrittlement problem in the prestressing steel[24,25].

The corrosivity of the sodium thiocyanate (NaSCN) based admixture for prestressing wire, has been investigated using stress corrosion cracking tests[26]. The specimen were tested in synthetic pore water solution containing a NaSCN-based admixture of 90 fl OZ per 100 lb of cement. The concentration of NaSCN was 0.203 per cent by the weight of cement. After 48 hours of exposure, no evidence of rusting or crack advancement was observed in any of the test specimen. From this study, it has been concluded that a NaSCN concentration of 0.203 per cent by weight of cement will not induce stress corrosion cracking in low relaxation and stress-relieved cables. It has also been concluded that the use of NaSCN based accelerating admixtures is safe for reinforced concrete applications for NaSCN concentrations of upto 0.75 to 1 per cent by weight of cement.

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5.4. Coatings to Concrete Surface

5.4.1. Introduction: Concrete is permeable to water and solutions of chlorides and sulphates[1]. Penetration of chemical or salt solutions through concrete is likely to cause corrosion of reinforcing steel. Rebar embedded in the concrete can be protected by several methods, such as, inhibitor admixtures, cathodic protection, coating to reinforcing steel and coating to concrete surface. The application of a more impermeable protective coating, to prevent corrosion, can be considered for both existing as well as newly constructed structures. During the last two decades, numerous organic and inorganic coatings have been developed for concrete surfaces. The various compositions that have been used to coat concrete includes bituminous coatings, linseed oil, chlorinated rubber, polyvinyl copolymers, acrylics, polyurethanes, epoxy resins, etc.[2-9]. A brief summary of these coatings are given below:

5.4.2. Inorganic surface coatings: Aqueous solutions of sodium silicate, magnesium and zinc fluosilicates can react with the soluble calcium compounds and form insoluble calcium silicate or calcium fluorosilicate, This treatment helps to increase surface hardness and reduce dusting. Concrete can also be treated with silicon tetrafluoride gas (Ocrate process) which is claimed to be more effective because of increased depth of penetration. However, these treatments have been found to have only limited chemical resistance.

5.4.3. Organic surface coating

(i) Bituminous materials

These are made with either asphalt or coal tar and have excellent resistance to water and moisture. Higher softening asphalts are preferred for tropical exposure. To prevent blistering, concrete surface should be in dry condition. Thin asphalt or coal tar "cut backs" are used as primers for hot melts. Inspite of their good resistance to acids, oxidizing solutions and concrete destroying salts, black colour and softening at high temperature restrict their use.

Coal tar pitches have excellent water resistance with moderate resistance to acids and alkalies. They are also resistant to barnacle attack in sea water. Their poor resistance against sunlight and weather prohibits their use in open exposure conditions. Coal tar coatings have very poor abrasion resistance. They tend to soften at high temperatures. Usually, coal tar coatings do not adhere well to concrete unless suitable primer is used.

(ii) Emulsions

These are classified into two types. They are, Bituminous emulsions and latex emulsions. Bituminous emulsions are made using either asphalt or coal tar base binders. Both these emulsions can be applied on slight damp surfaces. But they suffer from the disadvantage of higher permeability. In humid conditions, drying becomes difficult. Coatings may be damaged by rain.

Acrylic latexes are resistant to heat, sunlight and weather. They are moderately resistant to acids, alkalies and oils but are not recommended for immersion conditions. Adhesion of this coating to concrete is good.

Polyvinyl acetate latex shows good resistance to chlorine and hydrogen sulphide gases. This is having very good adhesion property. But this is not suitable for immersed condition. In open atmosphere, these latexes are resistant to blistering and weathering.

Styrene-butadiene latexes are not suitable for exterior use.

(iii) Rubber and resins

Coumarone-indene (fractionate of crude coal tar) and petroleum resins are used with plasticizers, such as, the chlorinated waxes. These coatings are having excellent water resistance and good abrasion resistance. Adhesion property of this coating to concrete is excellent. But it will tend to fade under sunlight.

The styrene-butadine resins possess excellent resistance to both strong and weak acids as well as strong alkali, oil grease and water. Exterior durability is good. But these resins also tend to fade under sunlight.

Coatings containing chlorinated rubber resins are having excellent resistance to alkalies, moisture and abrasion. Adhesion to concrete is good. For open exposure conditions, coatings are to be suitably pigmented. These coatings can be readily over-coated.

Chlorosulfonated polyethylene resins are having good flexibility, low permeability to moisture, excellent weather resistance and good chemical resistance. It has excellent resistance to abrasion, to sunlight, to ozone and to oxidation. But it is highly expensive and needs special application techniques.

High density polyethylene coating is having excellent flexibility and high chemical resistance, but it is sensitive to temperature. Weather resistance is good. It will be further improved by adding carbon black. This coating is permeable to several gases. Applying polyethylene on concrete surface is highly impractical.

(iv) Acrylic

Acrylic lacquer coatings based on ethyle methacrylate or methyl methacrylate produce tough coatings, but with pinholes. These coatings are resistant to heat, sunlight, weather and to acids, alkalies and oils. These coatings are not suitable for immersion conditions.

(v) Vinyls

Polyvinyl Alcohol

These coatings adhere extremely well to concrete surface, moderately resistant to water and mildly aggressive chemicals. They have good abrasion resistance, flexibility and hardness.

Polyvinyl Chloride

These coatings are having excellent acid resistance when used as top coat. But adhesion to concrete is very poor. Over coating is also difficult. These coatings are not suitable in the moist surfaces.

Polyvinyl Chloride - acetate

These are having excellent abrasion resistance as well as chemical resistance. Exterior durability is also good. But, it is sensitive to degradation by heat and sunlight.

Polyvinylidence Chloride

These coatings do not bond well and multiple coatings are needed for adequate film thickness. Impact resistance of these coatings is also poor.

Polyvinyl Burtyral

Concrete surface needs special preparation for these type of coatings. It has outstanding abrasion resistance, toughness, flexibility, water and heat resistance.

(vi) Thermosetting coatings

Drying oils (linseed oils, tung, soyabeans, castor) are used as penetrating surface sealers for concrete. These coatings are provided moderate protection against chlorides, sulphate and fatty acids. This is having very poor abrasion resistance and durability.

Alkyd resins are having good weatherability, flexibility and abrasion. But the surface of the concrete requires careful preparation.

Phenolic oil is having good resistance to water, dilute mineral acids. These coatings generally chalk and loose their glossiness. Resistance to impact is good but the abrasion resistance is fair. Adhesion of this coating is very good. These coatings are mainly used to reduce dustings.

Epoxy-esters are having excellent adhesion, flexibility and chemical resistance, but vulnerable to caustic attack. They have good gloss and colour retention. An alkali resistant primer is required for concrete surfaces. However, epoxy-esters are inferior to two component epoxy systems.

Silicone-alkyds are similar to epoxy-esters in their properties. They are also needed an alkali-resistant primer for concrete surface.

(vii) Neoprene

Neoprene coatings usually contain a built in primer to improve adhesion to concrete. These coatings are having excellent weather resistance property.

(viii) Urethane

These coatings are oustanding in their toughness, abrasion resistance and water and chemical resistance properties. They are reasonably resistant to strong acids. They are having excellent durability but gloss retention is not so good. The disadvantage of this coating is lack of adhesion unless the surface is carefully prepared.

(ix) Two-pack Systems

(a) Polyester

This system can be used with fabrics or glass fibres to repair or protect concrete surfaces. They are resistant to erosion, chemical, weather, solvents, etc. But this coating requires special application technique.

(b) Epoxy

Two component epoxy resin films are extremely tough, hard and chemically resistant to most aggressive agents. Resistance to weathering is generally good. These coatings are also having good abrasion resistance. Although, there are many types of epoxy resins available, the one normally used for protective barriers is based on a reaction product of bisphenol A and epichlorohydrin. The epoxy resin, which is usually a liquid, must react with another chemical called a curing agent or hardener before it comes a solid and develops chemical resistance, hardness and abrasion resistance. The curing agent used with the epoxy resin has a major influence on the mechanical and chemical resistance properties of the hardened resin. Curing agents most commonly used are aliphatic amines, amine adducts, amido-amines, polyamides, polysulfides, etc.

Amine cured product has high strength and good chemical and abrasion resistance. But, it is brittle in nature.

Polyamines produce coatings which are very hard with good adhesion and excellent resistance.

Polyamide resins provide excellent adhesion to concrete surfaces. They have excellent toughness, impact resistance and abrasion resistance. Besides they are non-toxic and non-irritating, they can be applied even on damp surfaces.

Polysulfide produces extremely flexible and impact resistance coatings.

(c) Furan

Because of the inherent brittleness and lack of flexibility, films of furan resins are limited in use on concrete surfaces.

(d) Neoprene

Two-part neoprene coatings* are used on concrete surfaces where very high dry film thickness is required (0.25 to 1.75 cm). With adequate curing, these coatings provide a high degree of water, chemical and oil resistance. These coatings show a high degree of resistance to most strong and weak acids.

(e) Polysulfide

Two-pack polysulfide coatings exhibit outstanding resistance to ozone, sunlight, oxidation and weathering. But they are having poor heat and abrasion resistance properties.

(f) Coal tar epoxy

This is quite tough with good chemical resistance. Durability is also good. For immersed conditions and good abrasion resistance, a high resin content is required.

(g) Polysulfide-epoxy

This system combines the inherent rubber-like flexibility of polysulfides with the excellent adhesion and strength of epoxies. Adhesion to damp concrete is improved by this. Chemical resistance of this system is similar to epoxies.

Selection of a protective system, which provides optimum performance at the lowest cost is complicating, because, there are so many systems available. To help in the selection process, protective barrier systems are divided into three general categories according to the severity of the chemical service conditions and these are mild, intermediate and severe[4]. General categories of protective barrier systems are given in Table 5.1.

5.4.4. Concrete sealers: Water proofing is one practical way of preventing corrosion of embedded steel reinforcement[10]. Sealers are used as water poofing material for concrete surfaces. Sealers supplies as a solution or as a suspension in a solvent, merely reduce water inflow. They either fill or partially fill the pores in the surface or line them with water repellent material, which reduces the tendency of the surface to absorb liquid water. A wide range of penetrating sealers have been used from water repellent silicone resins to acrylic resin solutions, silicone resins, epoxy resins and polyurethanes. Aqueous solutions of alkaline silicates and silicofluorides have also been used as sealers on concrete surfaces.

Several laboratory studies have been carried out to evaluate sealers, but mainly for the purposes of highway bridge deck protection[11-16]. It has been concluded that substantial differences were found in the laboratory tests on the effectiveness of the various sealers. Some sealers decrease

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*Similar two component epoxy coatings in 3 coats have been provided for superstructure and substructure of some bridges in India, like, Thane Creek Bridge, Mandovi Bridge, Kasheli and Kalwa Bridges, etc., on all external faces except the top of deck. It is now a practice in India to cover the deck surface with mastic asphalt layer of 12 mm thickness to retard ingress of water.

Table 5.1. Protective Barrier System - General Categories

Severity of chemical environment	Total nominal thickness range	Typical protective barrier systems	Typical but not exclusive uses of protective systems in order of severity
Mild	Under 40 mil (1 mm)	Polyvinyl butyral, polyurethane, epoxy, acrylic, chlorinated rubber, styrene acrylic copolymer	* Protection against deicing salts * Improve freeze thaw resistance
		Asphalt, coal tar, chlorinated rubber, epoxy, polyurethane, vinyl, neoprene, coal tar epoxy, coal tar urethane	* Prevent stainning of concrete * Use of high-purity water service * Protect concrete in contact with chemical solutions having a pH as low as 4, depending on the chemical
Intermediate	125-375 mil (3-9 mm)	Sand filled epoxy, sand filled polyester, sand-filled polyurethane, bituminous materials	* Protect concrete from abrasion and intermittent exposure to dilute acids in chemical, dairy and food processing plants
Severe	20-250 mil (1/2 - 6 mm)	Glass-reinforced epoxy, glass-reinforced polyester, procured neoprene sheet, plasticized PVC sheet	* Protect concrete tanks and floors during continuous exposure to dilute mineral, (pH is below 3) organic acids, salt solutions, strong alkalies
Severe	20-280 mil (1/2 - 6 3/4 mm)	Composite systems: (a) Sand-filled epoxy system top-coated with a pigmented but unfilled epoxy	* Protect concrete tanks during continuous or intermittent immersion, exposure to water, dilute acids, strong alkalies and salt solutions
	Over 250 mil (6 mm)	(b) Asphalt membrane covered with acid-proof brick using a chemical resistant mortar	* Protect concrete from concentrated acids or acid/solvent combinations

water absorption of concrete to approximately one-third of that of uncoated concrete, while others have little effect or even increase water uptake by as much as 30 per cent.

It has been reported that although surface sealers give good water repulsive properties to the concrete surface, their permeability to oxygen, carbon-di-oxide and water vapour were high[9]. In some conditions, the effect of the sealer was to lower the water content of the surface pores, so that they were no longer saturated. It has also been reported that rate of carbonation was increased by the use of surface sealers.

Coating Evaluation Studies

The corrosion preventive effects of concrete surface coating used as a protection measure for steel in concrete has been described[17]. Reinforced concrete specimen coated with urethane coating on the surface of the concrete were exposed to marine environments. It has been concluded that urethane coating over the concrete was not effective.

The Florida DOT has developed a rapid test for defining the relative effectiveness of coatings and penetrants (placed on concrete not subject to abrasion) in preventing chloride penetration and subsequent rebar corrosion[18,19]. A 10 x 12 cm size concrete cylinder containing a reinforcing bar and coated with the material under test, was partially immersed in a 5 per cent NaCl solution at constant temperature. An impressed current of 6 volts DC was then applied between a rebar in the salt water (cathode) and the rebar in the concrete cylinder (anode). The DC current causes accelerated movement of chloride through the concrete (if the coating was not completely impermeable) and accelerated corrosion of the anode once chloride reaches the reinforcing bar in the cylinder. Current flow to each specimen was monitored daily until concrete cracking. An uncoated cylinder of typical concrete failed in several days, while well coated specimen could withstand 100 days or more of this treatment without damage. It has been reported that FHWA staff researching on the basic corrosion process, support the use of this type of test to evaluate coatings or characteristics of the concrete.

5.4.5. Work carried out by CECRI, Karaikudi: For rapidly assessing the usefulness of different water proofing materials, an alternate wetting and drying cyclic test has been developed[20]. About forty coating systems have been evaluated using this test. 10 cm concrete cubes with mild steel rods were cast. After demoulding, the top surface of the cube was coated with different coating materials. After the coating had dried all specimen were subjected to the test, without any curing so as to speed up the commencement of rusting.

The specimen were dried using a heating arrangement. Heating coils fixed in ceramic bases were connected to the main supply. To obtain the required temperature of 45^0 to 50^0C for drying, the distance between the top surfaces of the specimen and the heating coils can be adjusted by means of the pulley arrangement. The whole heating system was fixed to a steel frame for stability. After drying, the specimen were wetted with sodium chloride solution. 3 cycles per eight hour working day had been used.

It has been reported that corrosion of steel reinforcement takes place within 45 days under this accelerated test, while in the immersion test no corrosion was observed even at the end of eighty days. It has been concluded that this accelerated drying and wetting test is more severe than the other tests. It has also been concluded that various coatings to concrete can be evaluated using this method.

From ISI Hand Book, BSI Catalogue and Annual Book of ASTM Standards, it has been noted that no standard acceptance tests are available for corrosion protection coatings on reinforced concrete surface[4,21-23]. Hence, CECRI, Karaikudi has developed three different types of acceptance test for evaluating paint systems[24]. These tests are, precracked cantilever model slab test, resistivity measurement and adhesion test. All these tests yield quantitative data to enable us to assess the relative performance of different coating system. Ten proprietory systems were evaluated using the above three test methods and the paint systems were graded.

Precracked Cantilever Model Slab Test (Fig. 5.3)

This test can be used to test the flexibility and corrosion protection of the coating under loaded condition. A notched and cantilever loaded slab was used to produce crack of specified width parallel to the length of test specimen embedded in concrete. The slab was 900 mm long, 250 mm wide and 75 mm thick. M20 concrete with a water-cement ratio of 0.5 was used. 6 mm diameter mild steel bars were used as reinforcement in the slab. To control the direction of the crack, a notch was provided near the fixed end of the slab and at the other end 25 mm diameter hole was provided. Polished and weighed steel specimen were introduced both below the notch and at the free end of

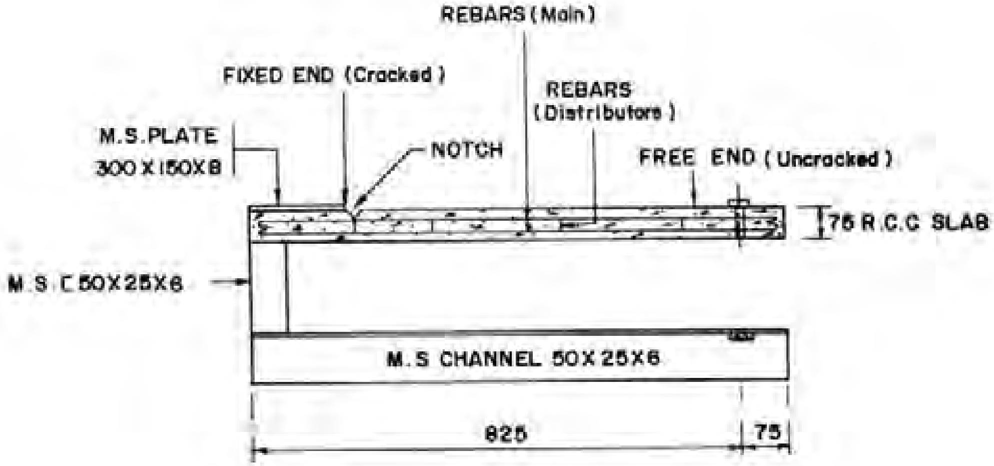

Fig. 5.3. Cantilever Model Slab

the slab. Before use, the steel reinforcement cage was derusted by pickling, rinsed in tap water and deionised water and air-dried.

After the curing period, the top surface of the slab was sand blasted. Then the paint system was applied by using brush over the sand blasted surface and allowed to cure.

The slab was then fixed on the loading frame. Using the "Grifford Udall Prestressing Unit", the slab was stressed and the downward deflection at the free end was gradually varied by tensioning a prestressing steel wire passing through a hole at the free end of the slab. By this downward deflection at the free end, a crack was formed along the root of the notch parallel to the test specimen and the width of the crack was accurately monitored by using a travelling microscope. After formation of a crack of specified width, wedge-grips were locked in place at the end anchors and thus the downward load deflection on the slabs as well as the corresponding crack width could be permanently maintained throughout the test period.

The precracked slab along with the loading frame was taken to the exposure yard. There a 25 mm high bund was constructed along the edges of the slab using cement mortar and the joints were sealed with epoxy to prevent leakage. They were then subjected to cycles of alternate wetting with 3 per cent NaCl solution and drying. The number of days taken for the appearance of first rust spot at the notched surface was noted in each system. After the exposure period of 120 days, the slabs were broken open and the surface conditions of the rebar were visually examined. The test specimen were derusted and reweighed again and corrosion rates in mmpy were calculated.

Resistivity Measurement (Fig. 5.4)

This measurement can be used for testing impermeability and chemical resistance property of the coating. The direct-reading digital resistivity meter designed and fabricated at CECRI was used in this study. The meter consists of built-in-spring loaded, four-probe unit and of electronic

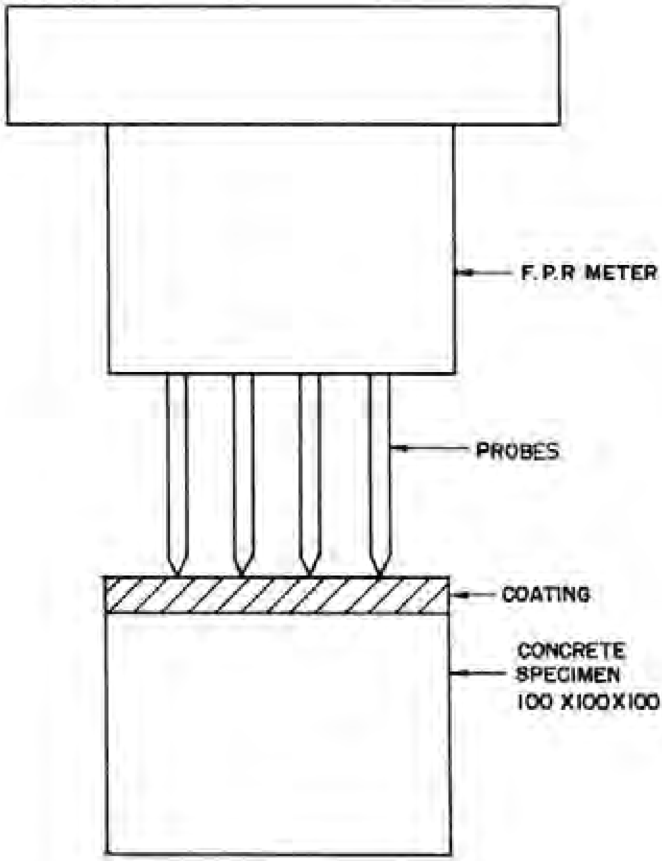

Fig. 5.4. Resistivity Measurement

circuits and scanning of the data to display the resistivity directly in a digital panel meter. The tips of the four probes were wrapped with sponge and saturated with potable water, for making effective contact with the concrete surface. The meter with built-in probe is highly sensitive and is also sufficiently rugged for field use. It is portable and battery-operated. The meter displays the resistivity values directly. 100 mm size cubes of M20 concrete with water-cement ratio of 0.5 were used in these studies. After curing period, all faces of the test cube were sand blasted. Then the surface coatings were applied to the required thickness.

The cubes were air cured for ten days and initial resistivity was measured. After that, the cubes were subjected to alternate wetting in 3 per cent NaCl solution and drying. At the end of 10 cycles resistivity was measured on all four vertical faces of each cube and the values were recorded. It has been observed that in certain cases there is considerable decrease in electrical resistivity due to the penetration of chloride ions. While in certain other cases, there is considerable increase in electrical resistivity. This indicates the impermeable property of the coating system.

Adhesion Test (Fig. 5.5)

A 20 KW capacity MONSANTO Tensometer was used for testing the adhesive strength of the paint over concrete surface. 50 mm diameter and 75 mm thick concrete cylindrical specimen was cast with a cylindrical steel specimen. After curing period, the flat surface of the concrete specimen was sand blasted and the specified paint system was applied. Painted surface was air cured for ten days. Another 25 mm diameter and 50 mm long steel specimen was given a tack coat of araldite and then fixed to the middle portion of the painted surface of the coated concrete specimen. The joint was allowed to set for three days.

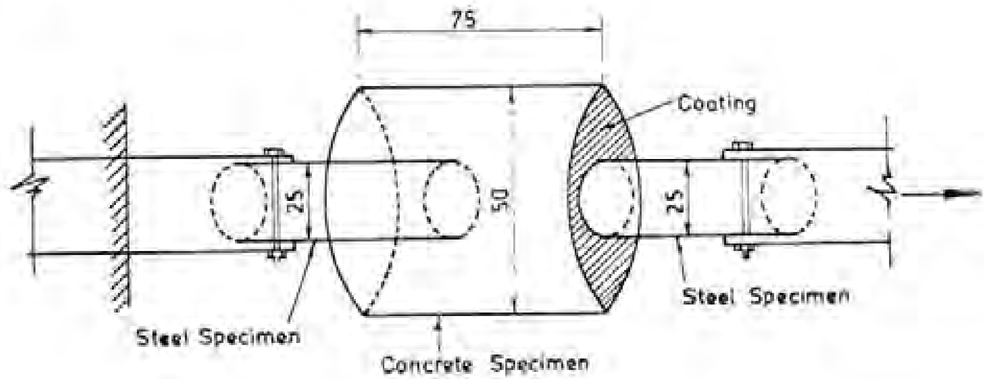

Fig. 5.5. Adhesion Test Set-up

Then the specimen was fixed in the MANSANTO Tensometer for testing. During testing, one end of the specimen was kept fixed and the other end was pulled at a rate of 1 mm/min. The maximum pulling force was recorded and the adhesion strength was calculated.

It has been concluded that the efficacy of different surface coatings for concrete can be effectively and quantitatively assessed by conducting the alternate wetting with 3 per cent NaCl solution and drying of reinforced concrete model slabs under cracked condition using the precracked cantilever loading system. This can also be assessed with the secondary tests, such as, electrical resistivity measurements and adhesion tests.

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5.5. Cathodic Protection

5.5.1. Introduction: Cathodic protection is an electrochemical method of controlling corrosion[1]. Corrosion is an electrochemical reaction between dissimilar metals or dissimilarities on the same metal. An electric current flows from one part of the metal to another part. Where the current leaves the metal surface and enters the surrounding electrolyte, corrosion occurs. Where the current re-enters the metal, no corrosion occurs. The place where current leaves and corrosion occurs is called anode. The place where current is collected and no corrosion occurs is called cathode[2]. The best method of controlling corrosion is to take off the corrosion current with an equal and opposite current applied from an external source[3]. This practice is known as cathodic protection.

There are two types of cathodic protection system:

1. By introducing anode metal to corrode and using the current it produces to save the metal under protection. This technique is called "Sacrificial or galvanic" cathodic protection.

2. By using ordinary electric service, converting the standard alternating current into direct current by means of rectifiers. This technique is called "Impressed current cathodic protection". Other energy sources, such as, solar panels, are also in use in remote areas where ordinary electric service is not available[4,5].

Sacrificial Cathodic Protection System

Sacrificial (galvanic) anode system produce the required protective current by an electrochemical reaction. This system consists of using a more reactive metal, such as, zinc, aluminium or magnesium to create the flow of current[6]. DC current is generated by the difference in the potential of the metal when connected. Current flows from anode (zinc, aluminium, magnesium, etc.) through the electrolyte to the corroding metal. When the current flow is adequate to make all the areas on the metal cathodic, cathodic protection is achieved.

Impressed Current System

The use and design of impressed current systems are more flexible than galvanic anode systems. Basically, the principle is the same for both systems, except impressed current systems energize anodes by use of an external energy source. Generally, a rectifier is used to convert available A.C. to D.C. The direct current is there introduced into the electrolyte by an anode or anodic bed especially designed to have a long life under relatively high current, high voltage conditions.

Many different types of anodes are available for use in "impressed current systems"[7]. They are graphite, high silicon, chromium-bearing cast iron, lead-silver alloy, carbon, platinum-coated niobium or titanium and conductive mastics and polymers.

One new "break-through" in this field is the automatic potential control system. This unit makes the use of cathodic protection, compensating for any changes which may alter the protective current requirements.

5.5.2. Cathodic protection in reinforced concrete structures: The first cathodic protection system devised for a bridge deck was installed in 1973[8]. In the past, the cathodic protection of concrete rebars has been applied on buried and offshore structures[9]. In recent years, this technique has also been successfully used to stop or prevent corrosion of rebars in concrete structures exposed to the atmosphere and contaminated by chloride. FHWA has officially stipulated that the only rehabilitation technique that has proven to stop corrosion in salt-contaminated bridge decks is cathodic protection[10].

When the cathodic protection system is energised, the oxygen reaching the steel surface is reduced to hydroxyl ions and the alkalinity generated restores the passive film around the reinforcing bars. As a consequence of the current flow, chloride ions are removed from the steel surface, thus eliminating one of the causative factors. Both these effects can restore the passivation conditions of reinforcing bars[11,12].

For under water reinforced concrete structures, such as, piers and piles, sacrificial anodes can be used[13]. But in the case of above water structures, sacrificial anodes are not suitable[14]. For concrete structures exposed to the atmosphere, the very high resistivity of the concrete and the low driving voltage available makes a sacrifical anode system generally inadequate to obtain sufficient current flow to make all areas of the reinforcing steel cathode. Furthermore, some sacrificial anodes (e.g., aluminium) have been demonstrated to be inappropriate, because their voluminous corrosion products break the surrounding concrete[15,16]. In practice, the cathodic protection of reinforced concrete structure is better realised by means of the impressed current system[17].

Early impressed current systems for protecting bridge decks utilised cast iron, ferrosilicon or graphite single anodes which were laid on the slab and covered with a thick layer of conductive asphalt (50 mm) as a secondary anode-backfil in order to distribute current uniformly[8,19-21]. Starting from 1977, the anode was replaced by titanium or platinum plated niobium wires. These anode systems were not suitable for application on vertical walls or ceilings. Moreover, their heavy weight and large thickness limited their applications.

Later on, slotted or mounted anode systems were developed for bridge decks in order to overcome the weight, size and possible surface wear disadvantages of conductive overlays[22]. The system consisted of a number of parallel slots cut into the concrete surface usually at 300 mm centers for effective current distribution[23-30]. In each slot, a wire anode (platinum or mixed metal oxide/ titanium) was placed surrounded by a conductive polymer grout to increase the anode area in contact with the concrete. To improve current distribution, the slotted anode system has been used in combination with carbon/graphite fibers installed perpendicularly to the primary anode wires to generate an anodic grid. These systems are complex to install and due to the high current density applied, acid attack of the area adjacent to the slots has been noticed.

Then conductive coatings systems were developed[2]. These coatings consist of carbon loaded organic coatings or sprayed metallic coatings applied over the entire surface to be protected. The most significant advantage of these systems is that they can be applied to complex shapes in any orientation and present no problems of weight or dimensional limitations. With the sprayed zinc anode system, it has been noticed that the application is labour intensive and time consuming, although the current distribution is good[31]. Short circuits between the zinc anode and the rebar may occur in some areas of minimum cover over the steel. These inconveniences, coupled with anode consumption and acid attack on the concrete caused by the anodic reactions, can make life of these systems quite short[32-34].

Distributed and Mesh anodes are the most recently developed. They are either copper coated with a conductive polymer material (a carbon-filled polyolefine) or of expanded titanium metal mesh coated with mixed metal oxides[35-37]. However, these polymer anodes are not able to compete with activated, titanium anodes in terms of mechanical, electrochemical and life-time properties. Mesh anodes have the advantage of achieving uniform current distribution combined with reduced installation cost compared to other types of anode systems. And also, this system does not require the use of conductive overlays. With respect to other anode systems, they provide uniform current distribution, long life and can be easily utilized for the protection of the most varied structures.

5.5.3. Cathodic protection for prestressed concrete structures: In 1985, a laboratory study was initiated to investigate whether cathodic protection can be used to protect pretensioned and post-tensioned steel in view of the risk of hydrogen embrittlement in high-strength steels at elevated potential levels and to determine whether the application of cathodic protection to protect the mild-steel reinforcement in prestressed structures may have an adverse effect of the prestressing steel[38].

These tests were conducted on full-size specimen simulating a typical deck and beam section of a pretensioned bridge and the deck of a thick-slab post-tensioned bridge. Prestressing wires were used under unstressed condition. A cathodic protection system was installed and surface half-cells and embedded probes were used to measure potential and current levels in the specimen.

The study concluded that cathodic protection could not effectively protect the tendons in post-tensioned concrete because of the shielding effect of the metal duct.

It was also found that current distribution along the length of the tendons was not uniform in the case of pretensioned beams. This could result in hydrogen evolution in higher current zones (i.e., over protection) while leaving some steel unprotected.

The study determined that cathodically protecting the mild-steel reinforcement in post-tensioned deck slabs would not adversely affect the tendons under normal operating conditions. Protecting the deck steel in a thin slab deck supported on a pretensioned girder was also judged unlikely to have any adverse effects on the prestressing strands provided that the applied current density did not exceed 21 mA/m^2 on the deck surface.

The application of cathodic protection to the prestressed tendons in new Italian highway bridge, namely, ''Auto strada del Fejus'' has been reported[39]. This highway has been built-up by the post-stressed tendon technique.

It has been suggested that the application of cathodic protection of ''old'' structures show more risks, because of low reliability in avoiding overprotection conditions. However, inspite of the delicacy of the problem, cathodic protection of reinforcements in prestressed concrete structures has become possible by the development of mesh anodes and new reliable monitoring systems, based on the use of Ag/AgCl reference electrodes stable for long periods.

Protection current density has been designed on the basis of approximately 10 mA/m^2 relative to rebar surface, which corresponds to about 25 to 60 m A/m² relative to concrete surface depending upon steel rebar surface density. The large variation has forced to a careful design, which has been managed by the use of the three different anode mesh type, in order to supply the correct current density for each type of surface.

In this bridge, activated titanium anode mesh "ELGARD" has been used. "Thorotop CP" has been used as the overlay.

The bridge has been divided into "Zones", having separate power supply unit. The current output is automatically regulated by the monitoring system. In case of over protection, the system will switch the power supply off and give alarm signal by suitable devices. Cathodic protection is still in the experimental stage.

5.5.4. Criteria for cathodic protection

5.5.4.1. Reinforced concrete structures: Cathodic protection has been successfully applied for the protection of buried pipelines for which sufficient current is supplied to maintain a minimum steel potential of -850 mV with respect to copper/copper sulphate electrode[40]. But in the case of concrete structures, the half-cell potential of the reinforcing steel varies with temperature, moisture, oxygen and salt contents. And also, problems arise due to concrete resistance flucutations, as dry concrete has a high resistance and wet concrete has a low resistance.

If the applied voltage is kept constant less current will flow in dry conditions. But in the case of moist conditon, more cathodic protection current will be delivered, which cause premature deterioration of the anode and surrounding concrete, and affects the bonding of the reinforcing steel[19,24,41,42,43]. Because of these fluctuations, working out a single potential criterion of cathodic protection for concrete structures is considerably more difficult than that for bare steel in soils or in seawater. For reinforced concrete, many potential criteria have been reported by various workers[44-55].

1. An "instant off" potential of -850 mV versus Cu $CuSO_4$ electrode
2. An "instant off" potential of -770 mV versus Cu $CuSO_4$ electrode
3. An instant off potential of a minimum of 300 mV more negative than the original static potential
4. A potential decay of atleast 100 mV when measured over a period of 4 hours between the instant off potential and the fully depolarised potential
5. That potential at which a change in the slope occurs in the potential versus log current curve (E-log I).

The E-log I, 100 mV polarised potential decay and the 300 mV potential shift are reported to give similar current requirements, whereas, achieving -770 or - 850 mV vs Cu-$CuSO_4$ electrode over the whole structure results in over-protection and excessive current[56]. It has been suggested that in the case of 300 mV potential shift criteria, the polarised potential of the steel in concrete should be less than the limit of -1100 mV vs Cu-$CuSO_4$ electrode.

It has been reported that the 100 mV polarised potential decay is the most reliable criterion for long-term monitoring. Based on the same, the NACE Task group T-3K-2 on corrosion of steel in concrete has selected a 100 mV polarised potential decay as the only potential criterion.

The Strategic Highway Research Program (SHRP) has done studies in the area of protection of prestressed concrete in addition to reinforced concrete structures[57]. The performance of seven cathodic protection systems installed on two mild steel reinforced concrete bridges was assessed. Based on the data and testing conducted for two years, it has been suggested that the polarisation decay of 100 mV may have resulted in under-protection. It has also suggested that the E-log I criterion can be a realistic method for determining the operating current required for cathodic protection[24,42].

5.5.4.2. Prestressed concrete structures

(A) It has been suggested that the protective potential value more than -1100 mV vs copper/copper sulphate electrode may lead to hydrogen embrittlement problems in the steel tendons of the prestressed concrete[58].

(B) It has been reported that the strands of a pretensioned tendon in a concrete slab, which were initially activated by an impressed anodic current followed by excessive cathodic polarisation (-1300 mV vs SCE) for 36 days did not exhibit any significant difference in average fracture load compared to strands of the same tendon polarized to 400 mV[59]. It has also reported that there is no hydrogen embrittlement problem in pretensioned tendon in concrete.

(C) The Strategic Highway Research Program (SHRP) has done research in the area of protection of prestressed concrete in addition to reinforced concrete structures[57]. Based on the experiments, it has been concluded that hydrogen gas was produced and the same diffused in the 7-wire strands when subjected to potentials greater than -900 mV with respect to Saturated Calomel Electrodes. The diffused hydrogen gas into 7-wire strands reduced the fracture stress compared to a control specimen.

These observations show that a cautious approach is necessary while considering cathodic protection of prestressed concrete structures.

5.5.5. Studies carried out by CECRI, Karaikudi: Central Electrochemical Research Institute, Karaikudi has carried out laboratory studies on various aspects of cathodic protection of steel in concrete. A brief summary is given below:

5.5.5.1. Studies in aqueous solutions

(A) Studies have been made in aqueous solutions representing good quality (0.04N NaOH) and bad quality concrete medium (0.001N NaOH) containing different amount of chloride ions from 100 ppm to 10,000 ppm[60]. Mild steel specimen was used as cathode and a large mesh type titanium substrate insoluble anode (TSIA) was used as anode. Saturated calomel electrode was used as reference electrode.

It has been found that the potential of the mild steel was more negative than -300 mV suggesting the medium is highly corrosive. Cathodic polarisation studies have shown that excessive polarisation, i.e., more negative than -1100 mV can lead to the evolution of hydrogen gas, which can cause loss of bond between the steel and the concrete and consequently, embrittlement of the steel can occur.

It has been observed that in solutions representing good quality concrete, complete protection of steel was achieved even at very high chloride concentration. It indicates that the cathodic protection current of 0.1 to 0.4 mA is sufficient to protect the steel from corrosion.

In the case of solutions representing bad quality concrete, it has been observed that a negative shift of 300 mV from corrosion potential was insufficient. A minimum negative potential shift of 400 mV has been found necessary under these conditions for attaining complete protection.

(B) Potentiostatic studies have been carried out in aqueous alkaline media representing different concrete environmental conditions, such as, pH 8, 10, 11 and 12.4 for finding out a potential criteria for cathodic protection of steel in concrete in simulated environment using aqueous sodium hydroxide solutions[61]. Chloride and sulphate concentrations in the solutions were varied from 100 to 35,000 ppm. A mesh type cylindrical titanium substrate insoluble anode (TSIA) was used as an anode. Mild steel specimen was used as cathode. Saturated calomel electrode was used as the reference electrode.

This study showed that in the case of chlorides, the maximum shift in potential from open circuit potential was -150 mV at the lowest alkalinity and it was about -100 mV in the highest alkalinity. In the case of sulfates, the maximum shift in potential was around - 90 mV at lower alkalinities and it was about -70 mV in the higher alkalinities. It has been reported that the variation in anion concentration (chloride or sulphate) does not influence the protection current requirement appreciably. It has also been observed that a shift in the potential to -800 mV vs SCE may be optimum for cathodic protection though shifting the potential to -700 mV vs SCE was found to give complete cathodic protection.

(C) Studies have been made in aqueous media representing concrete environment to find out optimum current density for cathodic protection of steel in concrete using Hull Cell technique[62]. The Hull cell was made of perspex and was trapezoidal in plan. Mild steel panel with a stem was used as cathode. A mesh type titanium substrate insoluble anode was used as impressed current anode. Experiments have been carried out in 0.4N, 0.001N and 0.0001N NaOH solutions. Chloride or sulphate was added to NaOH solution to represent contaminated concrete. Desired current was impressed immediately after immersion of the specimen in the electrolyte and maintained continuously for a period of 24 hours without any interuption.

This study showed that the current density requirement for chloride media lies in the range of 10-40 mA/m^2, whereas, in the case of sulphate, it is in between 20-80 mA/m^2. It has been concluded that Hull Cell experiment can be used to obtain the optimum current density for cathodic protection of steel in concrete.

5.5.5.2. Studies in concrete medium

(A) The polarisation behaviour of steel reinforcement embedded in concrete and also the current distribution pattern in the concrete were studied[61]. Galvanostatic studies have also been done in concrete medium. For polarisation studies, two mild steel rods were embedded in a concrete cube. The specimen were cured for 28 days. The steel rebars were then cathodically polarised from -870 mV to -1870 mV with reference to SCE in steps of 50 mV and the corresponding current was noted. For current distribution study, two reinforced concrete model slabs were cast. A Titanium substrate Insoluble Anode of size 100 x 100 mm was centrally placed. The slab was subjected to alternate wetting with 3 per cent NaCl solution and drying. The cathodic protection was continuously applied for a period of 200 days. Concurrently, another model slab, without cathodic protection was subjected to similar conditions for comparison. Open circuit potential (OCP) of control slab was periodically monitored.

From this study, it has been concluded that the current flow to the different region was limited by the electrical resistance of the concrete and the short distance between the anode and cathode. The effective current distribution was restricted to a distance of about 20 to 25 cm from the anode position. It has been suggested that to get equal shift in potential, in all the regions, number of anodes are to be placed with a spacing of about 20 to 25 cm. It has also been suggested that to know the real shift in potential of the steel reinforcements, it is necessary to adopt IR compensation methods.

For the galvanostatic experiments, a steel rebar was embedded in concrete specimen of different mix ratios, such as, 1:1:2, 1:1.5:3, 1:2:4 and 1:3:6. These specimen were kept in 3 per cent NaCl solution and a constant current was passed using TSIA and an auxiliary electrode. Constant cathodic currents of 0.1, 0.2, 0.4 and 0.6 mA were impressed on different specimen. The potentials without IR compensation were continuously monitored for a period of 60 days.

From this study, it has been observed that the steady state potential at the maximum current of 0.6 mA was around -1300 mV. It has been concluded that the applicaion of current to the extent of 0.6 mA and shifting the potential to -1300 mV without IR compensation was ineffective. Suitable IR Compensation technique should be adopted.

(B) A study has been carried out to find the current density pattern in a motel reinforced concrete structure with two different shapes of anodes[64]. A prototype structure comprising of slabs, beams, columns and footings was cast with mild steel reinforcement.Titanium Substrate Insoluble Anodes (TSIA) of mesh and strip type were embedded with cement mortar overlay. Mesh type configuration was used in slabs and strip type anode was used in beams, columns and footings. The potential of the rabar was shifted by impressing different current densities. Instant off technique was followed to measure the shifted potentials.

It has been found from this study that the beam with stripped anode needed the maximum current density of 300 to 350 mA/m^2, whereas, slabs with mesh type anode needed lesser current density in the order of 80 to 140 mA/m^2.

It has been concluded that the cathodic current requirement depends upon the configuration of the structural component and also design of the anode and spacing between anode and cathode. It has been suggested that a mesh type anode was preferable over strip type from the point of view of minimizing the anode current density.

(C) Studies have been carried out to predict the protection potential cirteria in chloride contaminated concrete using Hull-cell type (Trapezoidal shape) model prisms with mild steel reinforcements[64]. One specimen was cathodicaly protected using TSIA while another was unprotected. Both specimen were simultaneously subjected to alternate wetting in acidified 3 per cent NaCl solution (pH : 2-3) and drying, to induce aggressive corrosion. Constant current density was impresed to the prisms and potential gradient was periodically monitored in both wet and dry conditions. From this study, it has been concluded that for dry condition, the potential criteria lies in the range of -670 to -580 mV vs Saturated Calomel Electrode (SCE), whereas, for wet condition, it lies in between -780 to -680 mV vs SCE.

5.5.6. Work done in Gulf: Baluch et. al. collected protection current data for a corrosion damaged reinforced concrete panel as protected by Ferex-100 CP system, over a period of 18 months, with current adjusted to meet NACE standard "100 mV potential shift over four hours" criterion[65]. It has been reported that Ferex-100 CP system controls the corrosion in reinforced concrete members, provided protection current is monitored. It has been concluded that a high degree of variability in protection current required is noted in transiting from the extremely hot summer months with concrete temperatures of 54°C to the cooler winter periods. It has been suggested that a need to develop a criterion as an alternative to the NACE criterion that would be more suitable to the conditions in the Gulf, minimizing monitoring of protection current.

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5.6. Protection of External Cables

Rehabilitation of a prestressed concrete bridge from corrosion distress, requires measures to retard the progress of corrosion of prestressing steel in the body of concrete of the girders and restoration of the loss in prestress to the extent possible. External prestressing through post-tensioned tendons placed outside the structural concrete members is now being commonly used in many countries as a normal method of construction to impart the design prestress in prestressed concrete structures including bridges. Efficacy and safety of this system compared to the post-tensioning of tendons placed inside the concrete body of structural members and grouting after tensioning to develop the full bond between prestressed tendons and the surrounding concrete has been well established. The corrosion protection of external prestressing tendon is of critical importance especially due to the presence of permanent tension, abrasion of adjacent wires, etc.

5.6.1. Protection of strands: Strand has to be sheathed in a suitable material against corrosive attack of the aggressive environment. Cement grouted sheathings provide an invaluable protection against corrosion by creating an alkaline environment with high pH value. Injection of wax into a sheath can also be used. The individual protection consists of simultaneous extrusion of an inner tar-epoxy coat surrounding all the strand wires and an outer sheath of 1.5 mm thick polyethylene. The three tier protection is, thus, composed of,

galvanization
tar-epoxy
polyethylene

All wires are hot dip galvanized and a corrosion protection compound added during standing operations. The round wires have a zinc coating of minimum 270 gm/sq.m. and z shaped wires in 300 gm/sq.m. in accordance with DIN 2078 and DIN 779 respectively. The active corrosion protection compound consists of polyurethane oil with a high content of zinc dust filler.

Self propelled mechanism which initially wraps a cable with an elastomeric sheeting in a winding operation which fills in any existing surface flaws has been used[1]. This is followed by a wrapping of tinned copper and this jacket is totally impervious to the entry of oxygen or water.

5.6.2. Protection of stay cables in cable stayed bridges: External prestressing is analogus to cable stay-wires. In cable stayed bridges the corrosion protection is made by placing the wires inside the tube that is subsequently injected with mortar. In some of the bridges, these stayed cables are covered by polyethylene tube and the tubes are wrapped by plastic cover[2].

Cables can also be protected by wrapping with glass fibre tissue drenched in polyethylene or polyurethane, the voids between the wires being also filled with the resin. A new tubular system for corrosion protection has also been reported[1]. This is an in-situ process and consists of two coats of acrylin resin with glass mat followed by third coat of resin mortar to give the rough texture. Cables can also be protected by galvanizing and voids between the wires completely filled with a durable elastoplastic material to enable periodic visual inspection of cables[3]. Before twisting to required lay length, the wires are passed through specially formulated polyurethane resin[4]. The ends of the cables are fanned out into the metallic socket of the anchor. The socket is then filled with metallic alloy of tested quality.

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5.7. Alternate Material for Prestressing Tendons

The possibility of utilizing fiber reinforced plastic rods as prestressing tendons, in place of traditional steel tendons, in elements of prestressed concrete bridges exposed to corrosive environments has been investigated[1]. A survey was made to available information on the behaviour characteristics of fiber reinforced plastic tendon elements, and in particular those of glass fiber reinforced (GFR) tendon elements. Also, an analytical study was made of the flextural behaviour of concrete elements prestressed by GFR tendons.

Four GFR tendons with Con-Tech systems anchorages were tested, the primary variable being the embedded length of the GFR rods in the anchorages. All the tendons failed by the rods pulling out of the anchorages. For embedded length of 385 (15.2 in) or greater, the failure loads were 90 per cent of the specified tendon strength of 220 ksi, or about 100 per cent of the guaranteed tensile strength of 197 ksi (60 KN/rod).

A low carbon (about 0.4 per cent) induction heat treated steel, introduced to the Japanese prestressed precast industry in 1965 has been used as a prestressing tendon[2]. It has a well-defined yield point above 0.9 fts. Bars upto 17 mm in diameter can have warm formed button heads applied and rolled threads can be applied to any size required. Recently, a silicon-chrome alloy adaptation of the low carbon heat treated steel has been developed.

Results of fatigue, stress relaxation, prestress transfer length and accelerated stress corrosion tests on the new steel indicate that stress corrosion characteristics are similar to those of stress relieved strand.

A new material, namely, "Parafil" is available for prestressing tendons[3,4,5]. It consists of a closely packed essentially parallel, core of high strength, continuous synthetic yarns, contained within a thermoplastic sheath. This sheath maintains the circular profile of the rope and protect. the core from ultra-violet radiation which could cause degradation.

The core can be a number of different materials, but those most commonly used are based on polyester yarns (Type A parafil) or aramid, e.g., Kevlar 29 (Type F) and Kelvar 49 (Type G).

Type A (Kevlar 29) has a Youngs' modulus of about 12 KN/mm^2 and an ultimate strength of about 620 N/mm^2, which means that the ropes can sustain high strains (5-6 per cent). This makes them ideally suited for soil reinforcement. "Paraweb", a related product, is widely used as the reinforcing material for this application.

The stiffer aramid materials are more suited to structural applications. Both Type F and Type G versions have similar strength (1930 N/mm^2), but differ in their Young's modulus. Type F (Kevlar 29) d has a modulus of about 78 KN/mm^2, while Type G (Kevlar 49) has a modulus of about 126 KN/mm^2 (approximately 60 per cent of steel). The higher modulus of Type G parafil ropes means that they reach their peak stress at a strain (1.5 per cent) similar to that in cold-drawn steel wire at failure.

Kevlar achieves its high strength because it is a highly oriented, crystalline material, with high molecular weight. The basic molecule appears to be poly (P-phenylene terephthalamide) or PPT, and consists of long chain hydrocarbons.

The high modulus of Kevlar 49 makes it the most suitable of the core yarns for use as prestressing tendons in concrete. Kevlar 49 is resistant to more forms of corrosion, with two exceptions. It degrades in the presence of ultra-violet radiation and loses strength in the presence of strongly alkaline solutions. Thus, if the Kevlar yarns were used as reinforcement in intimate contact with concrete, degradation could be expected, given the high pH of concrete. However, the outer thermo plastic sheath of parafil ropes serves to protect the core yarn from this source of attack, as well as shielding the rope from ultra-violet light.

Parafil ropes have a number of attractive properties (high strength, high modulus and good corrosion resistance, amongst others), which make them clearly suitable for use as prestressing tendons.

It will be possible to use parafil ropes in applications where the tendency of steel to corrode means that prestressing would be inappropriate, or possible only if expensive measures were taken to protect the steel. Thus, it can be used in or near the ground and also in marine environments. It is also likely that parafil would be used for external tendons in bridge structures.

Another probable use is in the repair of structures. Many structures with inadequate prestress or reinforcement could be repaired by the application of prestress. The difficulty in existing structures has been the provision of a suitable duct to protect the steel tendon. If parafil were to be employed it would only be necessary to provide anchor blocks and deflection diaphragms.

Prestressing system for a pedestrian bridge in the Marienfelde District of Berlin (FRG) was designed as external prestressing without bond and was achieved by means of seven glass fiber tendons[6]. The nominal diameter of the glass fiber bars is 7.5 mm and the working load per tendon is 0.61 KN (134.9 lb). All seven glass fiber prestressing tendons will be permanently monitored by integrated optical fiber sensors.

ACI-ASCE Committee 423 has evaluated the acceptability for general prestressing applications of a low carbon Silicon Chrome Heat Treated (SI. CR.H.T.) prestressing steel produced in Japan[7]. This Steel has stress corrosion resistance properties comparable to those of wires strand and bars confirming to ASTM A421, A416 and A722 respectively and is suitable for general prestressed concrete use.

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5.8. Protection of Ground Anchorages

Different types of corrosion protection systems have been reported for prestressed ground anchorages[1,2]. Tendon of an anchored dam in Algeria was protected by coated tarpaulin, covered by mixture of grease and bitumen, with outer tarpaulin sheath over free length. The fixed length was covered by cement grout. Grease-impregnated glass fibre bandage and outer asphalt wrapping over free length was adopted in a prestressed concrete dam in Czechoslovakia. In Sweden, bitumen coating was used for anchor head in an underground power station crane beam. In West Germany, the anchorage of an underground power station was protected by jute wrapping impregnated with bitumen in free length. In Algeria, road oil loaded with red lead was used to protect the wires of an anchored dam. The free length of tendon in an anchored retaining wall was painted with bitumen in West Germany. In an underground power station, the free length of the tendon in anchorage bed was surrounded by a chemical filler (oil based unsaturated fatty acid polymer). In USA, the tendons in an anchored retaining wall were protected by bentonite/cement grout cover along with outer steel pipe in free length. In addition to that a sacrificial zinc ribbon anode was installed in each tendon. In Malaysia, an anchor outer head was protected by a sealing cap filled with grease. The free length portion contained in polypropylene sheath was surrounded by bitumen. In New Zealand, the wires in an anchorage bed was protected by polypropylene sheathing with a secondary portion of outer tube and mastic infilling in free length. Corrugated tube/grout encapsulation was adopted over fixed length.

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6. SUMMARY AND CONCLUSION

In earlier Chapters the available literature information has been presented in detail under various classified headings. References have also been included at the end of each aspect. Instrument details and addresses of suppliers have also been given wherever possible under monitoring aspects. In this Chapter, a condensed State of Art Report is presented.

6.1. Case Studies of Corrosion Failures

In order to have an overall perspective of failure problems, case studies relating to failure of post-tensioned concrete bridges and pre-tensioned concrete bridges, have been dealt with.

6.1.1. Post-tensioned concrete bridges: Under Post-tensioned concrete bridge, 30 case studies are described. Failure instances have occurred in different countries, like, Brazil, Netherlands, USA, UK, Denmark, Germany, Austria, Switzerland, Taiwan, France and India indicating the global nature of the problem.

Failures have taken place within a very short period, i.e., few days to few months. On the other hand, some long-term failures after 5 years have also occurred. This brings out the unpredictable nature of the problem.

Out of 30 cases cited, 10 cases come under short-term failure. Most of the failures were due to stress corrosion cracking/hydrogen embrittlement. The rest were due to general corrosion. Presence of hydrogen sulphide in the environment has been responsible for hydrogen embrittlement failure in many cases. A mixture of sulphur, lamp black, kaolin and motor oil used during bridge construction led to the failure of prestressing rods with 10 days of tensioning mainly because of H_2S liberation. Galvanized prestressing wires have failed within two days due to cracking in galvanized layer. Post-tensioned wires in a bridge have failed within few days when sodium carbonate had been added to accelerate setting of cement grout. Delayed grouting and imperfect grouting have contributed to accelerated failures. Long-term failures are mainly associated with the external aggressive environment or stray current. Marine atmosphere or seawater has contributed to the severe uniform corrosion of prestressing wires.

According to Transport and Road Research Laboratory, U.K., post-tensioned beams have been less satisfactory because the durability is influenced by the effectiveness of grouting. In an investigation of 12 bridges, voids were found in ducts of 10. Problem was mainly due to water carrying de-icing salts leaking into the ducts.

6.1.2. Pre-tensioned concrete bridges: Compared to post-tensioned bridges, pre-tensioned bridges appear to have performed well. The two case studies from USA indicate that even 0.1 per cent of chloride content in concrete can cause failures. Surprisingly, in both the cases, precast elements have been used. Concrete made from high alumina cement or blast furnace slag cement has contributed to hydrogen embrittlement failure of prestressing steel.

6.1.3. Failure during storage: Several cases of failures of prestressing wires while stored at site have been reported. Presence of nitrates in soil promoted stress corrosion of prestressing wires stored in coil form. In one case wires had failed after contact with clay.

6.2. Forms of Corrosion and Factors Influencing Corrosion

The high strength steel wires used as prestressing tendon in prestressed concrete is cold-drawn and stress-relieved eutectoid steel. Wire diameters are usually in the range of 1.5 - 8 mm. The steel contains 0.7-0.9 per cent carbon and upto 1 per cent manganese. The steel is produced from hot rolled rod or wire stock which is 'patented' to produce a fine pearlitic structure. Cold-drawing to 80-85 per cent reduction in area is used to obtain additional strength and steel is thermally stress relieved.

The prestressing steel because of its high strength and because of the cold-drawn surface is highly defect sensitive. Since prestressing wires are used in smaller diameters (usually 7 mm), even a small reduction in diameter due to corrosion may have profound effect on its performance. Moreover, the wires are permanently kept under tension (equivalent to 80 per cent of its proof stress) during service and this makes them highly vulnerable to dangerous forms of corrosion, such as, pitting and stress corrosion cracking. Under the case studies, it has been shown that prestressing wires may undergo any of the following type of corrosion depending on the environmental and stress conditions. Uniform corrosion, pitting corrosion, galvanic corrosion, stress corrosion cracking or hydrogen embrittlement, corrosion fatigue, microbial corrosion and stray current corrosion.

6.2.1. Uniform corrosion: Prestressing steel may suffer uniform corrosion when it is directly exposed to aggressive environment. Wires stored at site or wires while lying ungrouted inside the duct may be subjected to this general corrosion. Bare steel wire will undergo severe rusting when exposed to marine environment.

6.2.2. Pitting corrosion: Pitting in prestressing steel is much more serious compared to mild steel components. Under tension, pits formed are deep and narrow unlike shallow pits formed in mild steel. Pitting of prestressing steel is associated with break-down of the passive protective film formed by the alkaline cement environment. Break-down occurs in presence of chloride salts. Sulphate salts also can promote pitting corrosion of prestressing steel. Addition of suitable inhibitor admixture to the cement grout can help to neutralise the harmful effects of chloride.

6.2.3. Hydrogen embrittlement/stress corrosion: Case studies of failures have clearly indicated that most of the failure of prestressing tendons are due either to hydrogen embrittlement or to stress corrosion cracking. Stress Corrosion cracking is the spontaneous cracking which occurs due to the conjoint action of static tensile stress and localised corrosion. Corrosion acts as a notch defect and stress raiser since the wire is under 80 per cent proof stress, stress concentration effects occur at the corroded region and a crack is initiated at the tip of the defect. When the stress value reaches the yield value because of stress concentration effect, yielding and crack propagation takes place. Finally, fracture occurs.

Prestressing steel can fail by hydrogen embrittlement when the atomic hydrogen diffuses into the core of the metal and decreases the cohesive strength of metal atoms. Presence of tensile stress in steel accelerates the diffusion rate. At a critical concentration of stress and atomic hydrogen, steel becomes too brittle and fails by decohesion.

Stress corrosion failure and hydrogen embrittlement failure can be distinguished from normal mechanical failure by the absence of necking (cup and cone formation) at the snapped end.

Detailed studies undertaken at CECRI, Karaikudi have revealed that the prestressing steel can suffer stress corrosion/hydrogen embrittlement in any one of the following media:

(a) Ammonium nitrate at temperatures above 40°C
(b) Disodium phosphate at ambient temperatures
(c) Hydrogen sulphide at ambient temperatures and even at very low stresses (40 per cent proof stress)
(d) Ammonium thiocyanate at ambient temperatures and at very low stresses (20-30 per cent proof stress)

It was also found that there was a limiting pH of 9-10 above which probability of failure may be minimal. In many cases, hydrogen embrittlement failure was accelerated under cathodic polarisation indicating cathodic protection of prestressed concrete structures should be cautiously dealt with.

Studies carried out elsewhere have shown that the realistic environments, like, $Ca(OH)_2$ NaOH, NaCl, etc., probability of failure is more in precracked specimen. This indicates that any mechanical defect on the prestressing wire can drastically alter its cracking behaviour.

FIP has brought out a standard test procedure for testing the stress corrosion susceptibility of prestressing wire in 20 per cent ammonium thiocyanate solution at 50°C. Minimum test duration is 200 hours, advisably 500 hours. A pure tensile load equivalent to 80 per cent of maximum load should be maintained throughout the test period.

Eventhough corrosion fatigue has contributed to failure of few prestressed concrete members, no systematic investigation appears to have been undertaken on this phenomenon.

6.2.4. Stray current effects: Stray current can affect the durability of prestressed concrete structures and stray currents can come from nearby electrified railway track, from welding operations involving deliberate earthing, from a nearby cathodic protection system or due to static electrical discharges of lightning. Stray current effects are more severe on prestressing steel compared to mild steel reinforcement. Post-tensioned structures are more sensitive to stray current damage. Stray current can be avoided by using a proper well-insulated return cable in the circuit.

6.2.5. Microbial corrosion: Since the prestressing steel is highly sensitive to hydrogen embrittlement attack, presence of sulphate reducing bacteria (SRB) can provide such conditions. SRB has been identified in the hydroscopic grease used to protect prestressing tendon. Estuary water may contain considerable amount of SRB and when the water leaks into the prestressing tendon, failure may occur. This is a serious problem in the case of ground anchorages buried in soil which may contain SRB.

6.2.6. Factors influencing corrosion

(A) Pre-tensioned Structures

It is well known that lower the water cement ratio, better will be the corrosion protection by concrete cover. Percentage corrosion of steel has been reduced from 70 per cent to less than 10 per cent by lowering the water cement ratio from 0.62 to 0.49. Chloride permeability has been reduced by 80 per cent by lowering the water-cement ratio from 0.51 to 0.40. FHWA has recommended a water-cement ratio in the range of 0.32 to 0.44 for corrosion protection.

Cracks in concrete cover also influence corrosion. Whereas, crack widths upto 0.1 mm did not produce any corrosion, cracks widths in the range of 0.1 to 2.5 mm have produced severe corrosion.

In the case of pre-tensioned concrete pipes and tanks, guniting carried out for rehabilitation purposes if not proper results in accelerated corrosion. If the gunite mortar does not have proper bonding with parent concrete, fine capillaries are produced at the interface. This results in rapid seepage of chloride laden water or ground water.

Chloride sensitivity of prestressing steel is now well recognised and a maximum permissible limit of 0.06 per cent by weight of cement has been specified. FHWA report published in 1987 says that threshold value for chloride tolerance depends on the actual water cement ratio employed. Interestingly, a higher ratio of 0.4 showed a threshold value of 0.26 per cent by weight of cement, whereas, a lower water-cement ratio of 0.28 gave a threshold value of only 0.17 per cent by weight of cement. CECRI studies have indicated that pitting on prestressing steel may be initiated when the pore solution contains about 3200 ppm of chloride.

Curing practice has an important bearing on corrosion behaviour. Permeability of heat cured AASHTO - Quality concrete is measurably lower than that of three day moist cured concrete. However, a minimum cover of 50 mm was found necessary in both the cases so as to keep the chloride level at the depth of steel within limits.

Quality of the concrete cover depends on its pore structure also and it has been demonstrated that high cylinder strength can also be obtained with relatively porous concrete. Hence, measurements of permeability coefficient are essential. DnV has specified a maximum water permeability of 10^{-12} m/sec. for concrete cover.

(B) **Post-tensioned Structures**

There are several factors which can influence corrosion of post-tensioned tendons encased in cement grout and cable sheath. First of all, it is to be appreciated that corrosion can be initiated internally within the cable sheath. In case of poor quality concrete cover and cracks in concrete cover, external environment can diffuse through the concrete cover and initiate corrosion of cable sheath and wires.

Poor and improper grouting with lengthy duct profile, presence of voids in the grout, ingress of moisture and air-borne salts can cause severe internal corrosion. If expansion joints as well as anchorage recesses are leaky, water can directly enter the cable duct through the end anchorages, particularly through those which are anchored at the deck.

Cables left ungrouted over a long period both before and after prestressing usually suffer severe corrosion before grouting is done. If pitting corrosion occurs during these long periods of exposure, subsequent grouting may not be able to offer the necessary passivation protection and arrest corrosion.

6.3. Monitoring Aspects

In the case of prestressed concrete structures, the following aspects need regular monitoring:

(a) Corrosion monitoring of non-prestressing steels
(b) Corrosion monitoring of pre-stressing steel in pre-tensioned as well as in post-tensioned systems
(c) Examination of cement grout
(d) Monitoring the cracking behaviour and
(e) Monitoring the structural response

6.3.1. Corrosion monitoring of non-prestressing steel and of prestressing steel in pre-tensioned concrete

6.3.1.1. Open circuit potential: ASTM C 876-80 has prescribed a standard test method for half cell potentials of steel in concrete and indicated the various potential ranges for predicting the porbability of corrosion. CECRI has made use of this technique in many bridges and structures. OCP measurements by itself may not serve as a reliable criterion for assessing the condition of rebar since many parameters, like, moisture content, coating to steel, coating to concrete influence the value. Equipments, like, path finder, potential wheel, data bucket are available and all these suffer from the same limitations. Overlooking certain factors will lead to erroneous interpretation. Potential is a thermodynamic criterion and as such OCP cannot be used for quantification of corrosion.

6.3.1.2. Surface potential measurements: Surface potential technique has similar limitations as applied to OCP. At best, it can indicate the most vulnerable region. But by itself, it will not indicate the corrosion probability. It is to be coupled with the resistivity measurements to obtain a parameter called corrosion cell ratio.

6.3.1.3. Concrete resistivity: Electrical resistivity technique can be used as a quality control tool. Periodic monitoring of resistivity can give useful information about the deterioration of concrete. CECRI investigations have shown that the porosity in concrete can be monitored using the parameter ρ_{dry}/ρ_{wet}. This technique can also be used to monitor the durability of coatings on concrete surface. However, electrical resistivity of concrete cannot be directly correlated to corrosion rate.

6.3.1.4. corrosion cell ratio: Corrosion cell ratios are calculated from surface potential and resistivity data. CECRI has made use of this ratio for assessing the probability of corrosion. With proper interpretation, it can give some useful information. It is not useful for quantifying the actual corrosion damage.

6.3.1.5. Electrical resistance probe: Since the electrical resistance of any metal depends on its cross-sectional area, a decrease in cross-section due to uniform corrosion can be easily evaluated by periodically monitoring the changes in the electrical resistance. Sensitivity of measurement very much depends on the thickness of specimen. Thinner the dia, more sensitive will be the technique. Temperature compensation is also necessary. Because of those two factors, this technique cannot be directly applied on the steel reinforcement network. Usually, probes/sensors are suitably designed and embedded in concrete along with the steel rebars at the same cover depth. Probe element is so sensitive that even a small change in dia due to corrosion by the surrounding concrete environment can be readily assessed. CECRI has designed a suitable probe for use in concrete and also designed the monitoring instrument. The equipment is commercially available. Locations for installing probes are to be judiciously selected by the bridge authorities and they are to be made accessible by a suitable terminal system. Such probes have been installed in few girders of Pamban bridge in Tamilnadu.

Major disadvantage of this technique is its insensitivity towards pitting and stress corrosion cracking which are highly localised forms of corrosion. Advantage of this technique lies in its ability to give reliable data on reduction in diameter due to corrosion or the rate of corrosion whichever is desired.

6.3.1.6. Polarisation resistance technique: This is a well known electrochemical technique which can give instantaneous corrosion rate of metal under evaluation. However, application of this technique to reinforced/prestressed concrete structures poses some field problems. Electrical resistance of concrete is a variable parameter which introduces errors in calculation unless properly compensated by suitable instrumentation technique. Reliability of the method for large scale field application has not been proved. CECRI has tried this technique for quantifying the corrosion of sheet pile embedded in mortar in one of the marine structures in India. Monitoring is to be continued further to prove its reliability.

6.3.1.7. Impedance technique: Impedance analysis is slowly emerging as an useful tool and in recent years A.C. impedance spectroscopy is being experimented for in-situ quantification of corrosion of steel in concrete structures. The major problem which is yet to be solved is the irregular distribution of the electrical signal applied with a counter electrode of much smaller dimension compared to that of the large steel network embedded in concrete. Extensive simulation experiments are needed to facilitate accurate interpretation of the impedance data obtained in field measurements. Field measurements are yet to be developed.

6.3.1.8. Noise analysis: Electrochemical potential noise technique is an emerging technique which can be used for estimating the instantaneous corrosion rate. This technique has the ability to distinguish pitting or localised corrosion and general or uniform corrosion. Major problem coming in the way of its effective field utilisation is filteration of noises from external disturbances. Feasibility is yet to be established.

6.3.2. Corrosion monitoring of prestressing steel in post-tensioned concrete: Available literature information indicates that no fool-proof non-destructive technique has so far been developed to quantify corrosion of prestressing steel in post-tensioned concrete. Radiography as well as endoscopy at best can indicate the condition of grout and tendon in a qualitative way.

6.3.2.1. Electrical resistance technique: CECRI has carried out corrosion surveys on a few post-tensioned prestressed concrete bridges in our country. Prestressing steels were actually found to have severely rusted and in some cases even snapped. An electrical resistance technique was adopted to assess the condition of prestressing steel. Collected data indicated satisfactory correlation. However, there are certain uncertainty factors associated with this technique. For example, this technique may not be able to distinguish highly localised corrosion. If these factors could be properly understood and identified by carrying out laboratory simulaion studies, this technique may prove to be quite rewarding.

6.3.3. Direct examination of cement grout and prestressing steel

6.3.3.1. Radiography: Radiography has been extensively used in France since 1968 for examining the quality of grout in cable ducts. Upto 1985, gamma radiography with radioactive sources particularly Co 60 had been used. However, since 1985, linear accelerator radioscopy with X-rays has been found much safer and to yield better quality pictures. Radioscopy can give qualitative information on the following:

(a) Total or partial absence of grout
(b) Presence of broken or corroded wires
(c) Heterogenity of concrete
(d) Cracks in concrete and
(e) Lack of compaction

According to RILEM, detection of reinforcement corrosion is possible only when the thickness of corrosion product is greater than 0.2 mm.

British Standard BS:1881 (Part 205)-1986 has recommended that upto 50 cm thickness of concrete gamma ray sources may be employed and above 50 cm, the use of high energy X-rays is more appropriate.

Atomic Energy Research Establishment, Harwell, U.K. has used the Super-X linear accelerator manufactured by the Radiation Dynamics Ltd., U.K. for the radiography of Swathling Bridge (Reinforced concrete). The following defects in the concrete were identified. Loose compaction and cracks.

Extremely stringent safety precautions are necessary in such radiographic investigations.

6.3.3.2. Endoscopy: This is a semi-destructive type of examination. 20 mm dia holes are to be drilled through the concrete to reach the cable duct, wherever direct examination of grout is needed. Drilling holes should be done with due care to avoid cutting of non-prestressing steel. This technique has been successfully used to identify ungrouted locations and to regrout them. Quantification of corrosion damage to prestressing steel is not possible eventhough qualitative assessment can be made. This technique has been applied on a post-tensioned concrete bridge in Denmark.

6.3.4. Detection of defects in concrete

6.3.4.1. Ultrasonic pulse velocity: It is fairly well known non-destructive test method of assessing the quality of concrete with regard to homogenity, strength, voids, internal cracks, etc. Great care and sound experience are quite essential for accurate instrumentation and correct interpretation.

6.3.4.2. Ground penetrating radar technique: This is a remote sensing non-destructive type of technique useful for identifying deteriorated areas. High frequency electromagnetic pulses are sent into the concrete and reflected pulses analysed.

6.3.4.3. Infrared thermography technique: Infrared camera is used for detecting emitted thermal radiations from the concrete. It produces video signal and records thermal imagery on video tape capable of measuring temperature differences as low as 0.5°C. Internal features, such as, voids or delamination will influence the rate of heating and cooling and these defects may be identified from a study of temperature contours.

6.3.4.4. Holography: This is a sensitive non-destructive technique for detecting micro-cracks (upto 0.1 micron size) in concrete. High energy laser beam is used. However, the technique is highly sensitive even to small vibrations and displacements. It is also not suitable for corners, joints and curved portions. Due caution is necessary in interpreting the complex pattern of interference fringes.

6.3.4.5. Petrography: Petrography is nothing but closer examination of a thin section of concrete core sample taken from the structure under a suitable microscope. This is a destructive type of test as core sample is to be taken. However, it can give useful information about the mechanism of concrete deterioration, viz., leaching, sulphate attack, carbonation, etc.

6.3.5. Monitoring the structural response

6.3.5.1. Vibration analysis: This is a non-destructive technique which can be used to monitor the dynamic response of the structure. It is possible to arrive at the stiffness factor for any bridge deck system provided it is possible to accurately monitor the vibration response. The major problem lies in estimating the fundamental frequency. Eventhough loss of stiffness indirectly implies loss of strength, the same need not be true in all cases. Loss of strength can still occur without producing any measurable loss in stiffness.

6.3.5.2. Strain analysis: Long-term structural response of bridges can be evaluated by comparing the behaviour of the structure with static analysis. The importance of any differences with regard to the safety and serviceability of the bridge can be assessed. Surface strains can be monitored by measurement of bilateral displacement of attached metal-marks or built in pins by deformometers. Internal strains in the bridge structure can be monitored by built-in vibrating wire gauges. If the total measured strain corresponds to the theoretical strain, the behaviour of structure can be considered normal. Otherwise, further analysis is carried out to determine the factors responsible for unexpected strains.

6.3.5.3. Deflection analysis: Since the limit state of deflection is one of the design parameters influencing serviceability of bridge structures, deflection response can be monitored to assess the condition of the structure. Not much information is available in the literature regarding the influence of corrosion of reinforcement or corrosion of prestressing steel on the deflection characteristics. CECRI has carried out some studies on simply supported reinforced concrete beams and observed the corrosion-deflection behaviour. Extensive simulation model studies are necessary if such relationships are to be extended to realistic structures.

6.3.5.4. Acoustic emission technique: This non-destructive technique has generally been used as a diagnostic method for evaluation of cracks in concrete structures. Eventhough FIP has stated that this technique can be used for detecting the failures of prestressing strands, no data is available. Though, it is possible to follow propagation of cracks, no direct correlation has been established with regard to corrosion aspects.

6.3.5.5. Optical fibre sensors: Optical fibres based on hydraulic binders, glass or silica are embedded in the zones where cracks are more probable and then the behaviour of structure is monitored by two diodes, one light emitting and another recurring photo diode. Cracks can be detected by suitable alarm signals. No literature information is available with regard to the application of this technique in prestressed concrete structures.

6.4. Protective Aspects

Protective aspects should cover the structure as a whole including non-prestressing steel reinforcements and prestressing steel. It should also consider pre-tensioned structures as well as post-tensioned structures. Overall cost effectiveness (cost-benefit ratio) is of prime importance apart from proven performance. From the point of view of quality assurance and acceptance, each protective system should be subjected to certain standard acceptance tests before large scale use.

6.4.1. Coating to steel reinforcement (non-prestressing steel): Three different types of coatings are currently being used.

Galvanizing is a metallic coating which was initially preferred since zinc can give sacrificial protection to steel. This was used in few bridge decks in USA for experimentation and evaluation. Based on the performance data, it has been concluded that galvanizing at best can only delay the onset of corrosion and cannot stop corrosion. R&D investigations carried out at CECRI and elsewhere show that zinc has a very low tolerance to chloride attack in alkaline medium and at sufficiently high pH, zinc can undergo dissolution.

Powder epoxy coating is now increasingly used in USA, UK, Japan and other countries. To be really effective, each coating system should pass the accelerated corrosion test (impressed voltage test), chemical resistance tests and other tests specified in ASTM A775 M-84 standards. Powder epoxy coating acts as a barrier coating and hence any defect in the system is likely to promote localised corrosion. Bonding strength is another aspect which should be carefully considered while choosing any powder epoxy system. Though, this coating is claimed to give satisfactory corrosion protection, severe corrosion of epoxy coated rebars in a Florida bridge (USA) has also been reported. Increased corrosion activity in terms of potential has also been reported in several bridge case studies.

Japanese specification cautions that powder epoxy coating can undergo degradation on exposure to sunlight and recommends storing of epoxy coated bars under shade. In this connection, it is worth pointing out that no data is available with regard to its long-term durability in a tropical country, like, India. Few of the powder epoxy systems are found to have poor alkali resistance.

Inhibited and sealed cement slurry coating is being utilised on a large scale in India. It is a package system which takes into account the Indian field conditions and as an in-situ process, this differs from galvanizing and powder epoxy which are factory processes. Extensive long-term field trials have proved its performance and a durability factor of 25 has been obtained even under field exposure at Mandapam, Tamilnadu (one of the most aggressive sites in the world) with precracked and cantilever loaded model slabs. This is a passivating system unlike galvanizing or epoxy and hence has a higher tolerance towards defects. Compared to powder epoxy coated rebars, galvanic current flow under impressed voltage test is almost negligible in cement slurry coated bar. Undercutting is also absent. In this connection, it is worth pointing out that this inhibited cement slurry system also passes the impressed voltage test and chemical resistance tests specified in ASTM A775 M-84. As an in-situ process, this system should be applied to rebars after completion of bending and shaping operations. In fact specifications for galvanizing and powder epoxy coating also recommend treatment of bars after fabrication, since these coatings may get weakened at the bent portions.

6.4.2. Coating to prestressing steel embedded in pretensioned concrete: It has been reported that the cement slurry coating provides an excellent bonding medium besides protecting the steel. In the case of epoxy system, it has been reported that the coating by itself may not develop bond resistance between strand and concrete and hence it shuld be used with spraying of grits to improve bond resistance. Galvanizing of prestressing steel is not advisable since any crack in the coating under tension can lead to hydrogen embrittlement problem which is more serious in nature. Bituminous and metallic paints are unreliable. Fusion bonded epoxy coating appears promising. Many researchers have recommended a coat of neat cement slurry to the prestressing steel after stressing process and before concrete is placed.

6.4.3. Protection of prestressing steel in post-tensioned concrete

6.4.3.1. Temporary protection of prestressing steel: Use of water soluble oil or a more durable packing has been recommended for protection during transit and storage at site. Vapour phase inhibitor has been suggested both for protection during transit and storage at site and also for temporary protection while lying in cable duct.

CECRI's investigations have shown that the prestressing steel can be effectively protected by keeping it immersed in a passivating solution while lying in duct. Same method can be adopted for storing the coils at site.

6.4.3.2. Protection of cable sheathing: It has been reported that the conventional light corrugated metal sheaths can get easily perforated by corrosion. Both high density polyethylene and polypropylene tubes have been recommended. Polypropylene is reported to be slightly superior for tropical regions. There is some controversy about the use of non-metal sheathing. Alternatively, metallic sheaths can be protected with either cement slurry coating or powder epoxy coating.

6.4.3.3. Protection by grout: High quality, dense and impervious cement grout can provide satisfactory corrosion protection provided the prestressing steel is completely encased. Cement grout should not contain any corrosive salts, such as, chloride, sulphate, sulphide and nitrate. It is desirable to use an inhibitor admixture. CECRI studies have shown that a patented inhibitor admixture can be quite effective. OPC with inhibitor admixture can thus be an efficient grout material. Workability aids, such as, plasticisers should be free from corrosive salts, viz., chloride, sulphate, sulphide and nitrate.

Epoxy or polyester resin based grouts have also been used under some specific situations. But in general portland cement grout has been preferred.

In the case of unbonded tendons, grease with spirally wound kraft paper has been used. In many instances, this system appears to have resulted in corrosion of tendons. Hydrophobic greases or inhibited greases are preferable for corrosion protection of unbonded tendons. Inhibited petroleum jelly or lithium based greases may be suitable.

6.4.3.4. Anchorage protection: Anchorage end is the most vulnerable location. Sealing the anchorage with the conventional type of cement based mortar or concrete may not be effective. The following arrangement has been suggested. A specially made covering of rigid metal or non-metal should be fixed to the end anchorage so as to completely encapsulate the tendon and grips. An epoxy based bonding agent should be painted on to the sides of the anchorage pocket/recess and then this pocket should be filled with chloride free cement mortar or epoxy based cement mortar.

6.4.4. Surface coating for concrete: In the literature, many general types of coatings, such as, epoxy based, coaltar epoxy based, polyurethane based, acrylic resin based, furan resin based, chlorinated rubber based paints have been suggested for protection of concrete surfaces. Surprisingly, detailed information about the composition and the actual field performance data along with cost particulars are not available for any of the above types. Eventhough polyurethane coatings are claimed to be having superior chemical/weather resistance as a general class, actual studies have shown that urethane coating does not have any protective value. Similarly, concrete sealers may not have any protective value.

CECRI has evolved three acceptance tests for comparing the performance of different commercially available protective coatings to concrete. These tests are: (a) precracked cantilever loaded model slab test, (b) electrical resistivity measurements on coated concrete surface, (c) adhesion test. Based on these tests, 10 proprietory systems were evaluated and the top performer was identified.

6.4.5. Cathodic protection: Cathodic protection of reinforced bridge decks is fairly established in USA. It has been accepted as the only fool-proof technique for rehabilitation of bridge decks. However, a widely acceptable potential criteria has not yet been developed. Even the NACE recommendation is reported to have resulted in under protection in many bridges.

In the case of prestressed concrete structures, application of cathodic protection is rather complicating. There is controversy regarding the influence of cathodic protection on the hydrogen embrittlement failure of prestressing steel. Here also potential criterion is confusing. It is worth mentioning that even in USA, only recently R&D efforts are being focussed on prestressed concrete structures.

CECRI has evolved its own protection potential criterion for reinforced concrete structures. It has also successfully used Titanium Substrate Insoluble Anodes for impressed current cathodic protection. Large scale experiments are to be carried out to evolve the most suitable design criteria for our field conditions.

6.4.6. Protection of external cables: The corrosion protection of external prestressing tendon is of critical importance especially due to the presence of permanent tension, abrasion of adjacent wires, etc. In fact, external prestressing is analogous to cable stay wires. Conventional protective systems recommended for bare steel structures exposed to atmosphere do not satisfy the high functional requirements of external prestressing and cables in such situations have been protected by wrapping with glass fibre tissue drenched in polyethylene or polyurethane. The voids in between the wires should also be filled with the resin. A more elaborate system uses multilayers of glass mat and acrylic resin. There is also an opinion that the cable surface should be readily accessible for visible inspection and hence galvanized wires with an elastoplastic filler material should be used. Thus, it can be seen that options are vide open.

6.4.7. Alternate material for prestressing tendons: A new material based on polyester yarns called 'PARAFIL' or 'ARMID' appears quite promising. Close packed high strength synthetic yarns are contained in a thermoplastic sheath. However, these yarns undergo degradation due to ultraviolet radiation and lose their strength in strongly alkaline solutions. This material can also be used for external prestressing. Glass fibre tendons have also been used for external prestressing.

7. SCOPE FOR FURTHER RESEARCH

Gaps in Our Existing Knowledge

A critical examination of the available literature information has led to the following observations:

(a) No adequate attention has been paid to the alkalinity vs chloride relationship in concrete

(b) No reliable non-destructive technique is available to monitor corrosion of prestressing steel

(c) Many of the corrosion monitoring techniques are yet to be tuned to the actual field requirements

(d) Coating systems for steel reinforcement are based on powder epoxy or zinc. A passivating type cement based coating may prove to be more economical

(e) Prestressing steel requires effective protection at every stage right from the manufacturing stage upto grouting stage

(f) No standard acceptance tests are available at present to evaluate the performance of surface coating for concrete under loaded condition

(g) In the case of existing structures, no foolproof method is available to estimate the residual life

(h) Repair and rehabilitation measures are taken on an irrational basis because of (g)

Economical Impact

Reinforced and prestressed concrete structures are supposed to be maintenance-free. Design life of bridges in our country is 50 years (min). But actually, deterioration occurs within 10 to 20 years. Out of more than 1200 bridges, even if 50 per cent of them are to be replaced within 20 years, cost of replacement may be about 6000 crores (at Rs 10 crores/bridge). Annual loss due to this reduced life from 50 to 20 years and earlier replacement works out to Rs 180 crores. If similar exercise is made on the entire reinforced concrete structures, annual loss can be shown to be quite enormous. Hence, there is an utmost need to tackle this problem on war footing.

Since large scale studies are involved, the input for R&D work in terms of manpower, materials, equipment, etc. needs to be quite substantial. In-situ experimental field studies are also to be undertaken on some selected bridges and structures.

Major R&D Areas

The following aspects need immediate attention:

(I) New Structures

(a) Instrumentation to monitor corrosion of prestressing steel/reinforcement
(b) Cost effective corrosion protection system for prestressing steel and reinforcement
(c) Suitable methods of protection of reinforcement including epoxy coating treatment
(d) Suitable protective treatment of concrete surface including impregnation sealing and coating techniques
(e) Protection treatments to prestressing steel against corrosion including VPCI to prevent initial corrosion

(f) Efficacy of water proofing compounds, like, mastic asphalt and performed sheet resins

(II) Existing Structures

(a) Quantification of corrosion rate and estimation of residual life
(b) Cost effective repair and rehabilitation measures
(c) Techniques for corrosion quantification at a later stage
(d) Vacuum injection and other methods of grouting of cablesducts containing voids

R&D Proposals

The single goal of the whole R&D programme is to enhance and ensure durability of reinforced and prestressed concrete structures exposed to aggressive environment.

Four proposals (Annexures 1 to 4) have been prepared taking into accout the four major R&D areas mentioned earlier. Time frame is given in the form of PERT CHART (Figs. 7.1 to 7.4)

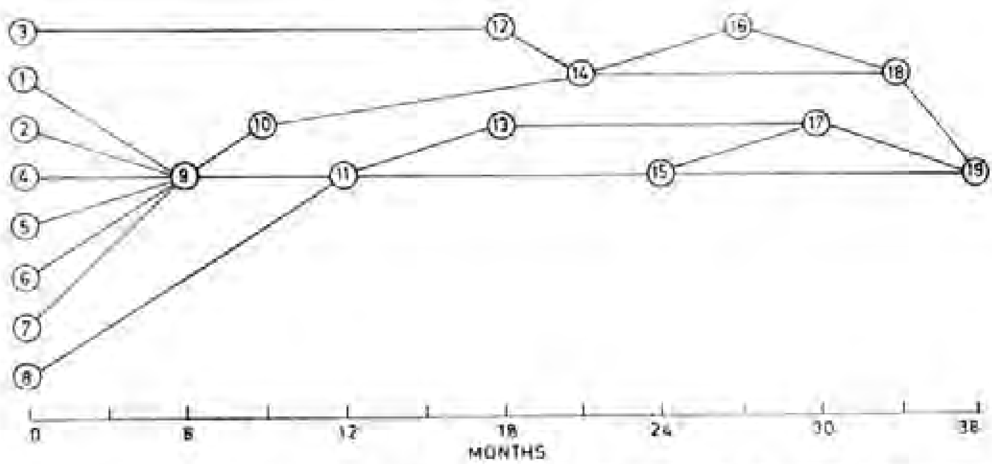

Fig. 7.1. PERT Chart for Instrumentation for Monitoring Corrosion of PSC and RCC Structures

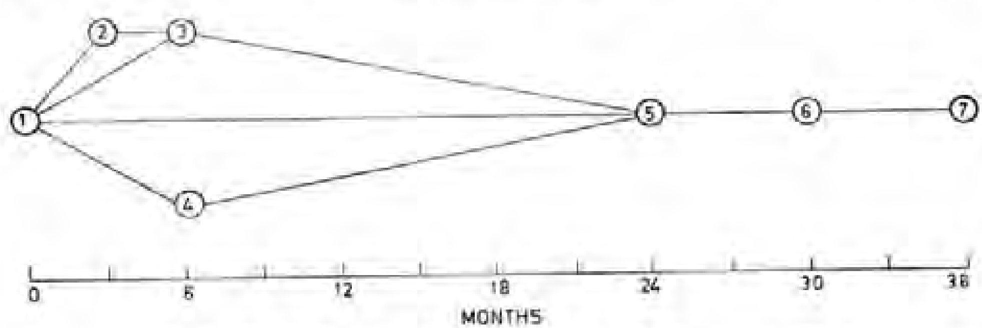

Fig. 7.2. PERT Chart for Corrosion Protection for Prestressing Steel

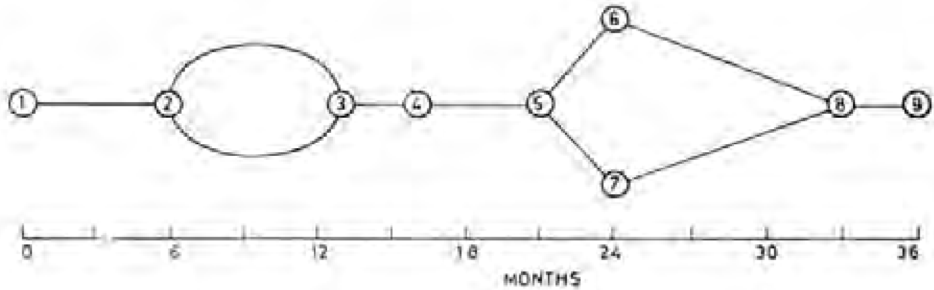

Fig. 7.3. PERT Chart for Standardisation of Repair and Rehabilitation Methods

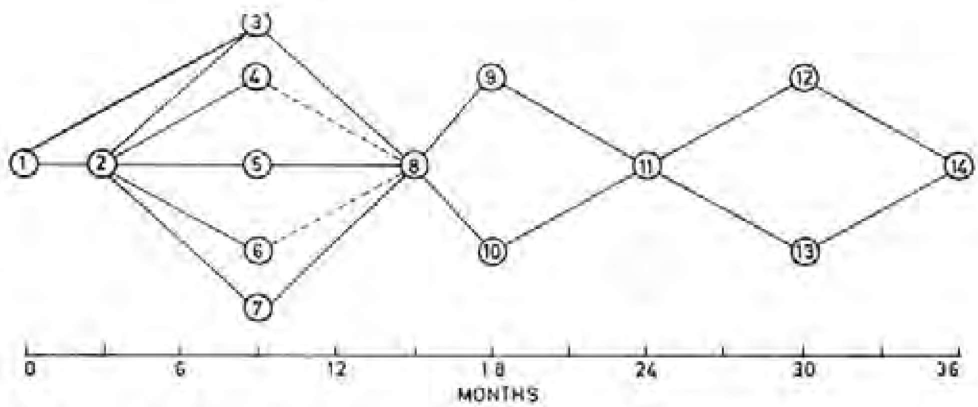

Fig. 7.4. PERT Chart for Quantification of Corrosion Rate and Estimation of Residual Life for PSC and RCC Structures

Annexure-I

R&D PROPOSAL NO. 1

Objectives/Goals

Instrumentation to monitor corrosion of prestressing steel/reinforcement in bridge

Task

1. Corrosion rate measurement based on electrical resistance probe
2. Electrical resistance measurements on prestressing steel
3. Gamma radiographic examination of girders
4. Deflection measurements using precision level with micrometer/water level system
5. Slope measurements using tilt meter
6. Strain measurements using mechanical/vibrating wire strain gauges
7. Measurement of natural frequencies of vibrations using accelerated and FFT analyser
8. Testing of concrete specimens

Action Plan for each Task

			Time in months	Events
Task-1				
	(a)	Procurement of equipments and probes	6	1-9
	(b)	Fixing Probes	6	9-11
	(c)	Measurements	6	11-13
	(d)	Analysis	12	13-17
	(e)	Report	6	17-19
Task-2				
	(a)	Procurement of equipment, etc	6	2-9
	(b)	Taking out electrical connection	3	9-10
	(c)	Measurements	12	10-14
	(d)	Analysis	12	14-18
	(e)	Report	3	18-19
Task-3				
	(a)	Procurement of gamma radiograph	18	3-12
	(b)	Safety precaution & pollution control precaution	3	12-14
	(c)	Exposure of gamma radiograph at bridge site	6	14-16
	(d)	Analysis	6	16-18
	(e)	Report	3	18-19
Task-4				
	(a)	Procurement of equipments, etc.	6	4-9
	(b)	Installation	6	9-11
	(c)	Measurements	12	11-15
	(d)	Analysis	6	15-17
	(e)	Report preparation	6	17-19

Task-5

(a)	Procurement of equipment, etc.	6	5-9
(b)	Installation	6	9-11
(c)	Measurements	12	11-15
(d)	Analysis	6	15-17
(e)	Report preparation	6	17-19

Task-6

(a)	Procurement of equipment, etc.	6	6-9
(b)	Installation	6	9-11
(c)	Measurements	12	11-15
(d)	Analysis	6	15-17
(e)	Report preparation	6	17-19

Task-7

(a)	Procurement	6	7-9
(b)	Installation	6	9-11
(c)	Measurements	12	11-15
(d)	Analysis	6	15-17
(e)	Report preparation	6	17-19

Task-8

(a)	Collection of concrete specimens	12	8-11
(b)	Laboratory analysis	12	11-15
(c)	Report preparation	12	15-19

Annexure-2

R&D PROPOSAL NO. 2

Objective

To identify the cost-effective corrosion protection system for prestressing steel and non-prestressing steel.

Task

1. Procurement of chemicals and equipments
2. Fabrication of anchorage bed
3. Studies on different passivity systems for protecting prestressing steel while lying in cable duct
4. Evaluation of inhibitor admixture for protecting prestressing steel during grouting
5. Studies on corrosion preventive, lacquer coating which can be removed whenever needed for protecting prestressing steel at the manufacturer's end
6. Studies on VPI wrappers for protecting prestressing steel during storage and transit
7. Studies on powder epoxy coating of prestressing steel

Action Plan for each Task

		Time in months	Event
Task-1			
(a)	Procurement of chemicals and equipments	3	1-2
(b)	Preparation of protective chemical system	3	2-3
Task-2	Fabrication of anchorage bed	6	1-3
Task-3	Study of different passivating systems	6	1-4
Task-4	Protection during grouting	18	3-5
Task-5	Protection during manufacture	18	4-5
Task-6	Protection during storage	24	1-5
Task-7	Powder epoxy coating to HTS	6	5-6
Task-8	Report	6	6-7

Annexure-3

R&D PROPOSAL NO. 3

Objective

To evolve and standardise cost effective repair and rehabilitation methods.

Task-1 Procurement of suitable repair materials such as epoxies, polymers, etc.
Task-2 Standardisation of steps for repair and rehabilitation methods
Task-3 Design and casting of model specimen for evaluating repair methods
Task-4 Evaluation of different systems
Task-5 Analysing data
Task-6 Report

Action Plan for each Task

		Time in months	Events
Task-1	Procurement of materials	6	1-2
Task-2	Standardisation steps		
	(a) Repair and rehabilitation of PSC structures	6	2-3
	(b) Repair and rehabilitation of RC structures	6	2-3
Task-3	Design and casting of models		
	(a) Design	4	3-4
	(b) Casting	5	4-5
Task-4	Evaluation of different systems		
	(a) For concrete	3	5-6
	(b) For steel	3	5-7
Task-5	Data Analysis		
	(a) For concrete	9	6-8
	(b) For steel	9	7-8
Task-6	Report	3	8-9

Annexure-4

R&D PROPOSAL NO. 4

Objective

Quantification of corrosion rate and estimation of residual life in existing PSC and RC structures.

Task

Task-1 Collection of design data - identifying worst loading condition/location
Task-2 Designing a prototype model and studying its behaviour. Deflection/vibration/strain
Task-3 Designing similar model with rebars of lesser dia at the vulnerable location and studying its behaviour
Task-4 Designing prototype model and including accelerated electrochemical corrosion - Noting the changes in the load carrying capacity
Task-5 Corrosion survey of an actual bridge using impedance spectroscopy or any other technique - Quantification of corrosion damage
Task-6 Monitoring deflection/vibration/strain of actual bridge and correlating the same with the design data
Task-7 Correlation of 5& 6 for confirmation
Task-8 Monitoring for two years (min) and arriving at the progressive reduction in load bearing capacity
Task-9 Extrapolation of results to predict residual life.

Action Plan for each Task

			Time in months	Events
Task-1	Collection of Design Data		3	1-2
Task-2	(a)	Procurement of equipments	3	1-2
	(b)	Design prototype models	6	2-3
	(c)	Studying for deflections	6	2-4
	(d)	Stydying for vibration	6	2-5
	(e)	Studying for strain	6	2-6
	(f)	Analysis	6	2-7
Task-3	(a)	Procurement	6	1-3
	(b)	Design prototype model	6	3-8
	(c)	Studying	6	5-8
	(d)	Analysis	6	7-8
Task-4	(a)	Procurement and design of prototype model	15	1-8
	(b)	Experimental analysis	6	8-10
Task-5	(a)	Procurement of equipments	6	9-10
	(b)	Quantification of corrosion rate	6	10-11
Task-6	(a)	Monitoring	6	11-12
	(b)	Analysis	6	11-13
Task-7	(a)	Monitoring	6	11-13
	(b)	Analysis	6	12-13