SERVICE LIFE PREDICTION OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO CORROSION

A thesis submitted in partial fulfillment of
requirement for the award of degree of

MASTER OF ENGINEERING
IN
STRUCTURES

Submitted by:
Pinchu Bansal
Roll no:-800922010
Under the supervision of

Dr. Maneek Kumar
Prof. and Head,CED

Dr. Shweta Goyal
Assistant Professor,CED

DEPARTMENT OF CIVIL ENGINEERING
THAPAR UNIVERSITY
(Established under the section 3 of UGC Act, 1956)
PATIALA-147004 (PUNJAB)
INDIA

JULY 2011

CERTIFICATE

This is to certify that the work which is presented in this thesis report entitled "**Service Life Prediction of RCC Structure Subjected to Corrosion**" being submitted by **Pinchu Bansal, Roll No. 800922010** in partial fulfillment of requirements for the award of degree of **MASTERS OF CIVIL ENGINEERING (STRUCTURAL ENGINEERING) at Civil Engineering Department, Thapar University, Patiala,** is a bonafied work carried out by her under the supervision of **Dr. Maneek Kumar**, Professor and Head, Thapar University, Patiala and **Dr. Shweta Goyal,** Assistant Professor, Thapar University, Patiala.

The matter embodied in this report has not been submitted in part or full to any other university or institute for the award of any degree.

Date **(Pinchu Bansal)**

This is to certify that the above declaration made by the student concerned is correct to the best of my knowledge and belief.

(Dr. Maneek Kumar)
Professor and Head, CED
Thapar University, Patiala

(Dr. Shweta Goyal)
Assistant Professor, CED
Thapar University, Patiala

Countersigned by:

(Dr. Maneek Kumar)
Professor and Head,CED
Thapar University, Patiala

(Dr. S.K. Mohapatra)
Dean, academics Affairs
Thapar University, Patiala

ACKNOWLEDGEMENT

A dissertation cannot be completed without the help of many people who contribute directly or indirectly through there constructive criticism in the evolution and preparation of this work. It would not be fair on my part, if I don't say a word of thanks to all those whose sincere advice made this period a real educative, enlightening, pleasurable and memorable one.

First of all, a special debt of gratitude is owned to my supervisors, **Dr. Maneek Kumar** and **Dr. Shweta Goyal** for their gracious efforts and keen pursuits, which has remained as a valuable asset for the successful completion of research work. Their dynamism and diligent enthusiasm has been highly instrumental in keeping my spirit high. The flawless and forthright suggestions blended with an innate intelligent application have crowned my task a success.

I also like to offer my sincere thanks to all faculty members, teaching and non-teaching staff of Civil Engineering Department (CED), and staff of central library, TU, Patiala for their assistance. I am extremely thankful to Mr. Amarjit, Mr. Ram Simran, Mr. Surinder, and all other manpower for helping me carry out experimental work.

I am highly obliged to Dr N.K. Verma and Mr Gurmeet of School of Material Sciences for their support and valuable suggestions in carrying out my research work.

I would also like to thank to my parents, brothers, sisters and my friends for their constant encouragement during the entire course of my seminar work.

PINCHU BANSAL

M.E CIVIL (STRUCTURES)

ROLL NO 800922010

ABSTRACT

The corrosion of embedded steel reinforcement in concrete due to the penetration of chlorides from deicing salts, groundwater or seawater is the most prevalent form of premature concrete deterioration worldwide and costs billions of dollars a year in terms of infrastructure repair and replacement.

The objective is to predict the service life of already existing structures by using simple test parameters. It is intended that the developed model will be simple to use for civil engineers with a basic knowledge on concrete durability.

Service life of the structure is divided into two stages- initiation stage and propagation stage. The initiation stage is the phase during which chloride ions penetrate the concrete cover and reach the reinforcing steel in sufficient quantities to depassivate it, therefore initiating the process of corrosion. The propagation time is the time from corrosion initiation until a specified level of corrosion-induced damage state is attained.

A number of models for predicting the service life of concrete structures exposed to chloride environments. The approaches adopted by the different models vary considerably and consequently, there can be significant variances between the solutions produced by individual models. Most of the models are complex and sophisticated that have been designed mainly for research purposes.

The time-to-cracking, from corrosion initiation to cracking of the reinforcing steel cover concrete, is one of the critical time periods for modeling the time to repair, rehabilitate, and replace reinforced concrete structures in corrosive environments.

Table of Contents

List of Figures

List of Tables

CHAPTER1 INTRODUCTION

1.1 PERFORMANCE OF A STRUCTURE

Concrete structures are designed so that they can satisfy requirements regarding safety, serviceability, durability and aesthetics throughout their design service life. Modern building codes are based on the performance of buildings and this performance is expected throughout the service life of the building. Performance is generally understood as its behavior related to use. It is invariably always a function of time and in addition, depends upon various degradation factors. Degradation means gradual decrease in performance of a material or a structure.

1.2 SERVICE LIFE OF A STRUCTURE

Concrete structures are designed so that they can satisfy requirements regarding safety, serviceability, durability and aesthetics throughout their design service life. Service life is the period of time after manufacture during which the performance requirements are fulfilled. Service life of a structure depends upon the mechanical and other structural parameters like performance, serviceability and convenience in use and aesthetics. Defects in materials may also lead to poor serviceability or inconvenience in use of a structure. Aesthetic aspects are included in the technical requirements, specifically, if the aesthetics defects of the structures are due to degradation of ageing of materials. Service life can also be defined as the time from construction until the chloride content at the depth of reinforcement is high enough to initiate corrosion.

1.3 REQUIREMENT OF SERVICE LIFE IN DESIGN

The theory of durability design is in principle based on the theory of safety (or structural reliability) traditionally used in structural design. In this context, safety denotes the capacity of a structure to resist, with a sufficient degree of certainty, the occurrence of failure in consequence of various potential hazards to which the structure is exposed. However, the theory has been confined more particularly to problems in which time plays only a subordinate part. But in the present theory of safety, the time into the design problems has been incorporated as it allows the possibility of treating degradation of materials as an essential part of the problem. Safety against failure (falling below the performance requirement) is a function of time. Designing a structure with the required safety now includes a requirement of time during which the safety requirement must be fulfilled. In other words, a requirement for the service life must be imposed. In the design of structures the required service life is called the target service life. The level of safety is expressed as the maximum allowable failure probability.

1.4 FACTORS AFFECTING SERVICE LIFE

Service life of a structure is affected by various parameters which further affect the durability models or the service life models. The various factors affecting durability models due to degradation are presented in subsequent section.

1.4.1 Corrosion of Reinforcement

The chemical effect of concrete is attributed to its alkalinity, which causes an oxide layer to form on the steel surface. This phenomenon is called passivation, as the oxide layer prevents propagation of corrosion. The concrete provides the steel with a physical barrier against agents that promote corrosion, such as water, oxygen and chlorides. In normal outdoor concrete structures, corrosion of reinforcement takes place only if changes occur in the concrete surrounding the steel. The changes may be physical, such as cracking and disintegration, exposing part of the steel surface to open air and leaving it without the physical and chemical protection of concrete.

The service life of corrosion-damaged reinforced concrete structures is idealized as a two-phased process: an initiation stage, in which chlorides penetrate the concrete and reach the reinforcement in sufficient quantities to initiate corrosion, and a propagation stage, in which distinct levels of damage build-up are attained. Service life is defined as the time until damage accumulation reaches an unacceptable level or ‘limit state’, i.e., it is the time when the failure probability reaches an unacceptable level. Limit states considered for structures with reinforcement corrosion include corrosion initiation, concrete cracking, spalling and delamination of the concrete cover, and ultimately structural failure.

The most important factors responsible for corrosion of reinforcement are:

1. Carbonation of concrete due to carbon dioxide in air;
2. Penetration of aggressive anions, especially chlorides, into concrete.

1.4.1.1 Carbonation

Concrete is a porous material, and the Carbon dioxide in the air may penetrate through the pores to the interior of the concrete. There, a chemical reaction takes place with calcium hydroxide. The chemical reaction may be described as

$$Ca\,(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \qquad (1.1)$$

It is calcium hydroxide that causes the pH of the concrete to develop, the pH will drop below 9 after the concrete has been totally carbonated. The rate determining process is the diffusion of CO_2 into the concrete.

Corrosion of steel due to carbonation usually occurs particularly in an urban area which has a high level of carbon dioxide, emitted from vehicles and industrial factories. Carbonation in concrete itself do not harm the performance of structure but when carbonation reaches at the level of rebar in RC structure, the high alkalinity of the concrete pore solution is neutralized and hydration products are dissolved then to lower the buffering capacity of hydrations against a pH fall. At this moment, the passivation layer on the steel surface, which otherwise would protect the steel embedment from a corrosive environment, is destroyed, and steel is directly exposed to oxygen and water, eventually to corrode.

The Carbon dioxide penetrates from the surface of the interior of the concrete. Consequently, the carbonation starts from the concrete surface and penetrates slowly to the interior of the concrete. Carbonation depends upon the degree of porosity of the structure. To refine the pore structures, various pozzolanic materials such as pulverized fuel ash and blast furnace slag are used. The rate of carbonation is proportional to the square root of time of exposure (Kropp,1995).

1.4.1.2 Penetration of Chlorides into Concrete

Penetration of chlorides takes place in two stages:

- **Initiation Stage**

  The initiation stage is the phase during which chloride ions penetrate the concrete cover and reach the reinforcing steel in sufficient quantities to depassivate it, therefore initiating the process of corrosion. Thus, the time to corrosion initiation is the time when the chloride concentration at the reinforcement level has reached the ‘threshold value’. The length of the corrosion initiation time is controlled by the depth and quality of the concrete cover (permeability, cracking intensity), the presence of protective systems (membrane, epoxy coating of reinforcement), the rate of chloride ingress, the chloride concentration at the surface, and the chloride threshold level. The duration of the initiation period depends upon the cover depth and the penetration rate of the aggressive agents as well as the concentration necessary to depassivate the steel.

- **Propagation Stage**

  The propagation time is the time from corrosion initiation until a specified level of corrosion-induced damage state is attained. Depending on the type of structure, failure mode and consequence, the time to onset of different damage levels such as longitudinal cracking, spalling/delamination of the concrete cover and a maximum acceptable damage (e.g., flexure or shear failure) can be adopted as propagation times.

Besides Carbon dioxide, chloride ions may penetrate through the pores to the interior of the concrete. Chloride is due either diffusion taking place in totally or partially water filled pores or capillary suction of chloride-containing water. Cement has a certain chemical and physical binding capacity for chloride ions, depending upon the chloride concentration in the pore water. Only the free chlorides are relevant to the corrosion of the reinforcement. After carbonation of concrete bound chlorides are released again, so that the chloride content in the pore water and the risk of corrosion due to chlorides will increase considerably. As a result of the diffusion process, the chloride concentration will decrease from the surface to the interior of the concrete which may affect the service life of the concrete structure.

1.4.2 EFFECT OF CONCRETE COVER

Presence of concrete cover also affects the service life of reinforced structures. If the cover of the concrete will be less than the corrosion will occur at the faster Rate and hence the service life of the structure will decrease. As the cover depth of the structure is less, chlorides will penetrate easily and the time taken by the chlorides to penetrate will also be less.

1.4.3 EFFECT OF TEMPERATURE

Temperature is the important parameter that may affect the service life of the structure. With an increase the temperature, the rate of carbonation also increases which may lead to an effect in the service life of the structure.

1.5 SERVICE LIFE PREDICTION MODELS

The corrosion of embedded steel reinforcement in concrete due to the penetration of chlorides from deicing salts, groundwater or seawater or due to carbonation is the most prevalent form of premature concrete deterioration worldwide and costs billions of dollars a year in terms of infrastructure repair and replacement. A number of models for predicting the service life of concrete structures exposed to chloride environments or for estimating life cycle costs of different corrosion protection strategies have been developed recently. The approaches adopted by the different models vary considerably and consequently there can be significant variances between the solutions produced by individual models. Most of the models are complex and sophisticated that have been designed mainly for research purposes.

1.6 OBJECTIVES OF PRESENT RESEARCH WORKS

Out of various factors that affect service life of RC structure, corrosion of rebar is most important. It is further observed that chloride ions are the dominant cause in rebar of RC structure. Service life of structure due to chloride ions induced corrosion can be divided into initiation period and propagation period. Various researchers have given different models based on initiation and propagation period.

The objectives of the present research work are

a) To investigate the effect of quality of concrete and diameter of rebar on corrosion of rebar by using electrochemical techniques.

b) To investigate upon the suitability of half-cell potential and coorosion current as stable indicators of rebar corrosion

c) To predict service life of RC structures by applying corrosion cracking theory.

e) To evaluate the correlation between permeability of concrete and corresponding corrosion resistance.

1.7 FORMAT OF THESIS

The thesis presentation has been organized in seven chapters.

In the first chapter, scope and objectives of the research work is presented.

Second chapter deals with the various techniques for service life prediction models.

In the third chapter, comprehensive and critical review of the relevant literature justifying the objectives of the research work which is presented.

Chapter four deals with the literature review on the factors affecting service life.

In fifth chapter, the experimental investigation carried out for measurement of half-cell potential, Linear Polarization resistance on slab specimens subjected to external chloride exposure to establish its suitability as a stable indicator of rebar corrosion initiation and also along with it RCPT and permeability test are performed to find the correlation between permeability of concrete and corresponding corrosion resistance.

Chapter sixth deals with the results and discussions of the experiment.

In seventh chapter, service life of the structure is found out by using corrosion cracking theory.

Finally in the eighth chapter, conclusions of the experimental results are given.

CHAPTER 2 VARIOUS TECHNIQUES USED TO CONSTRUCT SERVICE LIFE PREDICTION MODELS

2.1 INTRODUCTION

To predict the service life of a structure, various modeling techniques have been introduced. But there mechanism varies in predicting the service life of a structure.

2.2 LIMIT STATES FOR SERVICE LIFE

Two limit states can be identified with regard to service life (Figure 2.1):

2.2.1 Limit State Based on Initiation Period

The service life ends when the steel is depassivated. This rule is usually applied to all chloride-induced corrosion as the local attack penetration rate is still not safely quantified and uncertainties concerning the propagation period are therefore high. Thus the service life is limited to the initiation period only (time for the aggressive agent to reach the steel and induce depassivation). This rule is also applied to all prestressing steels. The tensile stress of tendons is normally so high that no reduction in the cross-sectional area is permissible and as a result of surface corrosion there is a risk of stress corrosion cracking.

In the cases where no corrosion is allowed, the following formula for service life can be used:

$$T_L = t_o$$

where,

T_L = the service life

t_o = the initiation time of corrosion

2.2.2 Limit State Based on initiation Period and Propagation Period

The limit state is based on cracking of the concrete cover due to oxides generated during corrosion. In this case the service life includes a certain propagation period of corrosion during which the cross-sectional area of steel is progressively decreased, the bond between steel and concrete is reduced and the effective cross-sectional area of concrete is diminished due to spalling of the cover. This approach is applied in cases where generalized corrosion is developing due to carbonation. The service life based on cracking of the concrete cover is defined as the sum of the initiation time of corrosion and the time for cracking of the concrete cover to a given limit.

$$T_{L} = t_{o} + t_{1}$$

where t_1 is the propagation time. The propagation time t_1 ends when a certain maximum allowable loss of the cross sectional area or loss of bond or crack width is reached. These values will depend upon the particular detailing and geometry of each element.

At cracks, originating from the beginning of service life, the initiation time t_0 is much shorter than in an uncracked cover or even t_0=0. In this case it may be written:

$$T_{L} = t_{1}$$

where t_1 is the free corrosion time.

Models for estimating t_0 and t_1 are presented below. When developing these models the assumption has been made that concrete surfaces are free from coatings and sealants.

2.3 MODELING TECHNIQUE FOR CHLORIDE INDUCED CORROSION

Modeling of chloride induced corrosion is nothing but an attempt to quantify the kinetic of chloride penetration using mathematical expressions. The first attempt was made by Collepardi et al. (1972). This mathematical model was first used by the permeability of molecules and then he applied this it on the concrete. For developing the model various assumptions were made before using this Fick's law. These are:

- The amount of chlorides bound by concrete is a function of their concentration in the pore solution and follows a linear relationship.
- The temperature gradients are weak in the material
- The material is kept fully saturated and not subjected to any water movement.

The fundamental laws that help in modeling the penetration of chloride into concrete are:

2.3.1 Ficks First Law of Diffusion

According to Fick's first law of diffusion, the transport of chloride ions in concrete through a unit area of a section of the concrete per unit of time is proportional to the concentration gradient of the ions $\frac{\partial c}{\partial x}$

$$F = -D \frac{\partial c}{\partial x} \qquad (2.1)$$

Where,

D=Chloride Diffusion co efficient which depends upon the parameters like the time for which the diffusion has taken place, location of the concrete, composition of concrete etc.

The negative sign shows that the diffusion of chloride ions occurs in the direction opposite to that of increasing concentration of chloride ions. The amount of chloride ingress into the concrete is the function of time and is defined by Fick's second law of diffusion.

2.3.2 Fick's Second Law of Diffusion

When chloride diffuses into concrete, a change of chloride concentration 'C' occurs at any time 'T' at any point 'x' of the concrete. Fick's second law predicts how diffusion causes the concentration field to change with time:

The governing differential equation:

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2}$$

(2.2)

where,

C = the chloride content,

D = the apparent diffusion coefficient,

x = the depth from the exposed surface, and

t = time.

The solution for a semi-infinite domain with a uniform concentration of C_x, at the surface (x = 0) is

$$C_x = C_s[1 - \mathrm{erf} \frac{x}{2\sqrt{D_e t}}] \qquad (2.3)$$

where,

C_x = the chloride concentration at depth x,

$C_{s=}$ the chloride concentration at the concrete surface,

x= the depth from the surface of the structure,

D_e= the diffusion co-efficient,
t= time.

The initiation of corrosion have said to started when the chlorides at the rebar surface reaches a threshold value (C_{th}). Therefore, the initiation time for corrosion is obtained by modifying eqn. 2.3 as

$$C_{th} = C_s\left[1 - erf\left(\frac{c}{2(Dt_0)^{1/2}}\right)\right] \quad (2.4)$$

When on integeration of Fick's second law of diffusion, constant value occurs. This constant value is then represented by an error function 'erf'.
where
C_{th}= the critical chloride content,
c=the concrete cover, and
t_0=the initiation time of corrosion.

This equation may further be simplified by replacing error function with a parabola function:

$$C_x = C_S\left[1 - \frac{x}{2(3Dt)^{1/2}}\right]^2 \quad (2.5)$$

2.4 LITERATURE REVIEW ON APPLICATION OF FICK'S LAW IN SERVICE LIFE PREDICTION

For the application of Fick's second law of diffusion, several techniques have been used to find solution of the partial differential equation. As the equation involves concentration of chloride ions C_x, as ppm at the exposed surface (at x = 0) and the concentration at a distance x from the surface, C_s, is conveniently measured as percentage by weight of cement (or concrete), the direct application of Fick's law is therefore not possible.

Weyers and Smith (1989) have adopted a nonlinear regression analysis and a finite difference method was used to determine diffusion coefficient by *Funahasi (1990)*. Numerical integration is pursued by *Midgley and Illston (1984)* and an iterative program has been used to determine the value of diffusion coefficient which best fits the data of chloride concentration by *Liam et al. (1992)*. *Nagano and Naito (1985)* provide a solution of Fick's law with stepwise uniform (periodic) functions for C_s at the boundary.

However, some researchers have concluded that the Fick's law can be applied under some conditions only like *Raharinaivo and Jean-Marie (1986)* concluded that the chloride penetration can be modeled by Fick's diffusion law in most cases if other factors such as pressure, evaporation and frost action are neglected. However, when cement has a high proportion of tricalcium aluminates content and when a concrete is of small porosity, Fick's law cannot be applied with full validity.

Weyers and Smith (1989) for studying the influence of w/c ratio, pozzolanic admixture and temperature on chloride ion diffusion used a non-linear regression analysis to determine diffusion coefficient and concluded that Fick's law was applicable in all cases except for silica fume mixture.

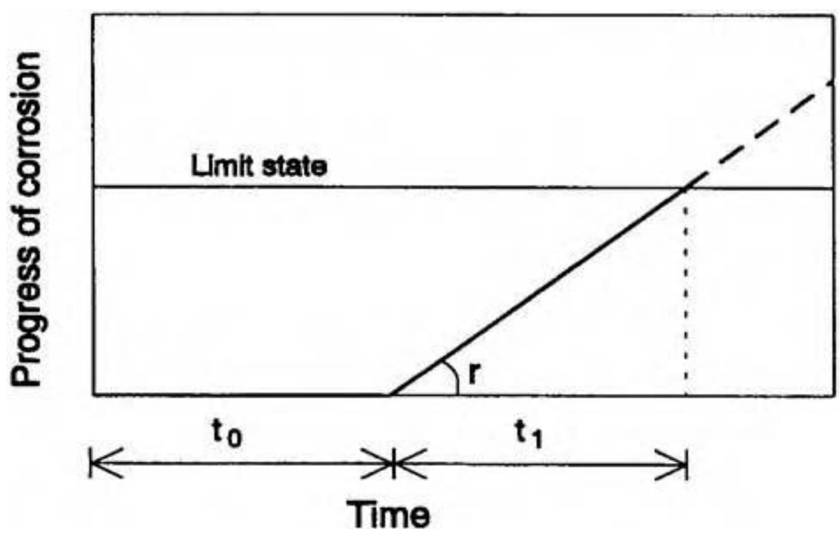


Fig.2.1 Determination of service life with respect to corrosion of reinforcement *(Vesikari and Sarja, 1996)*

CHAPTER 3 LITERATURE REVIEW ON VARIOUS MODELS AVAILABLE

3.1 MODELS BASED ON INITIATION PERIOD

General

As we already know that the service life consists of two phases- initiation phase and the propagation phase. Various scientists have developed different models which predict the service life of the structure that is the initiation and propagation phase. But many models which are developed are based on the prediction of the initiation phase of the structure. The various models are discussed below:

Liang et al (1999) proposed an analytical solution using Fick's first law and second law of diffusion combines with previous laboratory tests and empirical results from existing reinforced concrete structures and compared model predicted values with the experimental result which was performed by Yang (1994). They gave an expression to find out chloride concentration at any point in concrete to be a function of many dimensionless parameters. These parameters incorporated a term for accelerated diffusion, absorption of chloride ions into concrete, function of time, depth at which chloride concentration is required etc.

Liang used the basic equation of Fick's first law of diffusion and second law for predicting the service life of a structure and finally used Laplace transformation Equation which is given by:

C(z,τ) =

$$\frac{1}{2}exp\left[\left\{\frac{\lambda}{2}-\sqrt{\frac{\pi^2}{4}+\alpha}\right\}z\right]\left\{erfc\left[\frac{z}{2\sqrt{\tau}}-\sqrt{\left(\alpha+\frac{\lambda^2}{4}\right)\tau}\right]+exp\left(z\sqrt{4\alpha+\lambda^2}\right)erfc\left[\frac{z}{2\sqrt{\tau}}+\sqrt{\left(\alpha+\frac{\lambda^2}{4}\right)\tau}\right]\right\}$$

(3.1)

$$erfc(x)=1-erf(x)=\frac{2}{\sqrt{\pi}}\int_x^{\infty}e^{-u^2}du$$

where,

erfc(x) = Complementary error function,

α, λ = Constants.

This analytical solution was then compared by an experiment which was conducted by Yang who used a 20× 20 × 25 cm (length × width × height) chloride migration apparatus with accelerated electrical potential to measure the diffusion coefficient in cross sections (10 cm diameter × 2 cm thickness) cut from cylindrical samples of concrete that had been cured in CaO solution for 91 days. It was found out that the values of the diffusion co-efficient were almost same at different potentials with a very little error.

The model was further validated the results by taking the cores from the two places from the existing structures. A concrete specimen with a thickness of 5 cm was bored from a reinforced concrete structure with cover thickness of 15 cm. This specimen was put into a chloride migration apparatus with accelerated electrical potential. After 208 h at an electric potential difference of 9 V, the chloride ion concentration was 0.003 times the critical concentration. From this result, the time needed for chloride ions to diffuse inwards until they reach the steel in the original structure (in other words, the service life) can be calculated. Fig. 3.1 shows the exact analytical solution; when the curve for the 5-cm thick specimen reaches the abscissa time of 208 h, the corresponding value of D_P can be read off the y axis. This value for D_P (7.8 3 10212 m2/s) can then be used in Fig. 3.2, which shows the relationship between D_P and time in the natural state (i.e., with zero effective electric potential difference). Choosing the curve that corresponds to a cover thickness of 15 cm from Fig. 3.2, a service life of 3330 days (9.12 years) can be read off the abscissa.

Another RC bridge with a deck of cover thickness of 5 cm is situated near the coast. Assuming that the degree of average porous saturation of bridge deck is 0.5, and that the environmental chloride concentration is 6% of the chloride concentration of seawater, the predicted time for the chloride front to reach the steel surface is to be determined. A core with thickness 3 cm was therefore drilled from the bridge deck and subjected to an accelerated electric field test. Since the environmental chloride concentration is 0.06 times that of seawater, the concentration front of interest evaluates to 0.05 times the environmental chloride concentration. This value was reached after 160 h at an electric potential difference of 6 V. From Fig. 3.3, this corresponds to a Dp value of 2.9 3 10212 m2/s. This value can now be used in Fig. 3.3 to predict the service life for a cover thickness of 5 cm. The predicted value is 1742 days.

So, the service life of the existing reinforced structures can be predicted by first finding out the diffusion co-efficient experimentally and then comparing this value from the graph, the natural life of the structure can be found out.

Hoseini et al. (2008) proposed a semi empirical model named DuraPGulf model to predict the service life of a structure and to evaluate the effect of various parameters such as mixture proportion, curing regime, exposure condition, temperature etc. on the service life of structure. The model was developed using the Finite Element (FE) technique. It is based on a simplified chloride diffusion equation that is the Fick's second law.

Sprinkel et al (2008) developed a service life model that incorporates the probabilistic nature of the diffusion parameters by using Monte Carlo statistical technique. The ingress of the chlorides into the concrete is modeled using Fick's second law of diffusion. They predicted the service life of a bridge deck using the following three distinct time periods

1. Time to corrosion initiation
2. Time from initiation to cracking
3. Time for corrosion damage to propagate to a limit state.

The time period for corrosion initiation is the time required for the chloride initiation concentration to be surpassed at 2% of the bridge deck reinforcing steel. The total corrosion propagation time is the time required for corrosion deterioration to advance from a level of 2% to a level of 12%. A deterioration level of 12% connotes the end of functional service life (EFSL) for a bridge deck and concluded that bridge decks are most commonly rehabilitated at a damage level of 12% for the worst span lane. The damage level is defined as the sum of the patched, spalled and delaminated areas in the top surface of the concrete bridge deck.

Modeling propagation of corrosion after initiation is a difficult task. Therefore, initiation period is predicted to a level of 2% using Fickian diffusion behavior. The model is developed using Monte Carlo Simulation technique.

3.2 MODELS BASED ON PROPAGATION PERIOD

General

Some models are based on initiation period but some researchers have also predicted models that are based on propagation period and also both on initiation period and propagation period. These are given below:

Li (2003) proposed a model in which he ignores the kinds of studies that deal with corrosion in the "first life cycle" (from the time of installation to the first signs of corrosion) and rather, looks into the "second life cycle" (from the time of the initiation of corrosion to the time the concrete beam or structure is "unserviceable").

Li conducted an experiment in which he used a total of 30 specimens that consisted of a variety of different concrete compositions (different water cement ratios and cement types). He put the tested samples under "simultaneous loading and salt spray" conditions, which were simulated in a big "corrosive environmental chamber" that was constructed exclusively for Li's research purposes. To achieve test results in a short amount of time, Li adopted accelerated conditions; the loads were "kept constant" on the concrete samples until they were removed from the environmental chamber for the testing procedure. It was determined that the corrosion growth was directly related to "crack distribution" as well as the pattern within the test sample itself. The following observations were made during the test.

He concluded from the test that the reinforced concrete flexural members deteriorate at different rates and stiffness deteriorates at a faster rate than a strength; for example, by the time the stiffness deteriorates to about 60% of its original condition, the strength of the concrete substance has only deteriorated to about 10% of its original state (during the same amount of time). Li suggests that the reason for this variation with reference to structural deterioration can be because of the following reasons: environmental conditions; material discrepancy; human error; and workmanship variations. Therefore, Li has taken the "phenomenological approach" – which takes into consideration the uncertainties associated with attempting to focus on statistics related to deterioration of strength and stiffness.

His model, shows that once the reinforced concrete flexural members break down to the point that they are "unserviceable" that tells engineers that there is "less than 15% of service life left."

Liang et al.(2002) proposed a model in which he said that the corrosion process has two stages, the initiation time (t_i) including depassivation time (tp) and the corrosion (propagation) time (tcor). The total service life of existing RC bridges can be expressed as $t= t_i+t_{cor}$. Many mathematical models were applied to calculate each t_i and t_{cor}. The Chung-Shan Bridge, 69 years old, in Taipei, was offered as an illustrative example for the modeling approach and service life predictions.

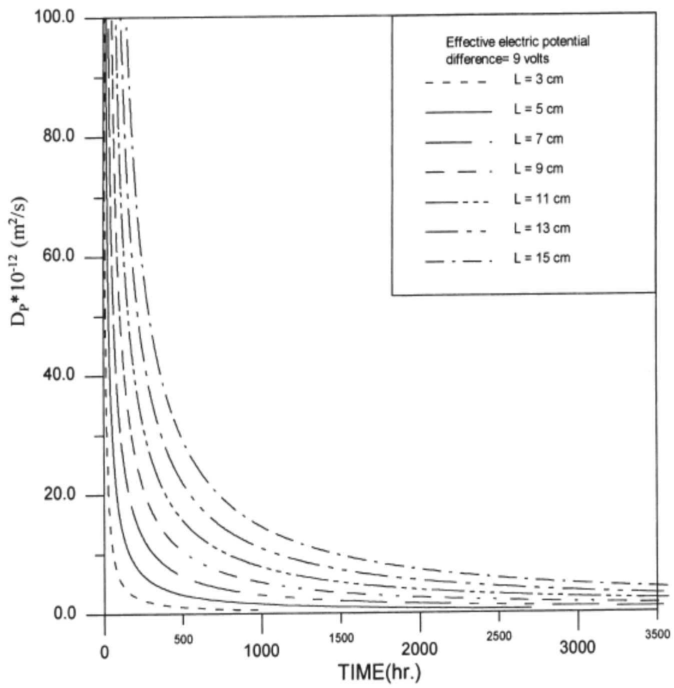


Fig. 3.1. Porous diffusion coefficient at effective electric potential difference 9 V vs. time (d = 1.0) ***(Liang et al, 1999)***

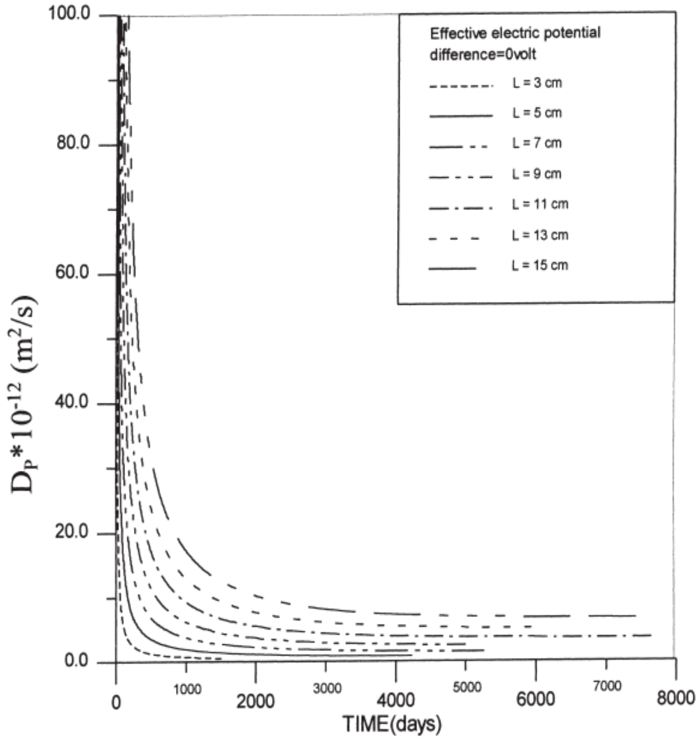


Fig. 3.2. Porous diffusion coefficient at natural state without electric potential difference versus time (d = 1.0); the unit of Dp is m^2/s*(Liang et al, 1999)*

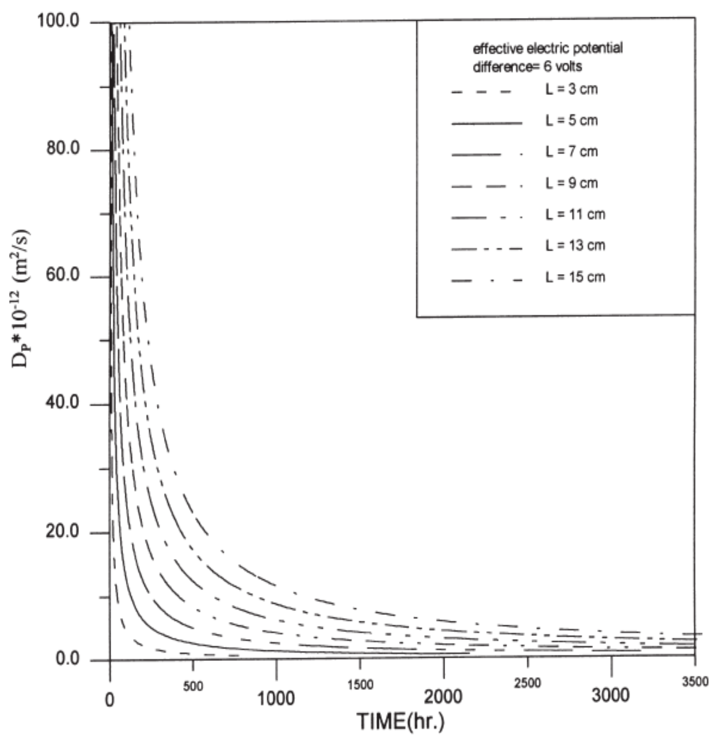


Fig. 3.3. Porous diffusion coefficient at effective electric potential difference 6 V vs. time based on Eq. (32) (*d* = 0.5)(*Lang et al, 1999)*

CHAPTER 4 LITERATURE REVIEW ON FACTORS AFFECTING SERVICE LIFE PREDICTION

4.1 General

Corrosion of reinforcing steel is a major type of deterioration in reinforced concrete structures. Products of corrosion exhibit volume expansion and induce tensile stress in concrete, which ultimately result in cracking and spalling of concrete cover. Due to loss of steel cross-secton and covered concrete, there could be a significant reduction in load bearing capacity of the structures. All these are also mainly affected by various parameters like concrete cover, water cement ratio, type of diameter etc. the effects of all these are given below:

4.2 Effect of water cement ratio

Stewart and Vu (2002) studied the effect of water cement ratio on the service life of the reinforced structures through an accelerated corrosion experiment by keeping slabs at the same cover but having different w/c ratios. The active accelerated corrosion process was achieved by applying an electrical current to the bars. The soffit of the specimen was immersed in a 5% NaCl solution. A current was then supplied to the bar (the anode) by a power supply and the cathode was a stainless steel plate submerged in the NaCl solution. The current regulator kept the current constant over time, in this case equivalent to a corrosion rate of 100mA/cm^2. It was observed that increasing w/c ratio resulted in increased crack propagation rates by up to 30% and 40% for 25mm and 50mm cover respectively. However, as was observed for concrete cover, the w/c ratio appears to mostly influence crack propagation when the crack width exceeds 0.15mm to 0.3mm.

Castro-Borges and Balancán (2011) found the chloride threshold for corrosion onset of reinforced concrete in two tropical marine microclimates. Sixty concrete cylinders were exposed for each tropical marine microclimate. One half of the samples had a reinforcing bar embedded at the center of the sample (corrosion measurements) and the other half was made with plain concrete (chloride). Five water/cement ratios were tested representing the regional construction practices. The corrosion rate was monitored using the polarization resistance technique and the chloride content was determined using an ion selective electrode. The chloride threshold was in the range of 0.43 to 4.26 % by weight of cement for specimens at 50 m and 100m from the seashore and it was concluded that the threshold changed with the concrete quality and the available data anticipate a possible effect of the microclimate.

Baldock et al (2003) summarizes the corrosion results of a 5.3 year study on the corrosion performance of reinforcing steel in concrete slabs containing supplementary cementitious materials and exposed to chlorides through a natural migration process, i.e. a ponding solution of 3.4% sodium chloride on the top slab surface of the concrete slab. The thickness of concrete cover to the steel reinforcing bars ranged from 13 mm to 76 mm. Different classes of fly ash were used and along with it silica fume and blast furnace slag was used and its effect were found on corrosion. Corrosion of the reinforcing steel bars was evaluated using the half-cell potential, linear polarization and AC impedance techniques. It was observed that the concrete incorporating fly ash Class C had the best performance with respect to chloride induced corrosion followed by the concrete containing silica fume and the control concrete with w/c of 0.32 and corrosion rate of the steel bars was relatively low, even with 13 mm concrete cover . The concretes made with Class F fly ash and blast furnace slag performed better than the Portland cement concretes with w/c of 0.43 and 0.55.

Hoseini et al (2008) studied the effect of different water cement ratio on the reinforced structures through an experiment in which 120 concrete prism specimens measuring 15×15×60cm were exposed to the marine environment of Bandar Abbas city in the south of Iran of four water to cement ratios (0.35, 0.40, 0.45, and 0.50). He concluded that the service of the structures with minimum water cement ratio along with superplastizer can be increased to 6years to 10years more.

4.3 Effect of Concrete Cover

Stewart and Vu (2002) studied the effect of concrete cover, with the cover being either 25mm or 50mm.The active accelerated corrosion process was achieved by applying an electrical current to the bars. The soffit of the specimen was immersed in a 5% NaCl solution. A current was then supplied to the bar (the anode) by a power supply and the cathode was a stainless steel plate submerged in the NaCl solution. The current regulator kept the current constant over time, in this case equivalent to a corrosion rate of 100mA/cm^2. It was observed that concrete cover influenced crack propagation for experimental slabs at 25mm and 50mm at different w/c ratios (i.e. 0.45; 0.5; 0.58). It was also noted that the cracking patterns at the cross sections were quite similar for both 25mm and 50mm cover. It was observed that the crack propagation time (tser) at 50mm cover is about 1.15; 1.2; 1.4 times that observed for 25mm cover slabs at 0.45; 0.5; 0.58 w/c ratios, respectively. However, the effect of concrete cover was not significant when crack widths were less than 0.15mm to 0.25mm.

Kim et al (2010) studied the effect of concrete cover on the service life of reinforced concrete structure by performing an experiment on a concrete bridge which was exposed to air containing high level of carbon dioxide for 18 years on sound, cracked and joint parts. He concluded that with the increase in the concrete cover the service life can get increased irrespective of the quality of concrete. When the concrete cover depth is above 60 mm, the service life, even for the cracked concrete, exceeds 50 years. It is also evident that concrete quality is very much dominant in the propagation rate of carbonation through concrete to the position of embedded steel, which subsequently induces corrosion of steel, thereby lowering the performance of concrete structures, due to reduced load carrying capacity of the bridge due to the corrosion of rebars.

In the Liu & Weyers (1996) model the time to crack initiation is the time when stresses resulting from the expansion of corrosion products exceed the tensile strength of concrete. The critical amount of corrosion products needed to cause first cracking consists of two parts: (i) the amount of corrosion products required to fill the total porous zone around the steel/concrete interface; and (ii) the amount of corrosion products then needed to generate the critical tensile stresses. The time to cracking is influenced by corrosion rate, cover, bar spacing, concrete quality, and material properties. Therefore, the model proposed by Liu & Weyers (1996) is assumed herein as suitable for estimating the time to first cracking.

CHAPTER 5 EXPERIMENTAL PROGRAM

5.1 GENERAL

In this chapter, the experimental setup is presented to find out the service life of the structure and to tell the parameters which effect the service life of the reinforced concrete structures. The procedure for monitoring the progression of corrosion of steel in concrete is done namely by electrochemical tests i.e. LPR measurement, half cell measurement.

5.2 TEST PROGRAMME

The objective of test programme is to find out the service life of RC structure having different water cement ratio and different diameters of bars when accelerated corrosion is done by applying voltage of 5V. The test programme involved

1. Determinations of basic properties of constituent materials namely cement, fine aggregates, coarse aggregates and steel bars as per relevant Indian standard specifications.

2. Casting of slabs of plan size 300 x 300 mm^2 with varying depth such that the clear cover is constant and different diameters 16mm, 20mm and 25 mm diameter TMT steel bar using different water cement ratio.

3. Subjecting the slab specimens to accelerated corrosion by providing initial voltage of 5V.

4. Monitoring the slabs till the specimen is severely corroded

An accelerated corrosion technique is used so that testing could be completed within a reasonable time. One power supply is used for this purpose. The power supplies allowed application of a constant voltage‘t’. The power supplies had voltage and current capacities of 150 V and 1000 milliamperes (mA) in increments of 1 V and 1 mA, respectively. The LCD screen displayed the instantaneous voltage and current, which allowed continuous monitoring of the fluctuation in the impressed current with time. **Fig. 5.1** shows a general view of the specimens and the power supplies used to accelerate corrosion.

Fig 5.1 Specimens and the Power Supplies used to accelerate corrosion

All specimens are exposed to the same environmental conditions to elucidate the effect of test parameters on corrosion activity and concrete cracking.

5.3 MATERIALS USED

Cement, fine aggregates, coarse aggregates, water and TMT bars are used in casting of slabs. The specifications and properties of these materials are as under:

5.3.1 Cement

43 grade Portland Pozzolana cement is used for the present investigation. Ultra Tech cement from a single lot is taken for the study. The cement is of uniform colour i.e. grey with a light greenish shade and is free from any hard lumps. Summary of the various tests conducted on cement are given in **Table 5.1.** All the tests are carried out in accordance with procedure laid down in IS: 8112-1989.

5.3.2 Fine Aggregates

The fine aggregates used for the experimental work is locally procured and conformed to grading zone III. Sieve Analysis of the fine aggregate is carried out in the laboratory as per IS 383-1870. The sand is first sieved through 4.75mm sieve to remove any particle greater than 4.75 mm sieve and then washed to remove the dust. The physical properties and sieve analysis of fine aggregates are shown in **Table 5.2 and 5.3.**

Table 5.1: Physical Properties of Cement

S.No.	**Characteristics**	**Values obtained**	**Standard values**
1.	Normal Consistency	33%	-
2.	Initial Setting time	48 min	Not be less than 30 minutes
3.	Final Setting time	240 min	Not be greater then 600 minutes
4.	Fineness	4.8 %	<10
5.	Specific gravity	3.09	-
Compressive strength:- Cement : Sand (1:3)			
1.	3 days	24.5 MPa	27 MPa
2	7 days	38 MPa	41 MPa
3.	28 days	45 MPa	43 MPa

Table 5.2: Physical Properties of Fine Aggregates

Sr. No.	**Characteristics**	**Value**
1.	Specific gravity	2.46
2.	Bulk density	1.4
3.	Fineness modulus	2.56
4.	Water absorption	0.85
5.	Grading Zone (Based on percentage passing 0.60mm)	Zone III

Table 5.3 Sieve Analysis of Fine Aggregate

Sr. No.	Sieve Size	Mass retained	Percentage Retained	Cumulative Percentage Retained	Percent Passing
1	4.75mm	4.0	0.4	0.4	99.6
2	2.36 mm	75.0	7.50	7.90	92.1
3	1.18 mm	178.0	17.8	25.70	74.3
4	600µm	220.0	22.0	47.70	52.3
5	300µm	274.0	27.4	75.10	24.9
6	150µm	246.5	24.65	99.75	0.25
7	2.50	0.25	0.25	∑=256.55	

Total weight taken = 1000 gm

Fineness Modulus of fine aggregates = 2.56

5.3.3 Coarse Aggregates

Crushed stone aggregate (locally available) of nominal size 10 mm are used throughout the experimental study. The aggregates are washed to remove dust and dirt and are dried to surface dry condition. The aggregates are tested as per IS: 383-1970. The results of various tests conducted on coarse aggregate are given in **Table 5.4 and Table 5.5** shows the sieve analysis results.

Table 5.4 Physical Properties of Coarse Aggregates

Sr. No	Characteristics	Value
1	Type	Crushed
2	Specific Gravity	2.66
3	Total Water Absorption	0.56
4	Fineness Modulus	6.83

Table 5.5 Sieve Analysis of Coarse Aggregates

Sr. No.	Sieve Size	Mass Retained (gm)	Percentage Retained	Cummulative Percentage Retained	Percent Passing
1	20 mm	0	0	0	100
2	10 mm	2516	83.89	83.87	16.13
3	4.75 mm	474	15.8	99.67	0.33
4	PAN	10	0.33	∑= 183.54	

Total weight taken = 3Kg

FM of 10 mm Coarse aggregate = (183.54+500) /100 = 6.83

5.3.4 Water

Fresh and clean tap water is used for casting the specimens in the present study. The water is relatively free from organic matter, silt, oil, sugar, chloride and acidic material as per Indian standard.

5.3.5 Steel Reinforcement

TMT steel bars of 16mm, 20mm and 25 mm diameters and 550 mm length are used as longitudinal reinforcement. The central 300 mm length of rebar is embedded in concrete and 250 mm is exposed on one side in order to make electrical connections.

5.4 DESIGN OF CONCRETE MIX

Concrete mix is prepared using 43 grade Portland pozzolana cement, fine aggregate (medium-sized natural river sand) and crushed stone coarse aggregate with nominal size of 10 mm. The mix is designed as per Indian Standard Guidelines. Different water ratios are used 0.51, 0.48, 0.45, 0.42, 0.39. The mixes are designed for constant water content and the mix proportions obtained for various w/c ratios are summarized in table 5.6.

Table 5.6 Various mix proportions

S.No.	W/C Ratio	Diameter of the Rebar	Water Kg/m3	Cement Kg/m3	Fine Aggregates Kg/m^3	Coarse Aggregates Kg/m^3	Mix Proportion
1	0.51	16mm	197	386	551	1176	1:1.95:8.76
2		20mm	197	386	551	1176	1:1.95:8.76
3		25mm	197	386	551	1176	1:1.95:8.76
4	0.48	16mm	197	410	532	1175	1:2.8:8.66
5		20mm	197	410	532	1175	1:2.8:8.66
6		25mm	197	410	532	1175	1:2.8:8.66
7	0.45	16mm	197	437	516	1167	1:2.2:8.54
8		20mm	197	437	516	1167	1:2.2:8.54
9		25mm	197	437	516	1167	1:2.2:8.54
10	0.42	16mm	197	469	500	1162	1:2.38:8.43
11		20mm	197	469	500	1162	1:2.38:8.43
12		25mm	197	469	500	1162	1:2.38:8.43
13	0.39	16mm	197	505	480	1149	1:2.56:8.26
14		20mm	197	505	480	1149	1:2.56:8.26
15		25mm	197	505	480	1149	1:2.56:8.26

5.5 TEST PROCEDURE

5.5.1 General

Corrosion initiation takes place when the chloride concentration at the rebar level reaches a critical value. The most established of the electrochemical techniques to assess initiation of corrosion activity is half-cell potential mapping. The method of measurement is based on a simple technique and well established equivalence is available for converting the potential obtained from one reference electrode to another. Thus half-cell potential can serve as a determining parameter for indicating initiation of corrosion. In the present study, the half cell potential measurements are carried out with reference to saturated calomel electrode (SCE). Along with half cell potential, linear polarization resistance test is conducted on all slab specimens by monitoring the specimens daily.

5.5.2 Preparation and Preconditioning of Steel Bars

Steel bars are cut to the required length of 550 mm. Each bar is then wire brushed to remove any surface scale. These are then cleaned by soaking in analytical reagent grade hexane and allowed to air dry. This steel specimen preparation is similar as specified in ASTM G 109.

5.5.3 Preparation of Slab Specimen

In the present program, a special moulding system is fabricated for casting the specimens. The slabs are cast in mould of plan size 300 x 300 mm^2 with varying depth such that the clear cover is constant. First of all the interior of slab mould is oiled, so that the slabs can be easily removed from the mould after 24 hours. While embedding these bars in concrete, they are kept in such a way that 250 mm lengths of these bars is protruded outside of the concrete specimen from one side. When the bars have been placed in position, concrete mix is poured and vibrations are given so that the mix gets compacted. The vibration is done until the mould is completely filled and there is no gap left. The slabs are then removed from the mould after 24 hours. After demoulding the slabs are cured for 28 days using jute bags. The concrete surface of the slabs is then cleaned and all dirt and loose materials are removed before initiation of corrosion work.

5.6 CORROSION MONITORING TECHNIQUES

Corrosion of steel embedded in concrete is not visually evident until the damage reaches to the external signs of deterioration as rust spots, cracks or spalling. In order to predict the corrosion service life of reinforced concrete structures and to determine the need of repair or rehabilitation, it is necessary to use non-destructive techniques for assessing the corrosion activity and measuring the corrosion rate of the reinforcements. In the present study, the corrosion rate of rebar is monitored by electrochemical methods.

Electrochemical Techniques

The electrochemical measurements are carried out using a versatile instrument (Make ACM, model: serial no.1463 field machine) that is capable of performing various electrochemical tests such as potential measurement, AC impedance technique, potentiostatic cyclic sweep test, LPR measurements etc. **Fig 5.2** shows ACM setup used for electrochemical monitoring**.** The instrument is capable of processing the data and plotting the outputs automatically.

The half cell potential measurement gives only an indication of the corrosion risk of the steel and is linked by empirical comparisons to the probability of corrosion. Therefore along with half cell, linear polarisation (corrosion rate) measurements are taken which provides a valuable insight into the instantaneous corrosion rate of the steel reinforcement, giving more detailed information than a simple potential survey. The LPR data enables a more detailed assessment of the structural condition and is a major tool in deciding upon the optimum remedial strategy to be adopted. Hence the two important electrochemical techniques that are used for the studying the corrosion activity are corrosion potential (E_{corr}) and corrosion current /current density (I_{corr}). These determining parameters indicate the corrosion initiation.

The following section describes the procedure in details:

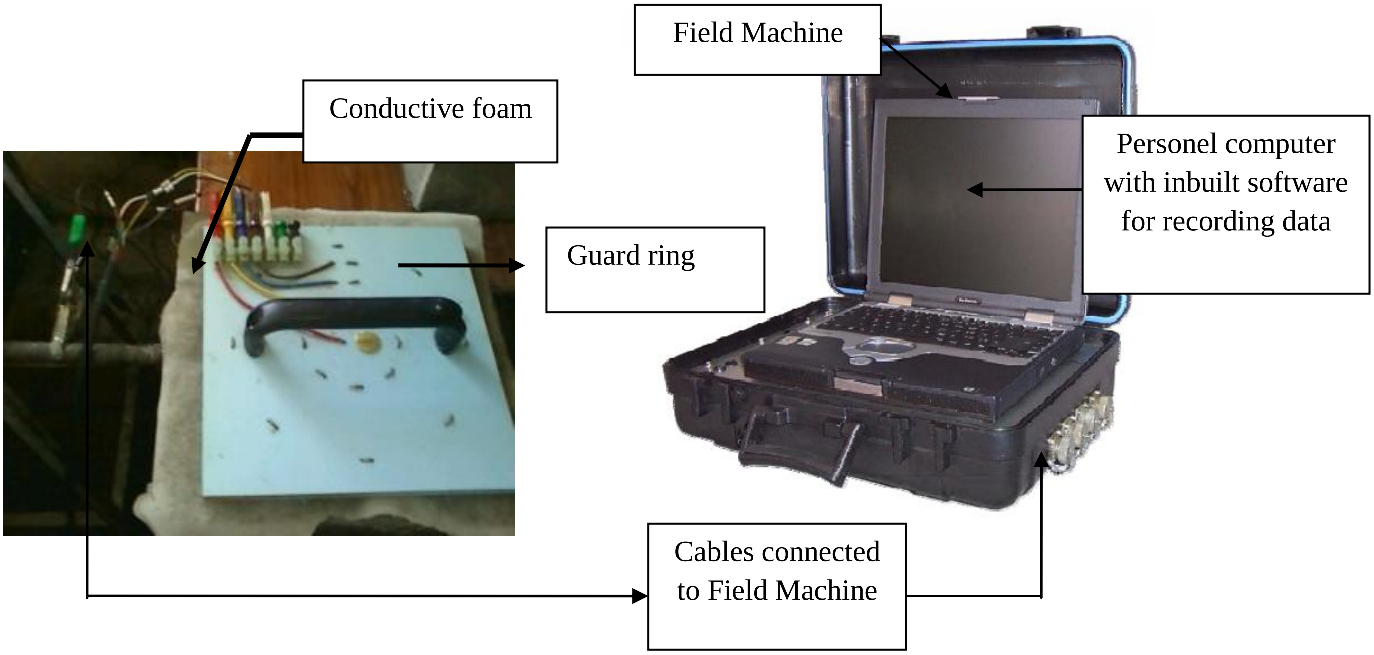


Fig 5.2: ACM Setup Used for Electrochemical Monitoring

5.6.1 Half cell potential measurements

In the present study, all the specimens are monitored daily by half-cell potential using a saturated calomel reference electrode by placing the electrode on top surface of the concrete. The procedure followed is ASTM Standard C 876. The power supply is switched off one hour before taking the half cell readings in order to completely depolarize it. To maintain a consistent testing environment over the experimental test period, the dripping salt water is replaced daily, electrodes are cleaned daily and the wiring and electrical connections checked twice a day. If the corrosion potential reading is more positive than -200mV, probability is that no reinforcing steel corrosion is occurring in the area at the time of measurement and if the potential reading is more negative than -426mV, probability is that the reinforcing steel corrosion is occurring [16]. The ASTM interpretation of half-cell potential (SCE) is summarized inn **Table 5.6**. The experimental arrangement for half cell measurement is shown in **Fig. 5.3**.

Table 5.7 The ASTM Interpretation of Half-Cell Potential Readings

Open circuit potential (OCP) values	Corrosion condition
< -426 mV	Severe corrosion, corrosion induced cracking may occur
< -276 mV	High risk, 90% probability of corrosion
-126 to -275 mV	Intermediate risk, corrosion activity in uncertain
0 to -125 mV	Low risk, 10% probability of corrosion

Fig 5.3 Half Cell Arrangement

5.6.2 Linear polarization resistance (LPR) measurements

Electrochemical LPR technique is especially good at measuring the localized corrosion. LPR measurements on concrete surfaces are performed using guard ring that is supplied with the field machine for precise location of rebar areas. The Guard Ring simply connects to the front panel via the supplied cables. Incorporated into the Guard Ring are a $Cu/CuSO_4$ reference electrode. Before performing the test, conducting sponge is wetted with NaCl solution and placed on the surface of the slab specimen to have proper electrical contact with the guard ring. Guard ring assembly is then placed above the wetted sponge. The electrical connections are made to the steel rebar.

For linear polarization resistance measurement, the working electrode i.e. the steel rebar is polarized to ± 20mV from the equilibrium potential at a scan rate of 0.1mV per second. The experimental arrangement for LPR measurement with guard ring arrangement is shown in **Fig. 5.5**. The polarized surface area of the steel rebar is taken to be that lying under a circle intersecting the midpoint between the two sensor electrodes and only the top half surface area of the steel reinforcement is assumed to be polarized.

Fig. 5.4 Guard Ring Arrangement

For calculation of the corrosion current density I_{corr}, Stern-Geary equation is used;

$$I_{corr} = \frac{B}{R_p}$$

Where B is the Stern-Geary constant and is given by B = ($\beta_a \times \beta_c$) /2.3($\beta_a + \beta_c$). β_a and β_c are anodic and cathodic Tafel constants respectively. The value of B is taken as 26mV considering steel in active condition. Rp is the polarization resistance.

5.7 Inducing corrosion in steel rebar

The objective of inducing corrosion to the reinforcing bar is to simulate the corrosion damaged concrete. The commonly used methods of inducing corrosion in RC specimens can be recalled as salt spray (Berver et al. 2001; Debaiky et al. 2002; Gadve et al. 2010; Batis and Rakanta 2005), Chloride diffusion (Tamer et al. 2005; Masoud and Soudki 2006; Tamer et al. 2006), alternate drying and wetting in salt water (Debaiky et al. 2002 and Soudki 2006) and impressing anodic current (Lee et al. 2000; Tamer et al. 2003; Wootton et al. 2003). Previous studies have shown that test specimens kept in a salt spray chamber for more than 100 days did not show any visible signs of corrosion. This method was not found suitable considering the time constraint. Method of adding chlorides artificially to the concrete during casting is an effective method of initiating corrosion in an steel rebar. This method was not considered because it did not simulate the present condition of interest. Alternate immersion into NaCl (Sodium Chloride) solution and drying of the specimens also induces corrosion. However, the quickest method of inducing corrosion is by impressing anodic current. In this method, NaCl solution is supplied to the specimens and a direct current is passed making the reinforcement bar as an anode and another metal nobler than steel in electro-chemical series as cathode. Incidentally, this method has been used by a number of previous investigators (Bonacci 2000; Craig 2002; Masoud et al. 2001; Bhavneet et al. 2010; Gadve et al. 2010).

Present method of inducing accelerated corrosion

In this investigation, the specimens are kept fully saturated by continuously dripping with 5% NaCl solution as shown in Fig 5.4. Mats are placed over the tops to provide even distribution of NaCl solution. The rebar is used as anode. A stainless steel (SS) mesh is rolled around 300 mm length of specimen and tied together with metal ties in order to assure electrical continuity and is used as cathode (Fig. 5.5). The reinforcement extended 250 mm on one side of the concrete to allow easy access for making electrical connections to the steel. The constant voltage of 5V is impressed in order to accelerate corrosion. The DC regulated power supplier (DCRPS) used in the present study could supply 1000mA DC at 30V. The rebar is connected to the positive terminal of the external DC source and negative terminal is connected to the SS mesh. It is more common to maintain a constant voltage between the cathode and the anode (Soudki 2006; Almusallam et al. 1996; Bonacci et al. 1998; Gadve et al. 2010). Half-Cell potential and linear polarization measurements are obtained daily for all the specimens throughout the duration of experiment.

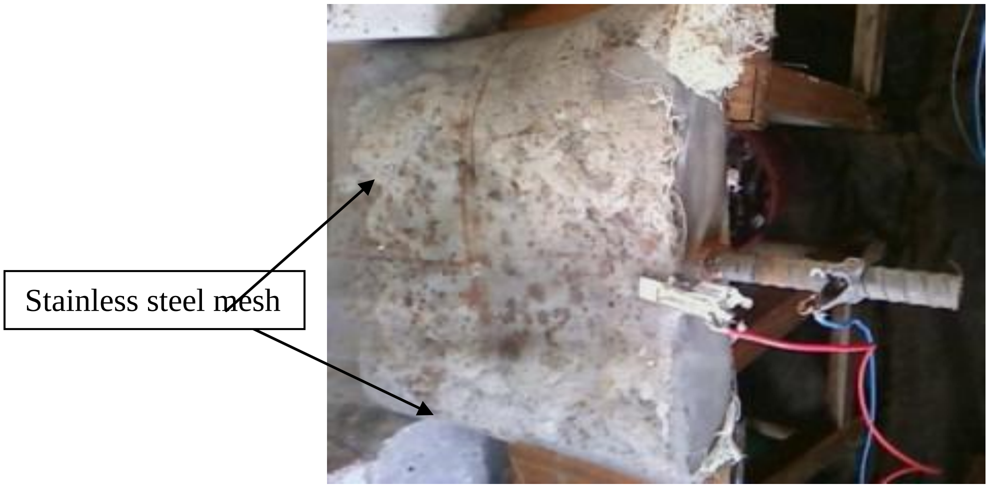


Fig. 5.5 View of Stainless Steel Mesh

5.8 Water permeability test

Deterioration of concrete structures has been the subject of major concern and research for the past few years. The main factors for deterioration of RC concrete structures are corrosion of reinforcement, sulfate attack and salt weathering and cracking due to environmental effects and potential aggregate- cement reactivity. These factors are visible manifestation of excessive salt inclusions in concrete mostly due to ingress of aggressive through cracks and pores. Therefore, these factors are mostly permeability oriented. The permeability of concrete is the most important property determining its long term durability in aggressive environment.

The permeability test gives a measure of the resistance of concrete against the penetration of external agents into cover concrete. In the present study, permeability of various mixes is measured by water impermeability test and RCPT test. The following sections explains the procedure.

5.8.1 Water Impermeability Test

Most deterioration processes have two stages. Initially, aggressive fluids (water, solution etc.) penetrate through the capillary pore structure of the concrete to reaction sites. This penetration is followed by the actual chemical or physical deterioration reactions. To access concrete durability during the first stage, tests that measure transport rates in concrete are required. The transport rate, normally called permeability of concrete, is measured in terms of flow of water through the concrete specimen. However, there is a problem with the permeability test: in good quality concrete, there is no flow of water through concrete even after some days also. It is expected that the water permeability tests might not give any data for comparison of mixes. Therefore, water impermeability test is performed instead of water permeability test. The test is performed as per the German Standard DIN 1048 (Part 5). For performing the test, 150 mm cubical specimens are cast similar to the procedure adopted for casting specimens for strength tests. The specimens are subjected to all the five curing regimes and the test is carried out at the age of 28 days. For conducting the test, the concrete specimens are exposed to a water pressure of 0.5 N/mm^2 acting normal to the mould filling direction, for a period of three days. Three cell apparatus is used for performing the test, i.e. three specimens are subjected to the said pressure at the same time (as shown in Plate 3.9). The pressure is kept constant throughout the test. At the end of three days, the pressure is released and the specimens are split down the center with the face which was exposed to water facing down. After about ten minutes, the maximum depth of penetration in the direction of cube thickness is measured on each of the split halves in mm. The mean of the maximum depth of penetration obtained from the three specimens tested is taken as the test result. In high water heads, a concrete specimen measuring 150x150x150 mm^3 is casted and after 28 days of curing, it is exposed from above to a water pressure of $0.5N/mm^2$ acting to the normal mould filling direction, for a period of three days. This pressure is kept constant throughout the test. Before start of the experiment it is checked that the specimen placed under pressure should not leak. If it leaks, then epoxy is applied around it. Immediately after the pressure has been released, the specimen shall be removed and split down the centre, with the face which is exposed to water facing down. When the split faces shows signs of drying (after about 5 to 10 minutes), the maximum depth of penetration in the direction of slab thickness, shall be measured in mm.

Fig.5.6 Water permeability Apparatus

Fig.5.7 Split Tensile Test of Cube

While performing the experiment, it should be noticed that there be no leakage in the specimen.

5.5.3 Rapid Chloride Permeability Test

This test method was originally developed by the Portland Cement Association, under a research program paid for by the Federal Highway Administration (FHWA). The original test method may be found in FHWA/RD-81/119, “Rapid Determination of the Chloride Permeability of Concrete.”

Since the test method was developed, it has been modified and adapted by various agencies and standard's organizations. These include:

• AASHTO T277, "Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete"

• ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration"

As the chlorides penetrate deeper into the concrete, the pore solution becomes more conductive and the current readings increase. In concretes with high conductivity values, however, these effects are small relative to changes in conductivity due to temperature rise.

In RCPT test, core of 50mm diameter and 100mm long sample is casted. After curing it for 28 days, a 50 mm (2 in.) specimen is cut from the sample using diamond-dressed coring bit. The cut sample should be clean on all sides.

Fig.5.8 Core Cutter Machine

Fig. 5.9 50mm long and 100mm diameter after cutting with the machine

After the curved surface of a test specimen is coated with epoxy, the specimen is vacuum saturated with water Rapid Chloride Permeability Tests and then soaked for 18 hours. After 18 hours, the specimen, is taken out of the vacuum and is then allowed to dry. The specimen is then placed in the testing apparatus where one end of the specimen is exposed to a solution containing sodium chloride (NaCl) and the other end is exposed to a solution containing sodium hydroxide (NaOH). To increase the rate of chloride penetration into the specimen, thus speeding up the test, a constant 60 V potential is applied across the specimen. The current across the specimen is measured at least every 30 minutes during the 6-hour test. At the end of 6 hours the sample is removed from the cell and the amount of coulombs passed through the specimen is calculated. The test results are then compared to the values given in the table 5.7.

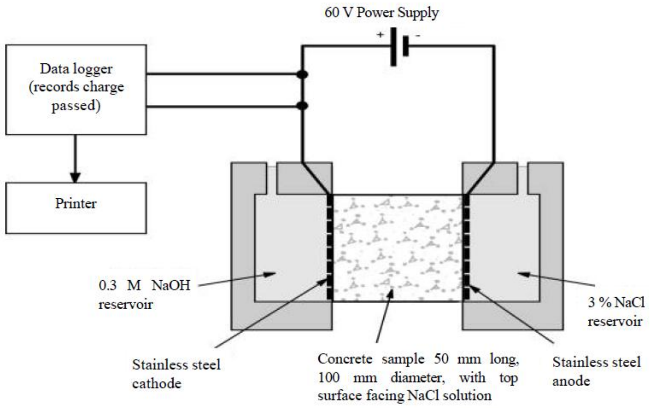


Fig. 5.10 AASHTO T277 (ASTM C1202) test setup.(KD Stanish)

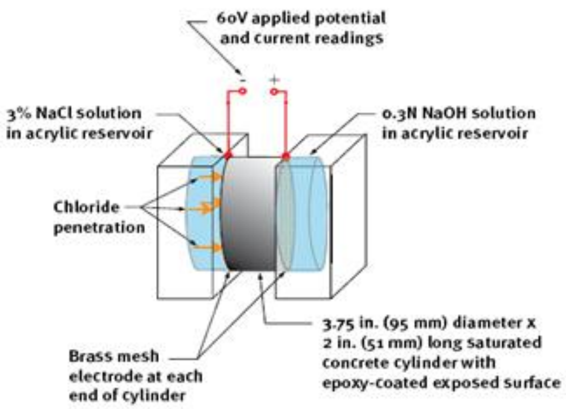


Fig. 5.12 Schematic of Rapid Chloride Permeabiity Test Setup

Table.5.8 Chloride ion Permeability based on charged passed

Charged passed, coloumbs	Chloride ion penetrability
>4000	High
2000 to 4000	Moderate
1000 to 2000	Low
100 to 1000	Very low
<100	Negligible

Fig. 5.12 Vaccum Pressure Equipment

5.9 Pullout Test

After corrosion of the slab, pullout tests were carried out on all the specimens. This was done by securing the slab in a universal testing machine (UTM). One end of the bar is gripped from dwn and on the other end, load is applied and then pullout strength is noted.

Fig. 5.13 Pull out setup and Cracking in the Slab

Fig.5.14 Corrosion of the rebar

CHAPTER 6 RESULTS AND DISCUSSION

6.1 INTRODUCTION

ASTM C876 suggests that the corrosion possibility of rebar embedded in concrete is higher than 90% when the open circuit potential is lower than -270 mV (SCE). In addition, the corrosion possibility is lower than 10% when the open circuit potential is higher than -120 mV (SCE).

6.2 EFFECT OF WATER CEMENT

6.2.1 On Half – Cell Potential

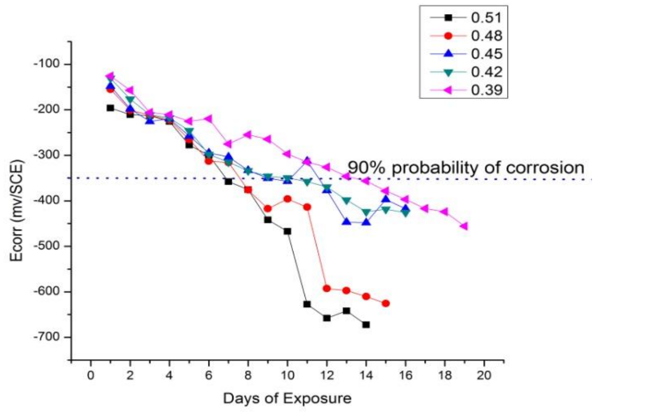
Half -cell potentials is measured on the reinforcing steel bars embedded in the concrete with different water cement ratios. For slab specimens with plan size of 300x300 mm^2 with 20mm cover it is observed that drop in half cell potential value occurred at different periods depending upon the water cement ratio.

From the experimental results, it is observed that the half cell measurements keep on increasing with time but within a certain defined range. In the first few days, the potential is less than -300 mV, which indicates that corrosion has not get initiated and further the value falls and becomes more negative and reaches the range of -300mV to -500mV, which indicates that there is possibility of corrosion initiation. Later the value even fall down to -700mV in certain cases.

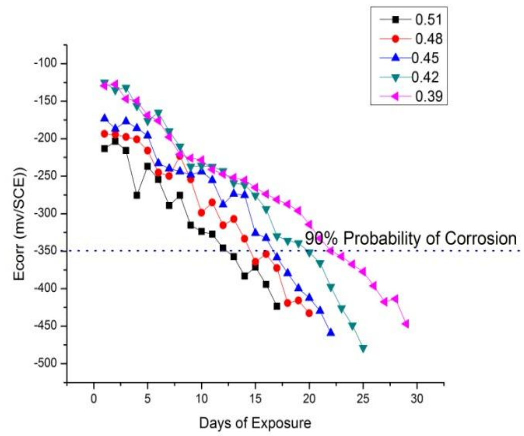
When 5V of voltage supply on the samples is applied having different water cement ratios 0.51,0.48, 0.45, 0.42 and 0.39, it is observed that the time taken by the lower water cement ratio to reach the threshold value will be more. Time taken by the chlorides to reach the threshold value increases with decrease in the water cement ratio.

It is observed from the figure 6.1 that irrespective of bar diameter, the time required to reach the threshold corrosion potential increase with the decrease in water cement ratio. It is because of the decrease in permeability with decrease in water cement ratio.

16mm



20mm



25mm

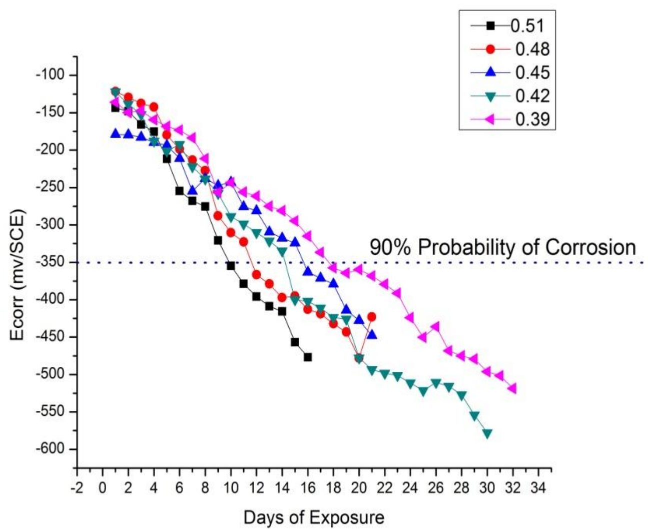


Fig. 6.1 Half cell Potential with different water cement ratio

Hoseini et al(2008) observed the effect of different water cement ratio on the reinforced structures through an experiment in which 120 concrete prism specimens measuring 15×15×60cm were exposed to the marine environment of Bandar Abbas city in the south of Iran of four water to cement ratios (0.35, 0.40, 0.45, and 0.50). They concluded that the service life of the structures with minimum water cement ratio along with superplastizer can be increased to 6 years to 10 years more.

Similar observation was carried by stewart and Vu(2002). They observed that increasing w/c ratio resulted in increased crack propagation rates by up to 30% and 40% for 25mm and 50mm cover respectively. It was also observed for concrete cover, the w/c ratio appears to mostly influence crack propagation when the crack width exceeds 0.15mm to 0.3mm.

6.2.2 Linear Polarization Resistance

Corrosion initiation described as the process by which chloride ions permeate into the cover concrete and accumulate around the reinforcing steel, thereby breaking down passivity of the reinforcing steel and allowing corrosion to initiate. Corrosion propagation is described as the process by which the rate of corrosion is accelerated by the electric current at a constant voltage. Half cell potential is effective only in monitoring the corrosion initiation. Therefore, LPR measurements are necessary as the values obtained from corrosion current density indicates the progression of corrosion in the propagation phase.

From the determined corrosion current density values by LPR method, it is observed that the corrosion current density decreases with decrease in water cement ratio. As the corrosion progresses in the slabs the value of corrosion current (I_{corr}) rises. This is because of the depassivation of layer formed around the rebar due to the concentration of chlorides.

When 5V of voltage supply on the samples was applied having different water cement ratios 0.51,0.48, 0.45, 0.42 and 0.39, it is observed that the time taken by the lower water cement ratio to reach the threshold value is more. Time taken by the chlorides to reach the threshold value increases with decrease in the water cement ratio.

From the graphs given in fig. 6.2, it is clear that as the water cement ratio increases, permeability keeps on increasing. Since the cover depth provided in the slabs are same but still the time taken by the chlorides to reach the threshold value depends on water cement ratio only. With decrease in water cement ratio, the permeability decreases. Hence, more time will be required by the chlorides to reach threshold value on rebar surface.

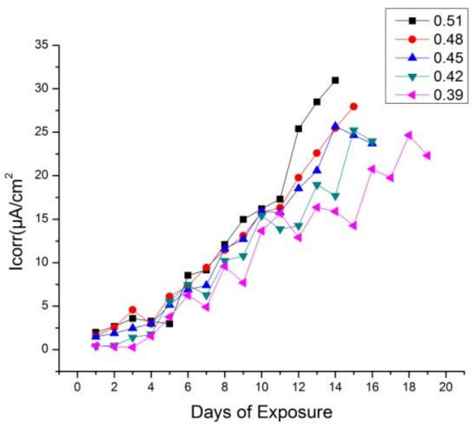
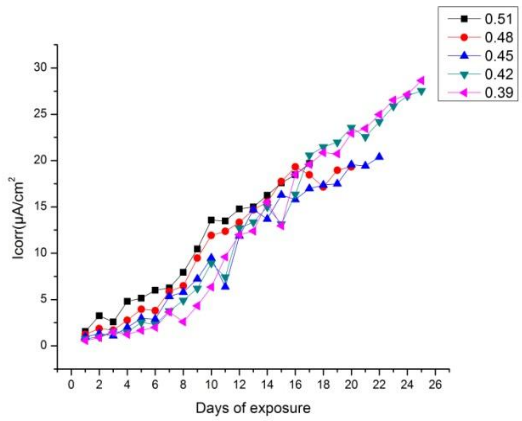
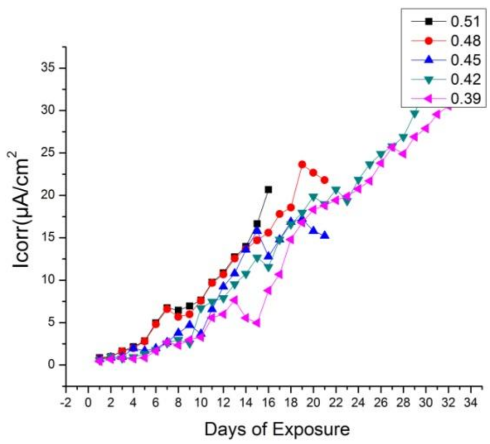
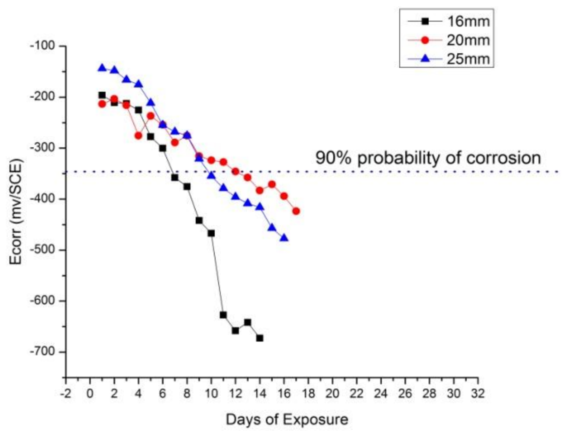
16mm Dia Bar	20mm Dia Bar
	
25mm Dia Bar 	

Fig.6.2 Linear Polarization Resistance with different water cement ratio

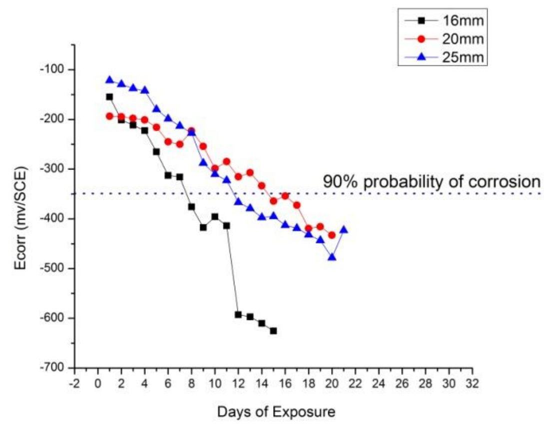
Stewart and Vu (2002) on the basis of the measured values of corrosion current density have given criteria for corrosion activity. Corrosion current density values less than 0.1µA/cm^2 correspond to passive condition and corrosion current density values ranging between 0.1µA/cm^2 and 1µA/cm^2 correspond to moderate corrosion rate. Similarly corrosion current density values ranging between 1µA/cm^2 and 10µA/cm^2 and between 10µA/cm^2 and 100µA/cm^2 correspond to high and very high corrosion rate respectively.

6.3 EFFECT OF DIAMETER ON HALF CELL POTENTIAL AND LINEAR POLARIZATION RESISTANCE

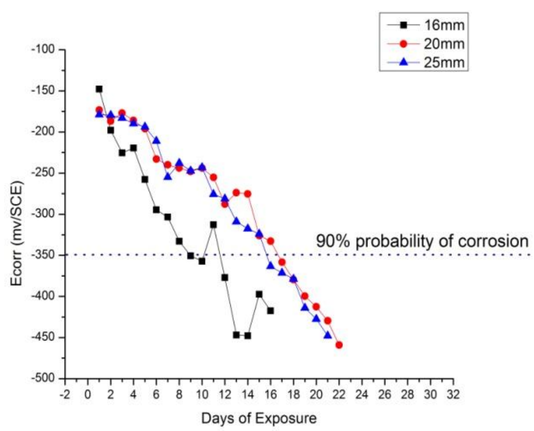
0.51 water cement ratio with different dia



0.48 water cement ratio with different dia

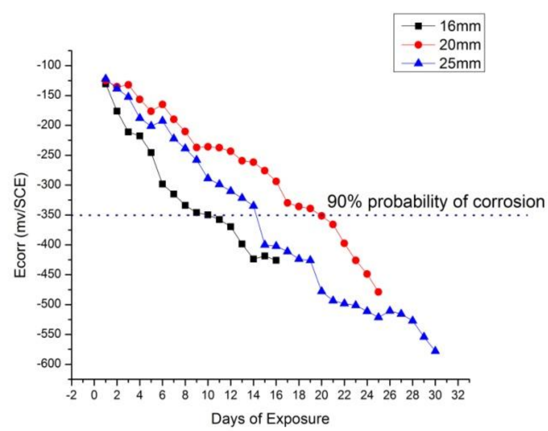


0.45 water cement ratio with different



dia

0.42 wayer cement ratio with different dia



0.39 water cement ratio with different dia

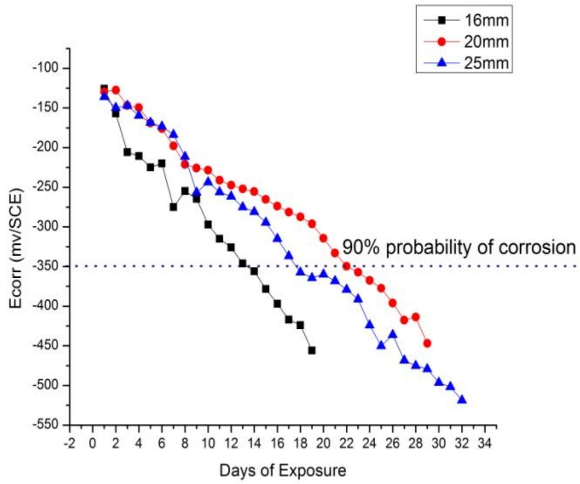
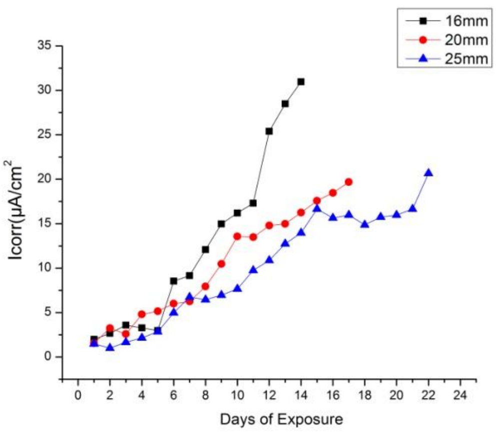
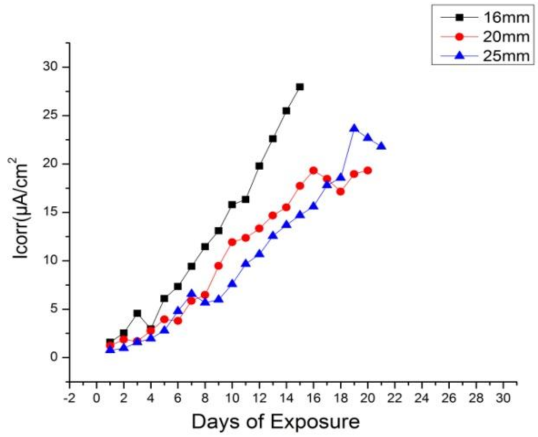
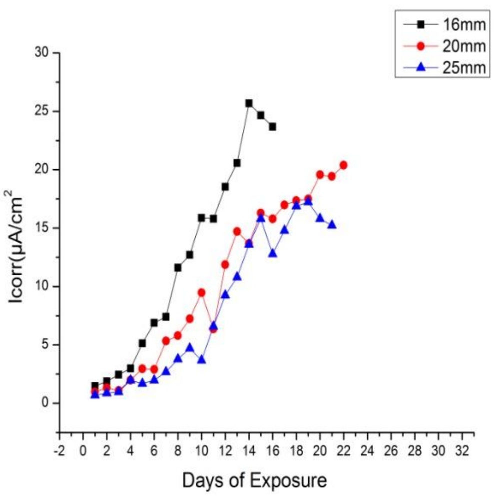
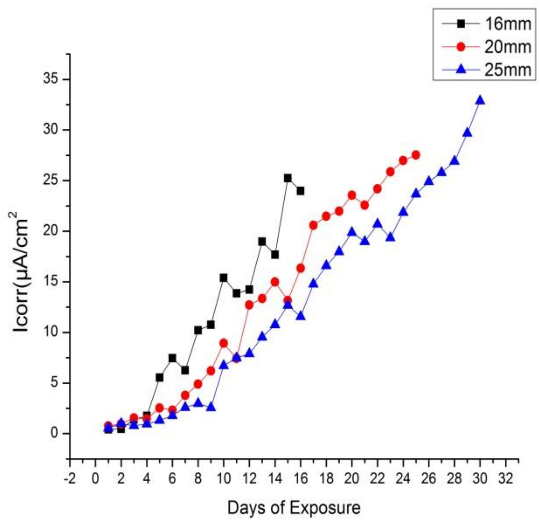


Fig.6.3 Half cell Potential of various Reinforcement Steel

Half cell potential monitors the corrosion initiation. As the diameter of the rebar increases, time taken by the rebar to reach corrosion initiation level at potential of -33mV that indicate 90% probability of corrosion. Time taken by the 16mm diameter bar to reach the corrosion initiation level is less as compared to 20mm diameter and 25mm diameter bar.

From the graphs given in fig. 6.3 and 6.4 of half cell and LPR, it can be concluded that effect of diameter is less as compared to water cement ratio. The values of half cell and i_{corr} of 20mm and 25mm diameter are relatively close to each other.

0.51 water cement ratio with different diameter 	0.48 water cement ratio with different diameter 
0.45 water cement ratio with different diameter 	0.42 water cement ratio with different diameter 

0.39 water cement ratio with different diameter

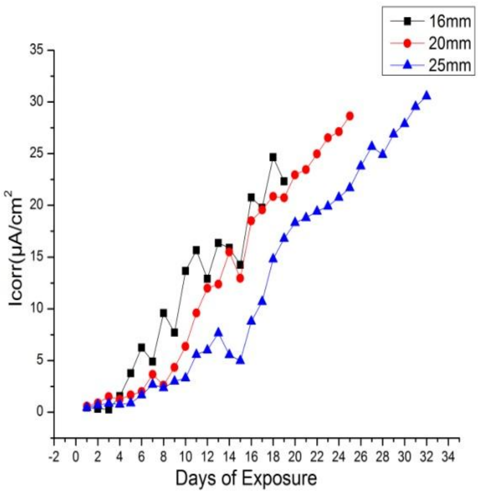


Fig.6.4 Linear Polarization Resistance of various Reinforcement Steel

6.4 EFFECT OF PERMEABILITY ON WATER CEMENT RATIO

Permeability of concrete is the most important property determining its long term durability in aggressive environment. Five different water cement ratios were taken of 0.51, 0.48, 0.45, 0.42 and 0.39 and its effect on permeability is monitored by two indirect permeability test methods: water impermeability test as per DIN 1048 (Part 5) and rapid chloride test as per ASTM C 1202. These tests are called indirect tests because these methods do not measure coefficient of permeability of concrete. The objective of comparing two methods is to judge most suitable method out of two.

From the graphs given in figure 6.5 and 6.6, it is clear that both the test shows the similar results, i.e., with decrease in water-cement ratio, the permeability of concrete decreases. Though the inference drawn from two test methods is same but it can be said that water impermeability test is more realistic and simple to perform. It is because this test gives the depth of water front in concrete after exposure to water under small pressure. On the other hand, RCPT test is lengthy and require skilled person to perform it accurately. Secondly, the wire mesh used in the RCPT test can cause error if not properly used. Moreover, this test method does not replicate actual conditions that concrete would experience in the field. In actual practice, concrete is never exposed to potential as high as 60-volt.

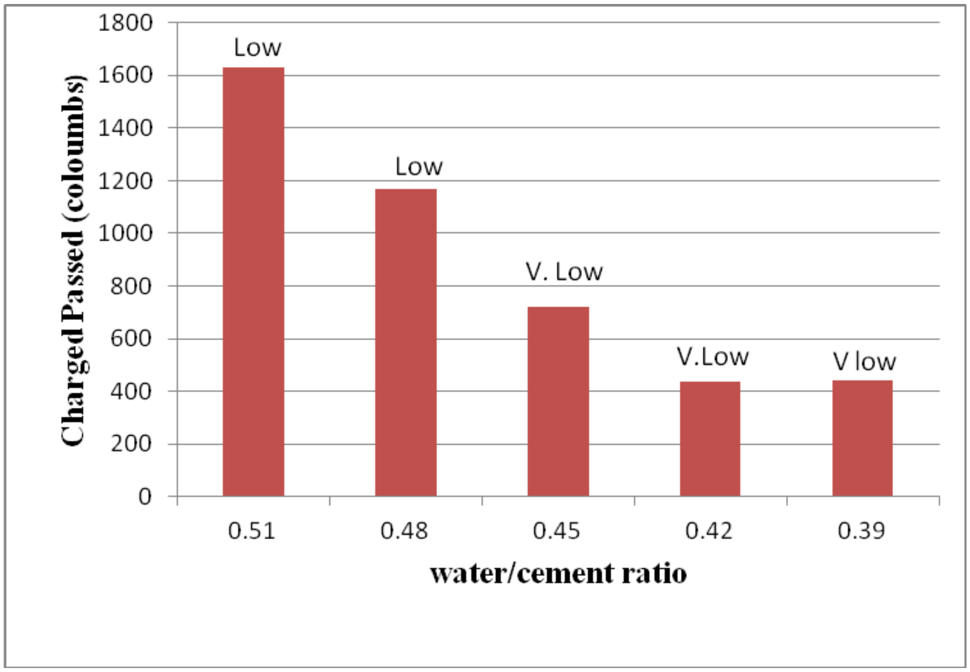


Fig.6.5 Variation of Water Cement Ratio on RCPT Test

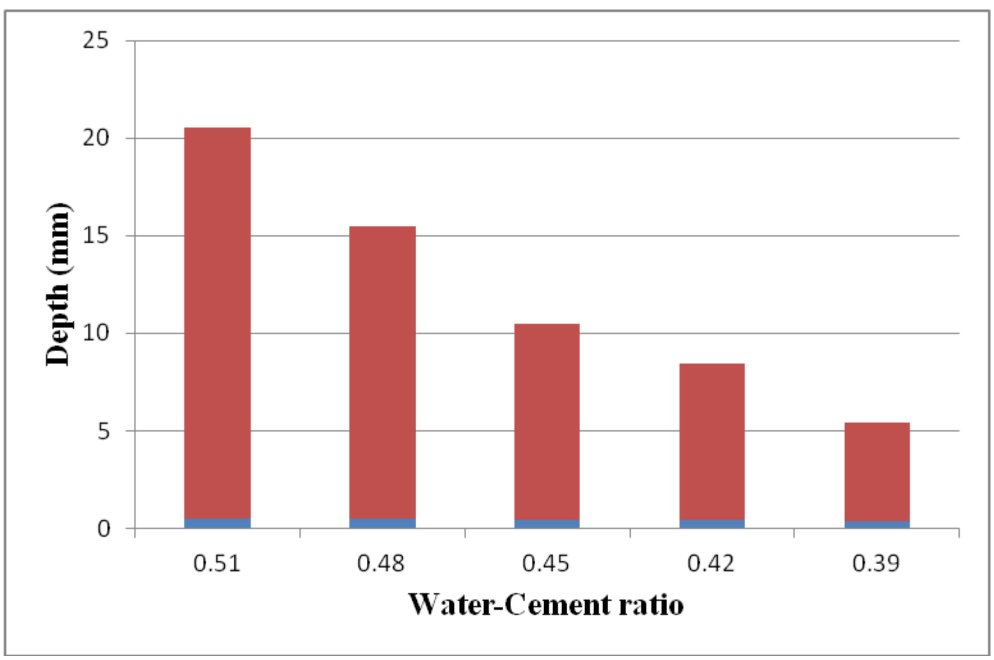


Fig.6.6 Variation of Water Cement Ratio on Water Permeability Test

6.5 Pull out Test

Pull out test was carried out on all the specimens after the corrosion in the slabs has taken place. It was carried out on universal testing machine (UTM). In this test, the bar was pulled out of the specimen. After that the corrosion was noticed on the rebar.

From the graphs shown in Fig 6.7, it is clear that the pullout strength of 0.39 water cement ratio is more as compared to other water cement ratios. when the load was applied on the slab, bond between the concrete and the steel weakens and the crack appears on the slab and finally the slab breaks.

According to Azad et al (2010), when reinforced concrete beams with different cross-section and different diameter were subjected to accelerated corrosion and after then beams were tested under four point load then it was noticed that the percentage-wise loss of metal is smaller for a large diameter bar compared with that for smaller diameter bar at a constant I_{corr}. Same observation was noticed when slabs of different water cement ratio with different diameter were first subjected to accelerated corrosion and then pullout test is done. It is observed that the pullout strength varies with water cement ratio and also with diameter of rebar.

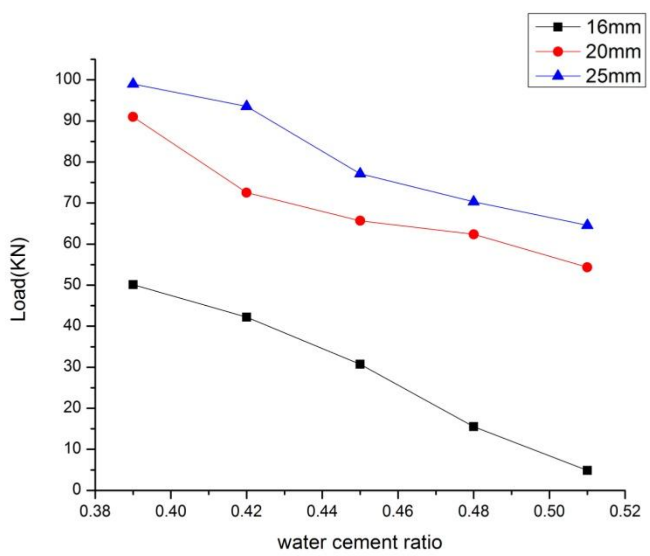


Fig.6.7 Effect of water cement ratio on Pullout Strength

CHAPTER 7 SERVICE LIFE PREDICTION USING CORROSION CRACKING THEORY

7.1 GENERAL

Chloride-induced steel corrosion is one of the major worldwide deterioration mechanisms for steel reinforced concrete structures and its repair cost is high. Therefore, through the use of deterioration models, cost-effective decisions concerning the time to repair, rehabilitate, or replace existing structures can be made along with predicting future maintenance and replacement needs as well as the selection of the most effective corrosion systems [liu and weyers ,1998). The time required for corrosion initiation corresponds to the time required for chloride ions to diffuse to the steel-to-concrete interface and activate corrosion. Extensive research work has been devoted to develop models that predicts the time for corrosion initiation. Few other researchers have attempted to develop simple mathematical models for prediction of time from corrosion initiation to corrosion cracking. However, the most effective model will be the one that take into consideration both initiation and propagation states. Corrosion cracking theory tries to incorporate both stages in the model. Hence, it is utilized to develop service life prediction model. The following sections describe corrosion cracking theory.

7.2 TUTTI’S MODEL AND ITS LIMITATION

As proposed by Tutti’s model, service life is divided into two phases, the time-to-initiate corrosion and the time to cracking after corrosion initiation. The initiation stage is the phase during which chloride ions penetrate the concrete cover and reach the reinforcing steel in sufficient quantities to depassivate it, therefore initiating the process of corrosion(fig.7.2). The propagation time is the time from corrosion initiation until a specified level of corrosion-induced damage state is attained. But this model underestimates the time to corrosion cracking compared with times obtained from field and laboratory observations. Weyers(1993) reported that not all corrosion products contribute to the expansive pressure on the concrete; some of them fill the voids and pores around the steel reinforcing bar and some migrates away from the steel-to-concrete interface through concrete pores.

7.3 MODIFIED TUTTI'S MODEL

According to weyers, in modified service life model, the propagation period is divided into two different periods. The first one is the free expansion period, T_{free}, corresponding to the time required for corrosion products to fill the porous zone around the corroding steel reinforcing bar. When the total amount of corrosion products W_T is less than the amount of corrosion products required to fill the porous zone around the steel/concrete interface W_P. The formation of corrosion products at this stage will not create any stress on the surrounding concrete. The second period encompasses the time in which the stress builds-up, T_{stress}, as corrosion products, having filled the porous zone, exert an expansive pressure on the surrounding concrete. As the total amount of corrosion products W_T exceeds the amount of corrosion products needed to fill the porous zone around the steel/concrete interface W_P , the formation of corrosion products starts to create expansive pressure on the surrounding concrete, and this pressure increases with an increase in corrosion products. Third period is the cracking stage in which the total amount of corrosion products W_T reaches the critical amount of corrosion products *Wcrit* (the limiting amount of the corrosion products needed to induce cracking of the cover concrete), the internal stress from the volume increase of rust products will exceed the tensile strength of concrete and crack the cover concrete

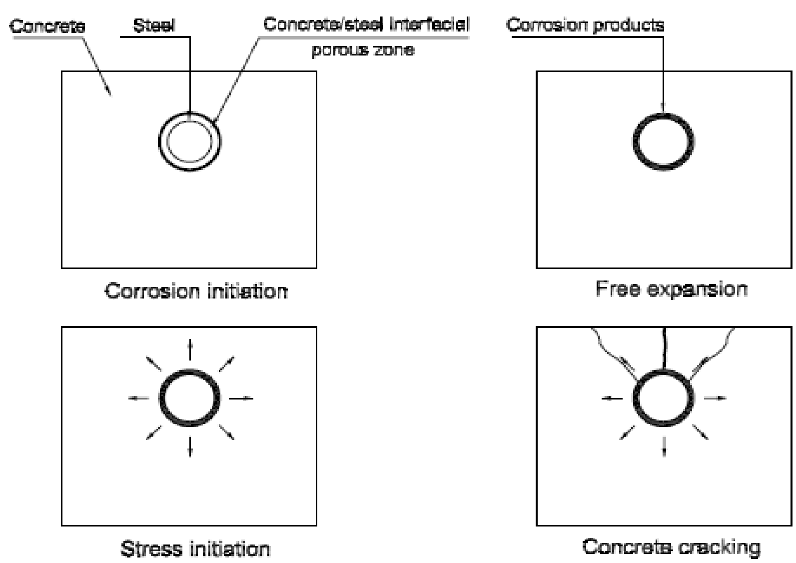
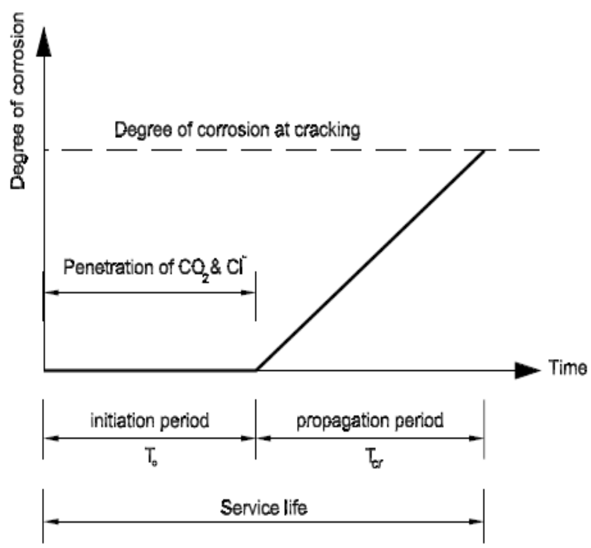


Fig.7.1 Schematic diagram of corrosion cracking processes (liu and weyers, 1998)

7.4 MATHEMATICAL REPRESENTATION FOR MODIFIED MODEL

Bazant model assumes that the pressure increases linearly as the volume of corrosion products increases until the internal tensile stresses exceed the tensile strength of concrete at which time cracking of concrete cover occurs. This assumption is valid up to the point of cracking, after which time pressure distribution is not uniform. Cover cracking marks the end of functional service life of a corroded structure where structural rehabilitation is needed.

Service life Prediction model(Tutti's Model)	Modified service life model (weyer's model)



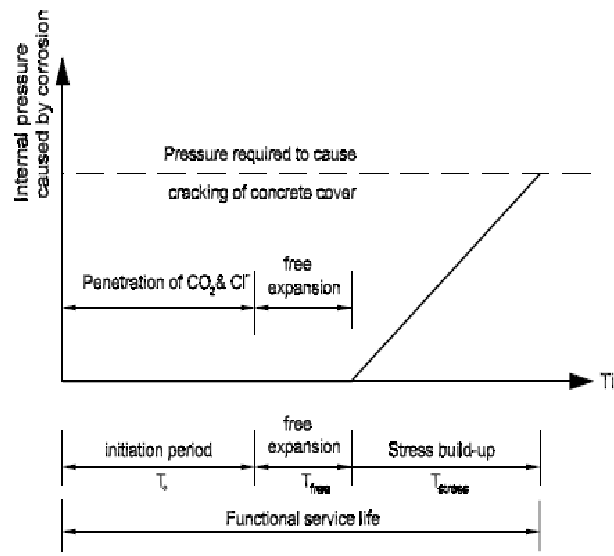


Fig. 7.2 Service life prediction model

Bazant suggested a mathematical model to calculate the time between corrosion initiation and corrosion cracking of RC bridge decks. According to Bazant's model, the time from corrosion initiation to corrosion cracking is mainly dependent on corrosion rate, cover depth, spacing between steel reinforcing bars, diameter of the steel reinforcing bar, and properties of concrete such as tensile strength, modulus of elasticity, Poisson's ratio, and creep coefficient. Bazant's model assumes that all corrosion products create pressure on the surrounding concrete which would underestimate the time to corrosion cracking.

However, it can be seen from liu and weyers model that till the time the corrosion products fill porous zone, no pressure will be created. Therefore, the model should incorporate this part separately to predict service life. The various steps involved in this method are:

1. Estimation of W_P

The amount of the corrosion products needed to fill the porous zone around the steel/concrete interface is difficult to measure and therefore it is considered that the steel/concrete interface is somewhat like the transition zone between cement paste and aggregate and influenced by the *w/c*, aggregate sizes, and steel reinforcement, the value of of W_P may be expressed as

$$W_P = \rho_{rust} V_P$$

(7.1)

Where,

ρ_{rust} is the density of corrosion products, and

V_P is the total volume of interconnected pores around the steel/concrete interface.

Let d_0 be the thickness of the porous zone around the steel/concrete interface that depends on the total volume of interconnected pores around the steel/concrete interface.

A steel bar having originally diameter D will increase its diameter to $D + 2d_0$ when the amount of corrosion products reaches W_P. For a unit length of steel bar, since $d0 << D$, W_P can be estimated from Eq. 7.2

$$W_P = \pi \rho_{rust} d_0 D \quad (7.2)$$

W_P is related to the size of the reinforcement, density of rust products, and property of steel/concrete interface.

2. Estimation of *Wcrit*

The critical amount of rust products consists of two parts: W_P , the amount of corrosion products required to fill the total interconnected porous around the steel/concrete interface, and W_S , the amount of corrosion products that generate the critical tensile stresses.

As $ds << D$, for a unit length of steel bar, the value of *WS* can be estimated from Eq. (7.3)

$$W_s = \pi\rho_{rust} \left(\pi(D + 2d_0) d_s + W_{st}/\rho_{st}\right) \quad (7.3)$$

Where,

ρ_{rust} is the density of corrosion products;

ρ_{st} is the density of steel;

d_0 is the thickness of pore band around the steel/ concrete interface;

d_s is the thickness of corrosion products needed to generate tensile stresses;

D is the diameter of steel reinforcement; and

W_{st} is the mass of the corroded steel.

Based on Eq. (7.2) and (7.3), the critical amount of corrosion products may be rewritten as follows, after neglecting the term 2 $d_0 d_s$ as being very small

$$W_{crit} = \rho_{rust} \left(\pi (d_s + d_0) D + W_{st}/\rho_{st}\right) \quad (7.4)$$

$W_{st,}$ the amount of steel corroded, equals aW_{crit}, in which α is expressed as the molecular weight of steel divided by the molecular weight of corrosion products. Value α depends on the kind of corrosion products; for example, α equals 0.523 when corrosion products are considered as $Fe(OH)_3$ and 0.622 as $Fe(OH)_2$.

The thickness of corrosion products d_s needed to generate the critical tensile stress that induces cracking of the cover concrete can be routinely solved by the finite element method.

Considering concrete to be a homogeneous elastic material and a thick-wall concrete cylinder with inner radius $a = (D + 2d_0)/2$, and outer radius $b = C + (D + 2d_0)/2$, in which *C* is cover depth, then the pressure *P* at concrete/rust products interface can be expressed as

$$P = \frac{2E_{ef} d_s}{(D + 2d_0)\left(\frac{b^2 + a^2}{b^2 + a^2} + V_c\right)} \quad (7.5)$$

Where,

v_c is Poisson's ratio of the concrete,

E_{ef} is an effective elastic modulus of the concrete where $E_{ef} = E_c/(1 + \varphi_{cr})$,

E_c is elastic modulus of the concrete, and

φ_{cr} is the creep coefficient of the concrete.

The radical displacement of the concrete *ds* is under pressure *P*. For this case, because the elastic modulus of steel is about seven times greater than that of concrete, it is considered that the radical displacement of the concrete under pressure *P* is the thickness of corrosion products generating the pressure on the concrete.

If, at failure, considering that cracking occurs above and parallel to the reinforcement (the observed corrosion cracks in reinforced concrete structures are located mostly above the steel reinforcing bar, as shown in Fig. 4), the minimum stress required to cause cracking of the cover concrete equals the tensile strength of concrete

$$P = \frac{2Cf_t'}{D + 2d_0} \qquad (7.6)$$

where *C* is the cover depth of concrete and *ft*' is the tensile strength of concrete.

From Eq. (7.5) and (7.6), d_s may be expressed as

$$d_s = \frac{Cf_t'}{E_{ef}}\left(\frac{a^2 + b^2}{b^2 - a^2} + V_c\right) \qquad (7.7)$$

Therefore, the critical amount of corrosion products needed to induce cracking of the cover concrete can be estimated from Eq. (7.4) and (7.7)

$$W_{crit} = \rho_{rust}\left(\pi\left[\left(\frac{Cf_t'}{E_{ef}}\left(\frac{a^2 + b^2}{b^2 - a^2} + V_c\right) + d_0\right]D + \right.\right.$$

(7.8)

It can be seen that the critical amount of corrosion products needed to induce cracking of the cover concrete is mainly dependent on the tensile strength of concrete, cover depth, elastic modulus of the concrete, and properties (void structure) of steel/concrete interface.

3. Growth of rust products

As the rust layer thickens, the iron ionic diffusion distance increases, and the rate of rust production decreases because diffusion is inversely proportional to the oxide layer thickness. The rate of rust production can be written as follows

$$\frac{dW_{rust}}{dt} = \frac{k_p}{W_{rust}} \qquad (7.9)$$

where W_{rust} is the amount of rust products (mg/mm), t is corrosion time (year), and k_p is the rate of rust production. Note that k_p is related to the rate of metal loss, may be expressed in terms of corrosion rate

$$k_p = 0.098(1 + \alpha)\pi D i_{corr} \qquad (7.10)$$

in which a is related to types of rust products, D is the steel diameter (mm), and i_{cor} is the annual mean corrosion rate (mA/ft^2).

Integrating Eq. (7.9), the growth of rust products can be obtained

$$W_{rust}^2 = 2\int_0^t k_p\, dt \qquad (7.11)$$

By knowing the corrosion rate, the amount of rust products for a certain period of corrosion can be estimated.

4. Time-to-cracking

As stated earlier, when the total amount of corrosion products reaches the critical amount of rust products, the internal expansion stress will exceed the tensile strength of concrete and cause the cracking of the cover concrete. According to Eq. (7.11), for a constant corrosion rate, the time-to-cracking *tcr* can be given as follows

$$t_{cr} = \frac{W_{crit}^2}{2k_p} \qquad (7.12)$$

where W_{crit} is the critical amount of corrosion products.

7.5 MODEL APPLICATIONS ON EXPERIMENTAL DATA

Bazant model was applied on the specimen of $300x300mm^2$ with different cover depth of 20mm.

For the outdoor specimens, fc = 4500 psi (31.5 MPa),

Let us take tensile strength ft¢ = 472 psi (3.3 MPa);

Elastic modulus of concrete Ec = 3,900,000 psi (27 GPa), and

Creep coefficient φ_{cr} = 2.0; Poissons ratio ν_c = 0.18;

Density of rust products ρ_{rust} = 225 lb/ft3 (3600 kg/m3);

Assume the composition of rust products is between Fe(OH)3 and Fe(OH)2 α varies from 0.523 to 0.622; and also, assume that the thickness of porous zone equals 4.9 mils (12.5 mm).

Table 7.1: Predicted Service life of the Experimental data

Water Cement Ratio	16mm	20mm	25mm
0.51	3.87 yrs	6.92 yrs	15 yrs
0.48	3.97 yrs	7.26 yrs	15.43 yrs
0.45	3.95 yrs	7.46 yrs	15.81 yrs
0.42	4.22 yrs	7.9 yrs	16.28 yrs
0.39	4.41 yrs	7.96 yrs	17 yrs

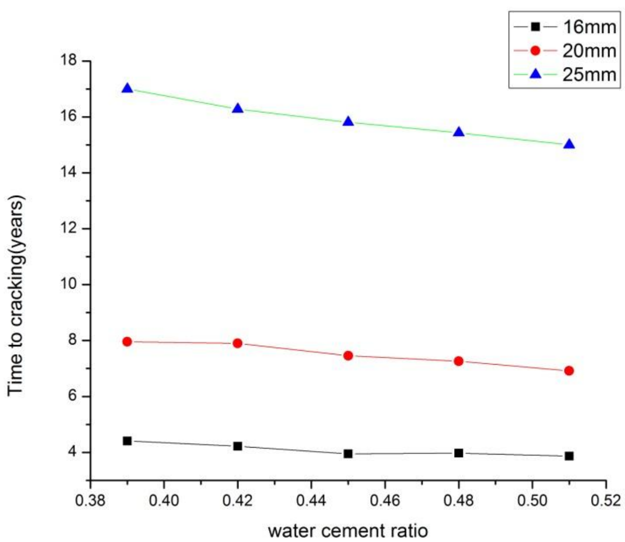


Fig. 7.3 Effect of water cement ratio on time to cracking

From the graph given in fig. 7.3 it is observed that the predicted service life of the specimen for 0.39 water cement ratio with 25mm diameter bar is maximum i.e. 17yrs.

CHAPTER 8 CONCLUSION

Service life is the period of time after manufacture during which the performance requirements are fulfilled. It depends upon the performance and degradation of the structure. Service life of a structure consists of two phases- initiation phase and the propagation phase. Different models are available to predict the service life of a structure depending upon whether it is initiation phase or propagation phase. The following observations were made:

From the experimental results it is concluded that the half cell measurements keep on increasing with time but within a certain defined range. Irrespective of bar diameter, the time required to reach the threshold corrosion potential increase with the decrease in water cement ratio. It is because of the decrease in permeability with decrease in water cement ratio.

From the determined corrosion current density values by LPR method, it is concluded that the corrosion current density decreases with decrease in water cement ratio. As the corrosion progresses in the slabs the value of corrosion current (I_{corr}) rises. This is because of the depassivation of layer formed around the rebar due to the concentration of chlorides.

Both the electrochemical techniques are good for the measurement of corrosion rate but half cell potential is effective only in monitoring the corrosion initiation and LPR measurements are necessary as the values obtained from corrosion current density indicates the progression of corrosion in the propagation phase. Therefore, linear polarization method gives better results than half cell potential measurement.

Service life of the structure can be calculated from the corrosion current values by using simple mathematical expressions based on corrosion cracing theory. The service life of the structure is found to increases with decrease in water cement ratio by using this theory. The results of this investigation may provide a basis for repair, strengthening, and demolition of existing RC bridges.

References

1. AL-amoundi (1985), "Studies on the evaluation of permeability and corrosion resisting characteristics of Portland pozzolan concrete."
2. Amey S. L., Johnson D. A., Miltenberger M. A., and Farzam, H. (1998), ‘‘Temperature dependence of compressive strength of conversion inhibited high alumina cement concrete.’’ ACI Struct. J., 95(1), 27– 36.
3. Andrade C., Gonzalez J.A., and Lopez. W (1992), " Influence of temperature on the service life of rebars." Cement and Concrete Research. Vol. 23, pp. 1130-1140.
4. Azad A.K., Ahmad S., and Al-Gohi B.H.A (2010)," Flexural strength of corroded reinforced concrete beams"(2010) Magazine of Concrete Research, 62, No. 6, June, 405–414.
5. Bazant, Z. P. (1979), ‘‘Physical model for steel corrosion in concrete sea structures-theory.’’ *J. Struct. Div., ASCE*, 105 (6), pp. 1137–1153.
6. Bazant, Z. P. (1979), ‘‘Physical model for steel corrosion in concrete sea structures-application.’’ *J. Struct. Div., ASCE,* 105 (6), pp.1155–1166.
7. Bamforth P.B.(1999), " The derivation of input data for modeling chloride ingress from eight –year UK coastal exposure trials", Magazine of concrete research, V.41,No. 2, pp87-96.
8. Bertolini L.(2008), "Steel corrosion and service life of reinforced concrete structures"; Structure and Infrastructure Engineering, Vol. 4, No. 2, pp 123 – 137.
9. Funahasi M.(1990), "Predicting **c**orrosion free service life of a concrete structure in a chloride environment", ACI Materials J., Nov./Dec., pp. 581-587.
10. Hoseini M., Chini M., Alizadeh R., Ghods P., and Shekarchi M.(2008), **"**DuraPGulf, A local service life model for the durability of concrete structures in the south of iran." The Arabian Journal for Science and Engineering, Volume 33, Number 1B.
11. Kim S.H., Ann K.Y., and Pack S-W.(2010), "Service life prediction of reinforced concrete structures subjected to carbonation." Construction and building material Vol. 24; 1494-1501.
12. Kropp J. (1995), "Relations between transport characteristics and durability". In: Kropp J, Hilsdorf HK, editors.Performance criteria for concrete durability, Rilem report 12. E&FN SPON; pp. 97–111.

13. Li C.Q.(2003),“Life-Cycle modeling of corrosion-affected concrete structures: propagation.” Journal of Structural Engineering Vol.129, No.6, pp: 753-761.
14. Liam K.C., Roy S.K., and Northwood D.C. (1992), "Chloride Ingress measurements and corrosion potential mapping study of a 24-year old reinforced concrete jetty structure in a tropical marine environment", Mag Concrete Res., Vol. 44, No. 160, Sept., pp. 205-215.
15. Liang M.T., Wang K.L., and Liang C.H.,(1999); “Service life prediction of reinforced concrete structures.”; Cement and Concrete Research 29; 1411–1418.
16. Liu Y., and Weyers R.E.(1998), “ Modeling the time to corrosion cracking in chloride contaminated reinforced concrete structures.”ACI Materials Journal,V.95,No.6,PP 675-681.
17. Liu, Y. & Weyers, R.E. 1996, ‘Time to cracking for chloride-induced corrosion in reinforced concrete”, in Corrosion of Reinforcement in Concrete Construction, eds C.L. Page, P.B. Bamforth, & J.W. Figg, Royal Society of Chemistry, Cambridge, England, pp. 88-104.
18. Malami C, Kaloidas V, Batis G, and Kouloumbi N.(1994),” Carbonation and porosity of mortar specimens with pozzolanic and hydraulic cement admixtures.” Cem Concr Res; 24(8), pp. 1444–54.
19. Mangat P.S., and Molloy B.T., “ Prediction of long term chloride concentration in concrete,” Materials and structures, V. 27, No. 140, 1994, pp. 338-346.
20. Midgley H.G. and Illston J.M. (1984),” The penetration of chloride into hardened Cement pastes”, Cement and Concrete Res, Vol. 14, pp. 546-558.
21. Nagano H. and Naito T. (1985), "Application of diffusion theory to chloride penetration into concrete located in splashing zone", Trans. Japan Concrete Inst., Vol. 7, pp. 157-164.
22. Raharinaivo A. and Jean-Marie G. (1986), "On the corrosion of reinforcing steel in concrete in the presence of chlorides", Materiales de Construccion, Vol. 36, No. 294, Oct./Nov./Dec., pp. 5-16.
23. Sarja A. and Vesikari E. (1996),”Durability design of concrete structures.” Report of RILEM Technical Committee 130-CSL.
24. Schiessl P. (1976), “Zur Frage der zulässigen Rissbreite und der erforderlichen Betondeckung in Stahlbetonbau unter besonderer Berücksichtigung der Karbonatisierung des Betons”. Deutscher Ausschuss für Stahlbeton, Berlin, Vol. 255, 175 pp.

25. Stewart M.G. and Vu Kat (2002) “Service life prediction of reinforced concrete structures exposed to aggressive environments.”
26. Thomas MDA and Matthews JD.(1992),” Carbonation of fly ash concrete.” Mag Concr Res ;44(160): pp.217–28.
27. Thomas M.D.A., and Bentz E.C.(2000), “Life 365: Compter programme for predicting the service life and Life cycle cost of reinforced concrete exposed to chloride”. pp. 43.
28. Tuutti K.(1982), “ Corrosion of steel in concrete.” Swedish Cement and Concrete Research Institute, Stochkolm.
29. Vu K., Stewart M.G., and Mallard J. (2005), “Corrosion induced cracking: experimental data and predictive models”, V.102, No. 5, pp. 719-726.
30. Weyers R.E. and Smith D.G. (1989), "Chloride diffusion constant for concrete", Structural Materials, New York, The Society, pp. 106-115.
31. Weyers R.E., Prowell B.D., Sprinkel M.M., and Vorster M.(1993), “Concrete bridge protection, repair, and rehabilitation relative to reinforcement corrosion:” A method application manual, strategic highway Research Program, Washington,DC, pp. 268.
32. Williamson G.S., Weyers R.E., Brown M.C., Ramniceanu A. and Sprinkle M.M., (2008),” Validation of probability based chloride induced corrosion service life model.”; ACI Materials Journal; pp 375-380.
33. Yang Y.W. (1994),” A study of diffusion behavior of chloride ion in high strength concrete.” Master Thesis, National Taiwan Ocean University, (in Chinese).