

ABSTRACT

Title of Thesis: EVALUATION OF CONCRETE SUBSTRUCTURE
ELEMENTS TO ASSIST IN DECISION MAKING OF
REMOVAL VS RETAINMENT/RESTORATION

Sanjeev K. Howlader, Master of Science, 2008

Thesis directed by: Research Professor, Chung C. Fu
Department of Civil and Environmental Engineering

Reinforced concrete bridge substructures are affected a great deal by chloride ions entering from deicing salts or saltwater. They destroy the passivity of the steel and cause it to corrode. A chloride diffusion spreadsheet model will help us to predict the amount of chloride concentration around the surface at different depths. A diffusion process was assumed for the chloride ingress. The binding materials, surface chloride, chloride diffusion coefficient, age factor and threshold value –all will have a great influence on the outputs of the model. Later, the result of the model is compared with that of another model life-365. Finally, a correlation is developed between the state's practice methods and the outputs from the model. This relationship would be very effective in determining the condition of the substructures of the bridges and in making a decision for either removal or rehabilitation while considering cost and time.

**EVALUATION OF CONCRETE SUBSTRUCTURE ELEMENTS TO
ASSIST IN DECISION MAKING OF REMOVAL VS
RETAINMENT/RESTORATION**

by

Sanjeev Kumar Howlader

Thesis submitted to the Faculty of the Graduate School of the
University of Maryland, College Park in partial fulfillment
of the requirements for the degree of
Master of Science
2008

Advisory Committee:

Professor Chung C. Fu, Chair

Professor Amde M. Amde

Professor M. S. Aggour

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Chapter 1

Introduction

1.0 Background

Enactment of the Federal Highway Act of 1956 created the needed funding mechanism for construction of our nation's modern interstate highway system. While the act created a 90–10 funding appropriation, with the federal government bearing the greater burden, the engineering birth of the Interstate highway system was created more than 25 years earlier with the issuance of a unified approach to highway design. Highway design originated with the publication of the ASCE's 1924 *Final Report on Specifications for Design and Construction of Steel Highway Superstructures* and later refined in the first edition (1931) of AASHTO's *Standard Specification for Highway Design*.

Prior to the Silver Bridge collapse (1967 resulting in 46 fatalities), there was no comprehensive, nationwide database of information about the number, type, location, and condition of our nation's bridges. The safety of our nation's bridges was again brought into question on April 5th, 1987, when disaster struck with the collapse of the New York State Thruway (I-90) Bridge across the Schoharie River. Localized flooding caused scour at a central pier, which was followed by a subsequent loss of bearing capacity at the foundation. Lessons learned from the failures, subsequent and ongoing advancements in research, and experience gained from more than 40 years of bridge monitoring and inspection has produced an educated work force of bridge inspectors who proactively monitor the safety of our nation's bridges. A summary of major bridge inspection and bridge program funding legislation is provided in Table 1.1.

The National Bridge Inventory System (NBIS) requires periodic visual inspection, with most structures (82%) evaluated once every 2 years. When safety concerns exist such as from fatigue, scour, and advanced deterioration, etc., inspection intervals may be more frequent. Approximately 14% of the nation's bridges are inspected at intervals of less than 2 years. Similarly, for structures with characteristics that have historically been free of concern, the period of observation may be increased to 4 years. Only 2% of bridges are inspected at intervals of greater than 2 years. NBI data can be used to quantify the bridge types constructed over a given period (Figure 1.1.)

Through periodic safety inspections, condition state data is collected on the following structural components:

- The bridge deck, including the wearing surface;
- The superstructure, including all primary load carrying members and connections;
- The substructure, considering the abutments and all piers;
- Culverts, recorded only for culvert bridges; and
- Channel and channel protective systems, for all structures crossing waterways.

According to the FHWA report, “Status of the nation’s highways, bridges, and transit: 2004 conditions and performance,” overall there are 162,869 bridges that are deficient within the highway bridge network. This represents 27.5% of the total inventory of highway bridges when bridges are weighted equally (FHWA 2004). According to the FHWA report, corrosion damage caused by deicing salts is considered one of the main problems that cause a bridge structure to be structurally deficient. State DOTs are using condition rating methods followed by load rating calculations for evaluating existing bridges in the United States. The AASHTO *Manual for Condition Evaluation and LRFR of Highway Bridges* (AASHTO 2003), as well as FHWA and State DOT’s inspection manuals provide provisions for determination of the safety and serviceability of existing bridge components.

On a bridge structure, the most vulnerable components to the routine application of deicing salts, repeated freeze-thaw cycles, and other damaging effects are bridge decks and elements of substructure under bridge joints. They are constantly experiencing observable deterioration. The concern arose in the 1970s when bridge decks designed for an expected service life of 30 to 50 years began to deteriorate after only 20 years in service. In addition to bridge decks, their supporting members, such as abutment walls, piers, and exposed piles, are also subject to deterioration, even more so those in coastal environments. Based on years of experience, Maryland has gained a certain degree of knowledge on bridge decks, but not enough on bridge substructures.

1.1 Background on Bridge Inspection

The NBIS uses a 10-point rating system, with 9 representing excellent, as-new condition and 0 representing a failed condition. A summary of the rating system is provided in the Table 1.2. Ratings of 3 and less classify the component as being deficient.

Bridge inspectors assign condition ratings based on experience, training and visual review of the subject component. The condition ratings are used to describe the existing, in place status of a component, not its

as-built state. Engineers assign condition ratings by evaluating the severity of deterioration or disrepair and the extent to which it is widespread throughout the component being rated. This methodology is highly subjective and produces variable results. Research findings from FHWA's study of the reliability of the visual inspection method found significant variability of condition rating assignments using visual inspection.

The status of our nation's bridges is based on the information contained in the NBI, which includes nine different appraisal ratings. The various appraisals used to classify the service state of our bridges are shown in the vertical axis of the Figure 1.2. The horizontal bars represent the number of bridges classified as either deficient (solid) or non-deficient (hatched) by the NBIS.

1.2 Problem Statement

On a bridge structure, the most vulnerable components to the routine application of deicing salts, repeated freeze-thaw cycles, and other damaging effects are bridge decks and elements of substructure under bridge joints. They are constantly experiencing observable deterioration. The concern arose in the 1970s when bridge decks designed for an expected service life of 30 to 50 years began to deteriorate after only 20 years in service. In addition to bridge decks, their supporting members, such as abutment walls, piers, and exposed piles, are also subject to deterioration, even more so those in coastal environments. Based on years of experience, Maryland has gained a certain degree of knowledge on bridge decks, but not enough on bridge substructures. Two questions must always be answered when rehabilitation work is required on a bridge: (1) How can the condition of the substructure be assessed? (2) Based on the assessment, can the bridge substructure unit be rehabilitated or must it be replaced? Having accurate assessment information is essential, since the cost associated with replacing every bridge substructure in doubt is extremely high. Therefore, typical damage mechanisms and test methods, including destructive and non-destructive testing, for determining the extent of damage due to chlorides need to be identified and correlations between the two be reported.

1.3 Research Objectives

The objective of this study is to identify typical damage mechanisms and test methods, including destructive and non-destructive testing, for determining the extent of damage on substructure elements and to develop correlations between testing results and substructure condition for MDSHA consideration on whether rehabilitation or replacement of a given bridge substructure element is most appropriate.

1.4 Approaches to Problem Solving

Implementation: The study is divided into four tasks:

Task 1 Collect and Study the State-of-the-Art and State-of-the-Practice Methods throughout the Bridge Community including Specifically Maryland Bridges with Substructures Replaced or Rehabilitated due to Material Problems.

Concrete bridges all experienced some type of concrete deterioration (e.g., cracks, spalling, erosion, staining, and corrosion of reinforcement). If untreated, the deterioration can worsen and eventually require major concrete repairs. Corrosion of the reinforcement is recognized as a major contributor in concrete deterioration and chloride attack is a major concern in causing the corrosion. When chloride ions from deicing salts or saltwater enter reinforced concrete bridge substructures they destroy the passivity of the steel, causing it to corrode. The focus of this phase is to locate and list all the available current state-of-the-art and state-of-the-practice methods for (1) corrosion detection, (2) chloride sampling and chloride testing, (3) condition assessment, and (4) common decision-making practice based on condition assessment. Published material on the subject areas will be searched through TRB, ASCE, the Transportation Research Information Services (TRIS), the National Technical Information Service (NTIS), the Transportation Research Laboratory (TRRL) and other states. The research team will also search the historical record of Maryland bridges across the state for replaced or rehabilitated substructures due to material problems over the last five to ten years. Material test results will be collected from the Office of Material Testing (OMT). Annual inspection reports of those bridges will be collected to find their years in service, concrete types, environments, and traffic. Also documented are the presence and type of bridge joints, salting history and frequencies. The focus will be on the causes for the replacements.

Task 2 Field investigation of replaced/rehabilitated substructure units and potentially problematic substructure elements.

This field investigation will include those substructure units being replaced / rehabilitated due to material problems and those substructure units which have similar problems and might be replaced / rehabilitated.

Use of core removal, coordinated through SHA/OMT, will be conducted if required to check corrosion, delamination, and chloride content. Further coring drills may be required to draw the potential map provided the integrity of the substructure unit will not be diminished. For those substructure units being replaced during the study period, a field observation of the material during the demolition process will be made and samples collected for laboratory testing.

Task 3 Develop correlations between testing results and substructure condition for SHA consideration in determining which substructure units can be rehabilitated and which must be replaced. Common problems due to material encountered during the inspection of piers and abutments include disintegration of the concrete, cracks at the pier bent caps and bearing seats, and presence of corrosion. Based on field investigations, material testing and analysis results, the research team will examine the severity of those problems and find correlations between the results and the substructure condition. The correlation will be in the expression of charts and figures to provide an easy-to-understand aid to be used by MDSHA to assist in making the decision to rehabilitation or replace a substructure unit.

Table 1-1: Summary of Major Bridge Inspection and Bridge Program Funding Legislation and Noteworthy Changes

Act and Date	Requirements
Federal Aid Highway Act of 1970 (P.L. 91-605)	<ul style="list-style-type: none"> • Inventory requirement for all bridges on the federal aid systems • Established minimum data collection requirements • Established minimum qualifications and inspector training programs • Established SBRP
Surface Transportation Assistance Act of 1978 (P.L. 95-599)	<ul style="list-style-type: none"> • Established HBRRP (extending funding to rehabilitation) to replace SBRP • Extended inventory requirement to all bridges on public roads in excess of 6.1 m • Provided \$4.2 billion for the HBRRP, over 4 years
Highway Improvement Act of 1982	<ul style="list-style-type: none"> • Provided \$7.1 billion for the HBRRP over 4 years
Surface Transportation and Uniform Relocation Assistance Act of 1987	<ul style="list-style-type: none"> • Provided \$8.2 billion for the HBRRP over 5 years. • Added requirements for underwater inspections and fracture critical inspections • Allowed increased inspection intervals for certain types of bridges
Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA)	<ul style="list-style-type: none"> • Provided \$16.1 billion for the HBRRP over 6 years • Mandated state implementation of BMSs
National Highway System Designation Act of 1995	<ul style="list-style-type: none"> • Repealed mandate for management system implementation
Transportation Equity Act for the 21st Century (TEA-21)	<ul style="list-style-type: none"> • Provided \$20.4 billion in HBRRP funding over 6 years
Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU)	<ul style="list-style-type: none"> • Provided funding for systematic preventive maintenance, in addition to bridge replacement and rehabilitation • Appropriates \$21.6 billion in bridge program funding over 5 years

Source: FHWA (2004 *Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance*)

Table 1-2: Bridge Condition Ratings

Rating	Category	Description
9	Excellent condition	
8	Very good condition	
7	Good condition	No problems noted.
6	Satisfactory condition	Some minor problems.
5	Fair condition	All primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	Poor condition	Advanced section loss, deterioration, spalling, or scour.
3	Serious condition	Loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical condition	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	Imminent failure condition	Major deterioration or section loss present in critical structural components, or obvious loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back into light service.
	Failed condition	Out of service; beyond corrective action.

Source: FHWA (*Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges*, 1995)

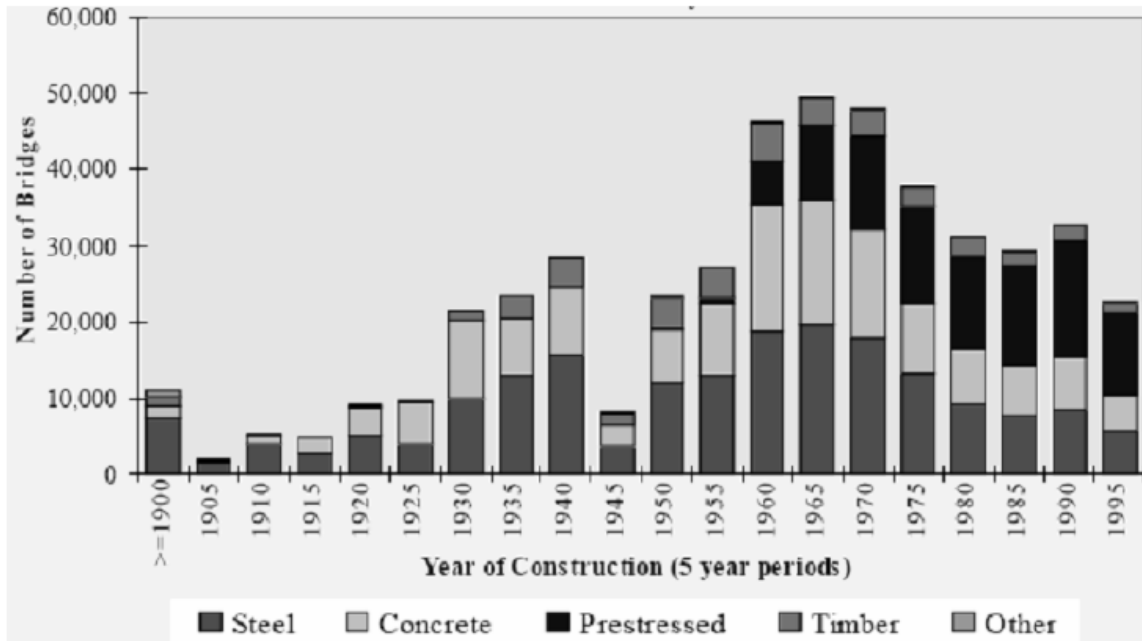


Figure 1-1: Distribution of bridges by year of construction and type of material

(Source: *Transportation Research Circular E-C104: 50 Years of Interstate Structures: Past, Present, and Future*)

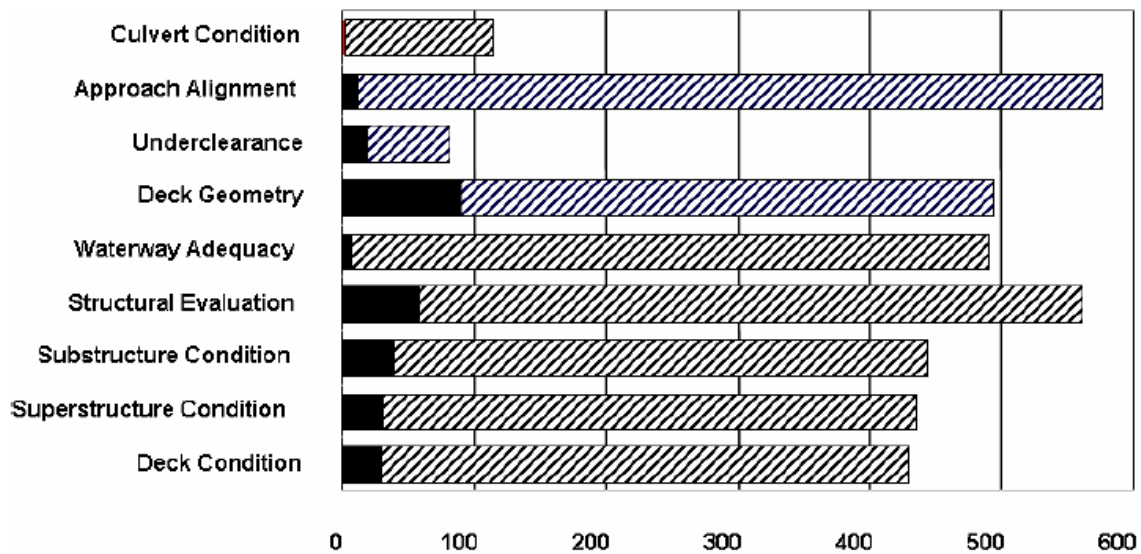


Figure 1-2: Summary of Bridge deficiencies from 2002 NBI data

(Source: *2004 Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance*)

Chapter 2

Methods of Assessment

2.1 Introduction

The durability of concrete substructures is essentially influenced by the process that involves the passage, into or through the material, of ions or molecules in the form of liquids and gases. The transport processes involved in the passage of potentially harmful agencies through concrete are:

- Gaseous diffusion (Oxygen, Carbon Dioxide)
- Vapor diffusion (Moisture movement)
- Ionic diffusion (Chlorides, Sulfates)
- Absorption and capillary rise (Chlorides dissolved in water)
- Pressure induced flow (Aggressive groundwater , freeze/ thaw)

Bridge rehabilitation is imminently connected with assessment and evaluation of the technical condition of the bridge performance prior to repair or retrofit works. Assessment and evaluation processes are based on special inspection systems and on field and laboratory tests as well as on advanced theoretical analysis in many cases. Depending on the bridge, its scale and structural system as well as its importance for the users, this process can be relatively complex and require special instrumentation and equipment. All the deterioration processes of bridge structures can be schematically summarized in the Figure 2.1. It can be seen that deterioration passes through two different phases, namely initiation and propagation.

According to S. Rostam (Workshop Ed. by G.Koenig & A.Nowak, 1992), no noticeable weakening occurs during the initiation phase but some protection barrier is overcome by aggressive media during the propagation phase, accelerated deterioration can be observed. In the initiation phase, no visual damage is observed in general, while in the propagation phase, visual damage usually occurs. Damage to the bridge structures can develop up to a certain acceptable limit or can even exceed this limit as shown in Figure 2.1. It should be emphasized that the “acceptable limit” is a decisive factor for safety and serviceability of bridge structures, which depend mostly on the structural material and system of the bridge, the type of traffic, the required stiffness of the structure and its durability, the required standard of bridge utilization (e.g., the bridge is located on a primary or a secondary road), etc.

“Acceptable limit” is usually determined by design or requirement and testing codes or other official regulations. Depending on the national or international regulation, the “acceptable limit” can be different

for particular requirements. However, the following structural and material criteria are generally taken into account:

- (a) Allowable stress level in concrete and steel or other used materials under dead and live loads as well as under any other type of loading, e.g. thermal effects.
- (b) Required stiffness of primary and secondary structural elements expressed in general by their deflections under live loads and usually presented as a certain fraction of the span of the structural element
- (c) Allowable differences in the settlement of the bridge piers and abutments (i.e., allowable non-uniform settlement of the structure)
- (d) Allowable crack width in concrete (usually 0.2 mm in normal environmental conditions and 0.1 mm in aggressive ones).

Moreover there are some other criteria of more specific nature such as the following ones:

- (e) Allowable level of vibration of the structure or its individual members regarding both the bridge safety (e.g. fatigue effects) and the standard of the bridge utilization (e.g., influence of excessive vibration of a pedestrian bridge on its users)
- (f) Required level of structural resistance to wind pressure and other wind effects
- (g) Required level of structural resistance to seismic effects.

It is obvious that the technical service life of a bridge structure is the time in which the initiation and propagation phases of bridge deterioration reach the “acceptable limit” criterion for the given bridge. After that time, bridge rehabilitation is necessary because one of the basic factors of the criteria denoted above exceeds the allowable or required level “acceptable limit” may be reached and then exceeded by the whole structure, certain parts of it or its individual structural members. Therefore, repair or rehabilitation works can be on a different scale from general to local only. The relevant decision should be based on the results of bridge inspection, monitoring or testing, depending on the scale of bridge deterioration and the importance of the bridge for the road traffic. Assessment and evaluation of the technical condition of bridge structures lead to the determination of whether the “acceptable limit” is exceeded or not and to the prediction of it and when (after how many years) this limit may be reached and exceeded. A general strategy of the investigation can be expressed in graphical form as shown in Figure 2.2, according to S. Rostam (workshop Ed. by G.Koenig & A.Nowak, 1992). A staged assessment is usually based on the selection of investigations of which type, number and location should be “a balance between precise tests giving deep local insight and general surveys giving an overview”.

The first stage is, in the great majority of cases, a general in-situ survey, which allows a preliminary estimation of safety of the bridge and an indication of the immediate safety precautions if they are needed. This stage is usually based on an inspection system, which is presented below:

The second state is based on numerous investigation techniques, which are available both in field and laboratory tests. The choice of technique should be made individually depending on the bridge type, structural detection and monitoring needs to assess the technical condition of the structure. The assessment itself is “a complex interaction between structural, environmental and service data, data from visual inspection, test data from in-situ and laboratory investigation”

2.2 Tests for Assessment

The following tests are associated with the condition assessment of the substructure of a Bridge.

- **Drilling a Core:** Cores of concrete are very often extracted from in situ structures in order to determine such properties as density; tensile strength; compressive strength; carbonation and permeability. The diameter of the core should be as large as possible to ensure that the local effects of the aggregates do not adversely affect the results. In the USA it is usual to cut a 1”,2” & 3” (25, 50 & 75 mm) diameter core, whereas in the UK it is 150 mm and 100 mm is becoming common. A water cooled diamond-tipped overcoring drill bit is used which is mounted on a stand and can be bolted to the surface of the concrete being examined. A typical arrangement is shown in Figure 2-3. If the permeability of the concrete is not being determined, then after the core has been extracted, it is trimmed at each end using a water-cooled diamond tipped rotary saw. Care should be taken not to cut through any steel reinforcement, or if this cannot be avoided, then a judgment must be made on whether such local damage will affect the strength of the bridge.
- **Using the Rebound Hammer:** This technique is a nondestructive test used to determine the superficial compression strength of concrete using very simple portable equipment. It is based on the relationship between the hardness of the concrete surface and its compression strength. The equipment measures the rebound of a calibrated weight that is initially compressed by a spring against the surface. The impact energy is well defined and the rebound of the hammer mass is dependent on the hardness of the concrete. Rebound values are indicated on a gauge built into the instrument. It is mostly a comparative technique. A number of measurements are therefore

required in the same location from which the mean and standard deviations values can be determined. The typical instrument is shown in Figure 2-4.

- **Ultrasonic Test:** The velocity of ultrasonic pulses traveling in a solid medium depends upon the density and elastic properties of the medium. The transmission of such pulses can, therefore, provide information on the integrity of plain, reinforced or prestressed concrete bridges. The ultrasonic technique measures the transit time of sound waves passing from an emitter transducer through the concrete to a receiver transducer. The pulse velocity can be calculated if the length of the path taken by the pulse is known. It is then possible to assess the quality of the concrete in relation to
 - a. The homogeneity of the concrete
 - b. The presence of voids, cracks or other imperfections
 - c. Changes in the concrete which occur with time
 - d. The quality of the concrete relating to strength.

- **Using Covermeter:** A covermeter is a device that detects the bars on the alteration they introduce a fixed magnetic field of the search unit. It is strictly nondestructive method whose results are affected simultaneously by the bar diameter and the cover thickness. It gives reliable indication about the bars position but it needs to be calibrated in some sections where the cover is really measured, in order to be used as bar diameter and cover indicator. X rays have also been used to detect reinforcement and prestressing steel. A radiation source is placed on one side of the concrete element and film sheets are placed on the other side. The radiation passing through concrete shows a picture of the existing reinforcement, becoming clearer as the reinforcement level is closer to the film. It is a method whose application has the drawback of being limited to concrete elements with a small thickness, which requires a long period of X-ray exposure, leading to the implementation of health protection measures.

- **Depth of Carbonation Test:** Carbonation is a naturally occurring phenomenon which occurs as the result of absorption of carbon dioxide from the atmosphere into pore water. To determine the depth of carbonation, the most usual test is to use a pH indicator.
 - ✓ **Half Cell Potential:** The half-cell potential is a parameter indicating the corrosion situation of a metal in a specific environment. The type of concrete, resistivity, humidity, carbonation etc., affect the potential value, so this test is mainly used to obtain maps of equal potential levels and to determine the zones where active corrosion has a high probability of occurring. The technique uses a reference electrode, as external gauge, that

is connected to a reinforcement bar. The potential between the electrode and the bar is measured at several points, from which a potential map of corrosion can be produced. The relationship between concrete cover and the difference between the potential values of passive and corroding steel is inversely proportional. An increase in concrete cover decreases the difference between the potential values of passive steel and actively corroding steel and may cause the potential values to become nearly identical. Therefore, locating small corroding areas becomes extremely difficult with increasing cover depth. As illustrated in Figure 2.5 the Half-cell potential measurements are affected by the resistivity of the concrete, which in turn is affected by concrete pore water and ion concentrations in the pore solution. Researchers have shown that reduced electrical resistance of the concrete increases the current flow in the reference copper-copper sulfate electrode (CSE), resulting in a lower half-cell potential reading that may suggest the presence of active corrosion

ASTM C876 (Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete, pp. 9-14) specifies that potential measurements more negative than -0.35 V using a copper-copper sulfate electrode (CSE) indicate a probability greater than 90 percent that corrosion is occurring. Potential measurements that are more positive than -0.20 V CSE indicate a probability greater than 90 percent that corrosion is not occurring in that area. Potential measurements between -0.20 and -0.35 V CSE indicate that corrosion in that area is uncertain. However, studies have been conducted that conflict with these threshold values designated in ASTM C 876 (Stratfull, R. F., 1973 and Elsener B. and H. Bohni., 1990). Therefore, published threshold values in ASTM C 876 should only be used as guidelines since a precise delineation of steel from a passive to an active state cannot be made to encompass all bridges. The Connecticut DOT specified that action should be taken when more than 40 percent of the potential measurements are more negative than -0.35 V CSE (Hema J., W. S. Guthrie, and F. Fonseca, 2004). The Rhode Island DOT confirmed that action should be taken when values are below -0.35 V CSE, provided that other forms of deterioration are present (Hema J., W. S. Guthrie, and F. Fonseca, 2004). It is noted that ASTM C876 (08) has not been approved yet. The reference values on this text are based on ASTM C876 (99).

- **Concrete Resistivity Test:** The rate at which rebar corrosion occurs in concrete is generally controlled by the resistivity (resistance per unit length of unit cross section) of the concrete. The easier it is for the corrosion current to flow through the surrounding concrete, the greater the amount of metal loss from a corroding length of reinforcement. The circulation of ions in concrete can be analyzed through its electrical resistivity. This analysis gives information on concrete quality, namely related to chloride diffusion. Resistivity measurements predict the probability of suffering significant levels of corrosion when half cell tests show that corrosion is possible. Resistivity, which is the inverse of electrical conductivity, is a measure of the ability of a material to behave as an electrolyte, or to support corrosive electrical currents. The electrical resistance of concrete is measured in Ohm centimeters. The resistivity meter measures both the current (I) and voltage (V) converts the readings to resistance within the meter, and stores the gathered data in a semi-permanent memory. The ability of a material to resist ionic current flow depends upon both the porosity and water content of the medium. For example, very porous concrete with a high degree of saturation has a much lower resistivity than denser concrete with lower water contents; porous, saturated concrete permits soluble ions from deicing salts and other sources to more readily infiltrate the concrete. Consequently, the rate of corrosion dramatically increases as chloride ions migrate through the concrete to the reinforcing steel at faster rates and accumulate in higher concentrations within the concrete.

- ✓ **AASHTO T259:** Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration (Salt Ponding Test): The AASHTO T259 test (commonly referred to as the salt ponding test) is a long- term test for measuring the penetration of chloride into concrete. The test requires three slabs at least 75 mm thick and having a surface area of 300 mm square. These slabs are moist cured for 14 days then stored in a drying room at 50 percent relative humidity for 28 days. The sides of the slabs are sealed but the bottom and top face are not. After the conditioning period, a 3 percent NaCl solution is ponded on the top surface for 90 days, while the bottom face is left exposed to the drying environment (see Figure 2.6). At the end of this time the slabs are removed from the drying environment and the chloride concentration of 0.5-inch thick slices is then determined [AASHTO T259].

Typically, 2 or 3 are taken at progressive depths. There is difficulty, however, in determining what the results mean. Part of this is due to the complicated testing conditions, discussed in the following paragraph, but part is also due to the crudeness of the evaluation. Little information is being gathered about the chloride profile. Only the

average chloride concentration in each 0.5-inch slice is determined, not the actual variation of the chloride concentration over that 0.5-inch. A situation could be envisioned where there are two concretes with the same average chloride concentration in their outer 0.5-inch slice. One concrete has an approximately uniform chloride concentration, while the other has a higher concentration near the surface and is lower further in. Obviously the first situation will result in a critical chloride concentration reached at some depth sooner than the second situation, yet this distinction would not be detected. The salt ponding test does provide a crude one-dimensional chloride ingress profile, but this profile is not just a function of chloride diffusion. Since the specimens have been left to dry for 28 days, there is an initial sorption effect when the slabs are first exposed to the solution. Salt solution is drawn quickly in to the pores of the concrete. Also, the exposure of the bottom face to a 50 percent relative humidity environment during the test causes chlorides to be drawn into the concrete through a mechanism other than pure diffusion. There is vapor transmission from the wet front in the concrete to the drier atmosphere at the external face, causing more water to be drawn into the concrete and bringing chloride ions with it. This effect is called wicking. While all these transport mechanisms may be present in a structure, the relative importance of each is not necessarily reflected by this test procedure. The test overemphasizes the importance of sorption, and to a lesser extent wicking. The relative amount of chloride pulled into the concrete by capillary absorption to the amount entering by diffusion will be greater when the test is only 90 days than when compared to the relative quantities entering during the lifetime of a structure. Also, if wicking is occurring in the concrete element of interest, the relative humidity gradient will likely be less, at least for part of the time, than that which is set up during the test. For some higher quality concretes, there has also been difficulty in developing a sufficient chloride profile. Insufficient chloride may penetrate in the 90-day duration for a meaningful profile to develop. This has resulted in a need to extend this duration to allow the evaluation of higher quality concretes [AASHTO T259].

- ✓ **AASHTO T277:** Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (Rapid Chloride Permeability Test) : In the AASHTO T277 (ASTM C1202) test, a water-saturated, 50-mm thick, 100-mm diameter concrete specimen is subjected to a 60 V applied DC voltage for 6 hours using the apparatus shown in Figure 2.7. In one reservoir is a 3.0 % NaCl solution and in the other reservoir is a 0.3 M NaOH solution. The total charge passed is determined and this is used to rate the concrete according to the

criteria included as Table 2.3. This test, originally developed by Whiting [1981], is commonly (though inaccurately) referred to as the “Rapid Chloride Permeability Test” (RCPT). This name is inaccurate as it is not the permeability that is being measured but ionic movement. In addition, the movement of all ions, not just chloride ions, affects the test result (the total charge passed). There have been a number of criticisms of this technique, although this test has been adopted as a standard test, is widely used in the literature [Saito and Ishimori, 1995; Goodspeed et al., 1995; Thomas and Jones, 1996; Samaha and Hover, 1996] and has been used to limit permeability in at least one standard [CSA/S413-94]. The main criticisms are: (i) the current passed is related to all ions in the pore solution not just chloride ions, (ii) the measurements are made before steady-state migration is achieved, and (iii) the high voltage applied leads to an increase in temperature, especially for low quality concretes, which further increases the charge passed [Andrade, 1993; Zhang and Gjorv, 1991; Malek and Roy, 1996; Roy, 1989; Geiker, et al., 1990]. Lower quality concretes heat more as the temperature rise is related to the product of the current and the voltage. The lower the quality of concrete, the greater the current at a given voltage and thus the greater heat energy produced. This heating leads to a further increase in the charge passed, over what would be experienced if the temperature remained constant. Thus, poor quality concrete looks even worse than it would otherwise. These objections all lead to a loss of confidence in this technique for measuring chloride ion penetrability. In addition, they also lead to a loss of precision. The ASTM C1202 statement on precision, based upon work by Mobasher and Mitchell [1988], states that the single operator coefficient of variation of a single test has been found to 12.3 %, and thus two properly conducted tests should vary by no more than 35 % if done by one person. The between-laboratory measurement is naturally less precise and a single test result will have a coefficient of variation of 18.0 %. To minimize the variation, three samples are generally tested and the average value reported. However, a precision statement is also given for this type of test and it is stated that the average of three samples should not differ by more than 29% between two separate laboratories [ASTM C1202]. Another difficulty with the RCPT test is that it depends upon the conductivity of the concrete being in some way related to the chloride ion penetrability. Thus, any conducting material present in the concrete sample will bias the results, causing them to be too high. This would be the case if any reinforcing steel is present, if conductive fibers are used (e.g. carbon or steel), or if a highly ionic conductive pore solution is present [ASTM C1202]. This pore solution effect may be noticed if calcium

nitrite is included as a corrosion inhibiting admixture, and other admixtures may also have this effect [ASTM C1202]. Because these conductors all influence the results so that a higher coulomb value than would otherwise be recorded is determined, the method still could serve as a quality control test. It can qualify a mix, but not necessarily disqualify it [Ozyildirim, 1994]. If an acceptably low rating is achieved, it is known that the concrete is not worse than that, at least within the precision of the test method. Despite these drawbacks and limitations, attempts have been made to correlate RCPT values with diffusion coefficients from other tests [Thomas and Jones, 1996; Berke and Hicks, 1992].

2.3 Survey for Assessment

The research group prepared a survey form, and sent it to the different states to know the practices they are following for assessment. The sample survey results from Maryland and New York are shown in the Table 2 -10.

2.4 Summary

Table 2.1 shows all the tests and corresponding Standards. Tables 2.2 to 2.9 show the threshold values for the tests. Table 2.10 shows samples of the threshold values published by some States.

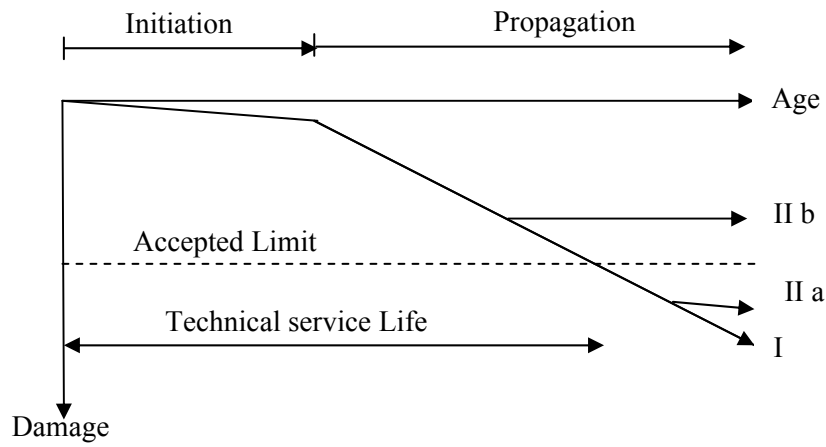


Figure 2-1: Schematical Representation of a Bridge Technical Service (Radomski, W., 2002)

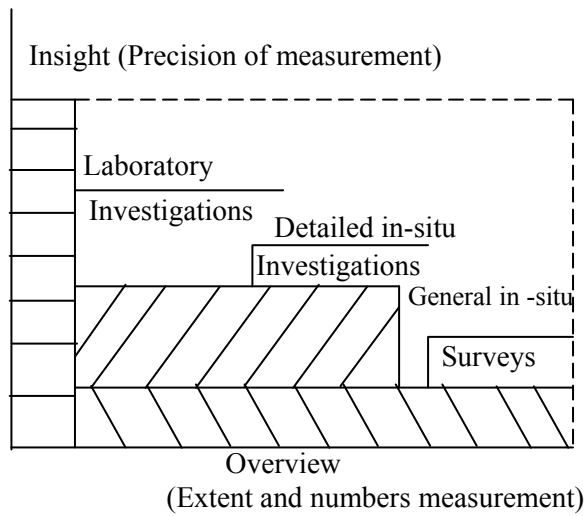


Figure 2-2: Information from different types of Investigation (Radomski, W., 2002)



Figure 2-3: Drilling the Core



Figure 2-4: Use of the Rebound Hammer

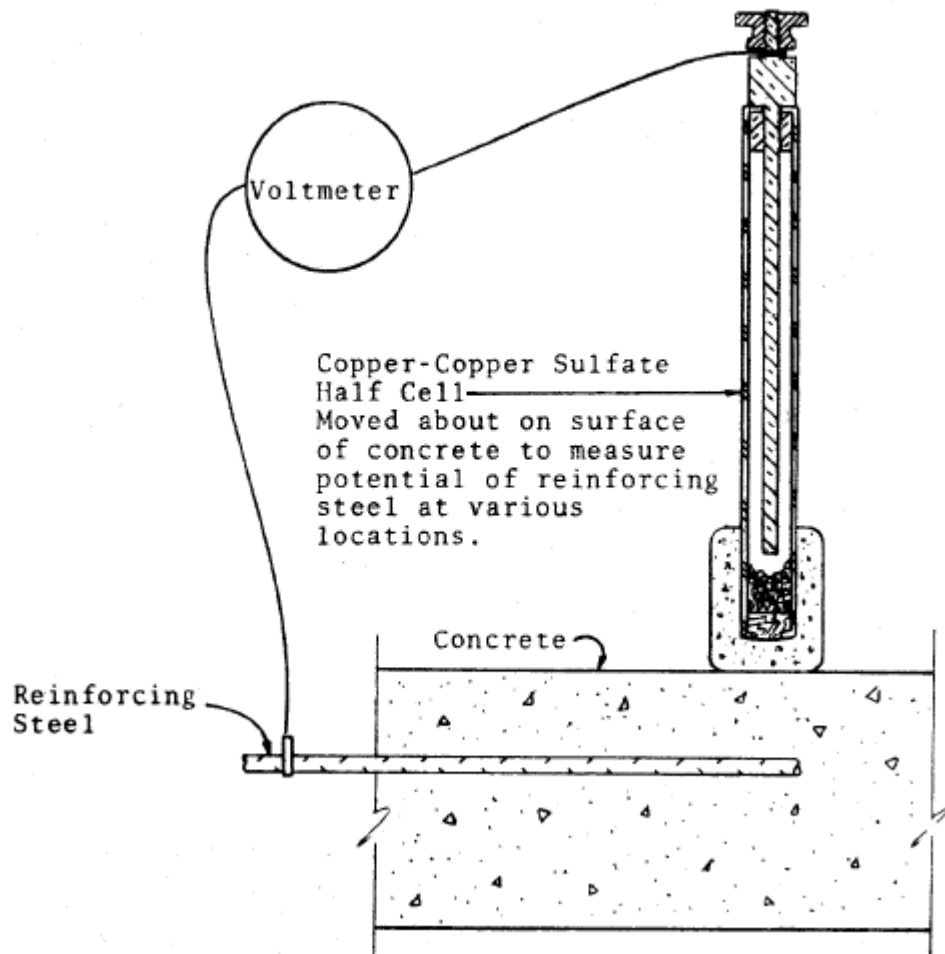


Figure 2-5: Diagram of a half-cell potentiometer (Tuttle, R.S., 2005)

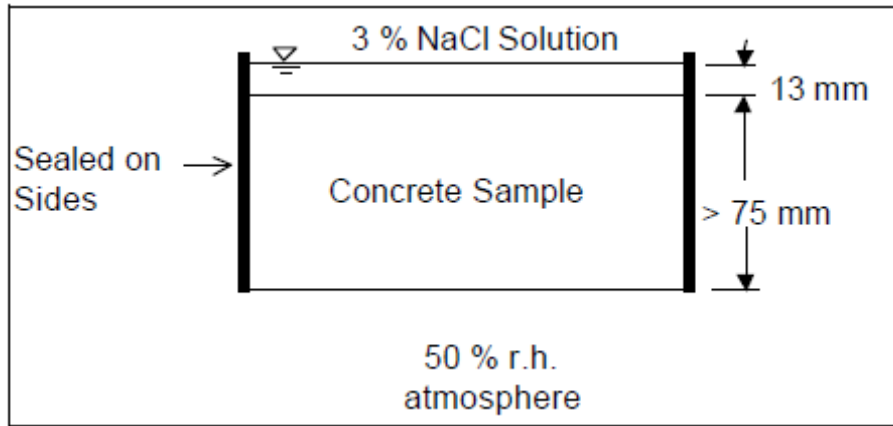


Figure 2-6: AASHTO T259 (salt ponding) test setup (Stanish, K.D., Hooton, R.D. and Thomas, M.D.A)

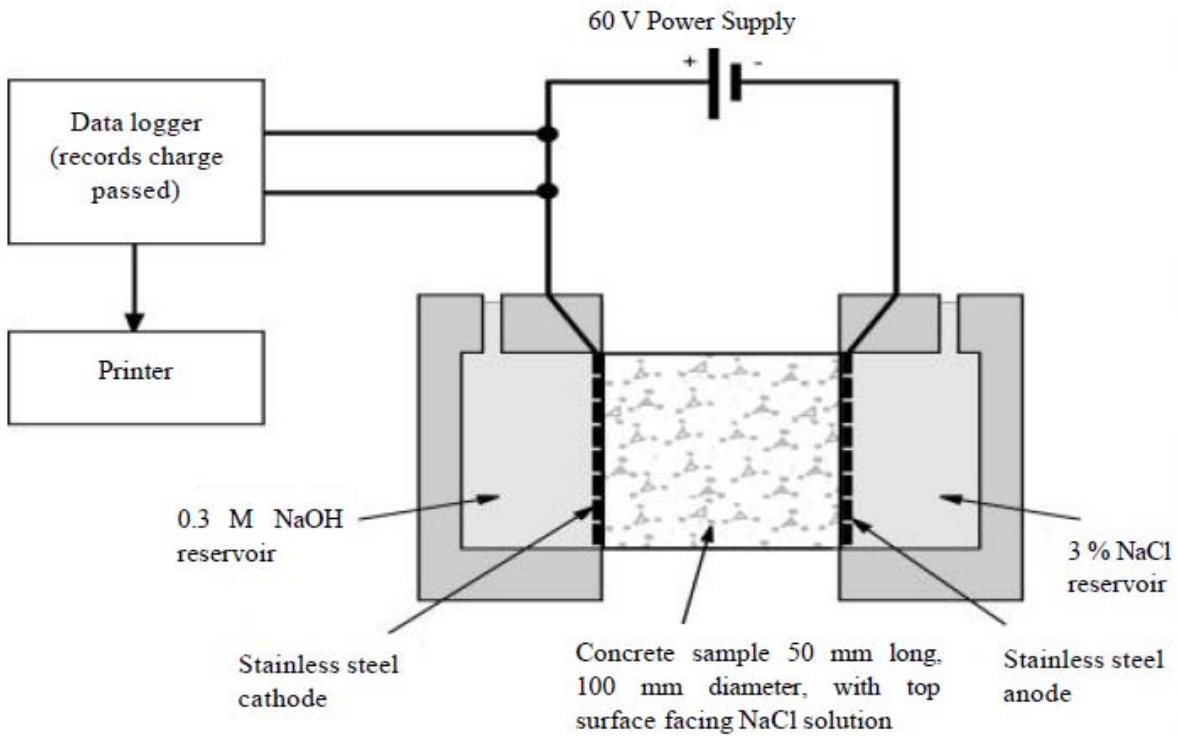


Figure 2-7: AASHTO T277 (ASTM C1202) test setup (Stanish, K. D., Hooton, R.D. and Thomas, M.D.A)

Table 2-1: Classifications of different kinds of Tests and corresponding Standard

Lab/In Situ Testing	Purpose	Method	Yes	No	D/ND/PD	ASTM	AASHTO	
Material Lab Testing:	a) Mechanical Properties	1) Compression Strength: Cylinder test						
		2) Tensile Strength: Split Test						
	b) Young Modulus	1) Cylindrical or prismatic						
	c) Creep							
	d) Shrinkage							
	e) Heat of Hydration							
	Durability properties							
	a) Water Absorption:	1) Capillary Absorption test						
		2) Immersion Test						
	b) Water permeability / Gas Permeability Test	1)Initial Surface Absorption Test				ND		
		2)Figg Air Test				PD		
	c) Chloride Diffusion /Concentration	1) Chloride Diffusion test					ASTM C 1202-07	AASHTO T 260-97
							ASTM C 1218-99	AASHTO T 277
							SHRP 2030	
		2) Immersion Method						
		3)Silver Nitrate Test				PD		
		4)Chemical Analysis applied to samples obtain with rotary hammer(Sampling drilling dust method)				PD		
	d) Chloride Migration Test							AASHTO T 277
	e) Gas Diffusion coefficient (Concrete Oxygen Diffusion)							
	In Situ Testing:	Evaluation of Concrete Strength	a) Drilling a Core				D	ASTM C 42/C42M-04
		b) Using the rebound hammer				ND	ASTM C805M-08	

		c) Ultrasonics			ND		
		c) Creep - Shrinkage Test done similar way as in laboratory					
	Evaluation of Concrete Uniformity	a) Ultrasonic Test (Use of ultrasonic measurement)			ND	ASTM E 494-05	
		b) Impact –Echo			ND	ASTM C1383-04	
		c) Radar Scan			ND		
		d) Thermography			ND		
	Evaluation of Reinforcement Cover & Position	a) Using covermeter			ND		
		b) Magnetic Field			ND		
		c) Radar Scan			ND		
		d) Radiography Scan			ND		
	Evaluation of in Situ Stresses	a) Dynamic Test					
		b) Anchorages Pulling					
		c) Stress release			PD		
	Durability Properties	a) Depth of Carbonation (Using Phenolphthalein)			PD	ASTM C 42-99	
						ASTM C 856-04	
		b) Corrosion Potential					
		1) Half-Cell Potential (Range)			PD	ASTM C 876-91	
		2) Constant anodic current polarization			PD		
		3) Polarization resistance probe			PD		
		c) Corrosion Rate: (Using Ultrasonic Method _Effective only Steel Bridge)—Relation Between Corrosion Rate and Corrosion Level.					
		d) Concrete Resistivity (Passing Electron)... Gives Information on Concrete quality (Range)			PD		
		e) Humidity in Concrete (Using Chemical Probe to measure in situ humidity)					
		f) Water Absorption					

		1) INSAT (Initial Surface Absorption Test)				
		2) Figg Method			PD	
		g) Air permeability				
		1) Figg method			PD	
	Post tensioning tendons corrosion or other destructive, including inadequacy of duct grouting	1) Exploratory hole drilling and no instrumented visual inspection			PD	
		2) Endoscope Inspection			PD	
		3) Vacuum pressure techniques			PD	
		4) Ultrasonics			ND	
		5) Radiography			ND	
		6) Impact echo			ND	
Load Testing:		a) The Reception Load Test				
		b) The Evaluation Load test				
		1) Static Load Test				
		2) Dynamic Load Test				

D= Destructive
ND=Non Destructive
PD=Partial Destructive

Table 2-2: Relationship Between Corrosion Potential and resistivity (Branco,F.A and Brito,J.D.,2003)

Corrosion Level	Resistivity (kΩ-cm)
Very High	<5
High	5-10
Moderate	10-20
Low	>20

Table 2-3: Interpretation of AASHTO T 277 results (Branco, F.A and Brito, J.D., 2003)

Chloride Penetration	Coulombs
High	>4000
Average	2,000-4,000
Low	1,000-2,000
Very Low	100-1,000
Negligible	<100

Table 2-4: Relationship between half-cell measurements and active corrosion probability (Branco, F.A and Brito, J.D., 2003)

Half Cell (mV)	Chance of Corrosion
<-350	90%
-250 to -350	50%
>-200	10%

Table 2-5: Relationship between Corrosion rate and corrosion level (Branco, F.A and Brito, J.D., 2003)

Corrosion Level	Ic (mA/cm ²)
High	>1
Average	0.5-1
Low	0.1-0.5
Negligible	<0.1

Table 2-6: Characteristics of concrete oxygen diffusion (Branco,F.A and Brito,J.D.,2003)

Concrete quality	Do(x 10E-8 m ² /s)
High	<0.5
Average	0.5-5
Low	>5

Table 2-7: Concrete Classification for Immersion Test (Branco, F.A and Brito, J.D., 2 003)

Concrete quality	A(%)
High	<3.0
Average	3.0-4.0
Low	>4.0

Table 2-8: Concrete classification for water permeability (Branco, F.A and Brito, J.D., 2003)

Concrete quality	Permeability	k(m/s)
High	Low	<10E-12
Average	Medium	10E-12 to 10E-10
Low	High	>10E-10

Table 2-9: Concrete classification for capillary absorption test (Branco,F.A and Brito,J.D.,2003)

Concrete quality	a(mm/min ^{.5})	Water height (mm)
High	<0.1	<10
Average	0.1-0.2	10-20
Low	>0.2	>20

Table 2-10: Sample Threshold values by States

Maryland	
Test	Acceptance limit
AASHTO T277	2000 coulombs
Half-Cell Potential	No-0-0.29 V, Minor 0.30-0.44 V Major 0.45 or more
Chloride Diffusion Test	2.0 #/cy
New york	
Tests	Acceptance limit
Chloride Diffusion Test	1.3 #/cy
Depth of Carbonation Test	Pachometer – 2” cover
Half-Cell Potential	0.35 V limit

Chapter 3

Model for Reinforcement Corrosion in Structures Exposed to Chlorides in Spreadsheet

3.1 Introduction

Numerous models have been proposed for predicting ingress into concrete. These vary in levels of complexity from very simple analytical models assuming uniaxial diffusion into a homogeneous concrete, to much more sophisticated numerical models which take account of depth and time dependent changes in concrete properties, chloride binding and leaching. As mentioned earlier capillary absorption, hydrostatic pressure, and diffusion are the means by which chloride ions can penetrate concrete. The most familiar method is diffusion, the movement of chloride ions under a concentration gradient. For this to occur the concrete must have a continuous liquid phase and there must be a chloride ion concentration gradient.

A second mechanism for chloride ingress is permeation, driven by pressure gradients. If there is an applied hydraulic head on one face of the concrete and chlorides are present, they may permeate into the concrete. A situation where a hydraulic head is maintained on a highway structure is rare, however. A more common transport method is absorption. As a concrete surface is exposed to the environment, it will undergo wetting and drying cycles. When water (possibly containing chlorides) encounters a dry surface, it will be drawn into the pore structure through capillary suction. Absorption is driven by moisture gradients. Typically, the depth of drying is small, however, and this transport mechanism will not, by itself, bring chlorides to the level of the reinforcing steel unless the concrete is of extremely poor quality and the reinforcing steel is shallow. It does serve to quickly bring chlorides to some depth in the concrete and reduce the distance that they must diffuse to reach the rebar [Thomas, et al., 1995]. Of the three transport mechanisms described above that can bring chlorides into the concrete to the level of the rebar, the principal method is that of diffusion. It is rare for a significant hydraulic head to be exerted on the structure, and the effect of absorption is typically limited to a shallow cover region. In the bulk of the concrete, the pores remain saturated and chloride ion movement is controlled by concentration gradients. In the longer term the movement of chlorides is determined by diffusion in the sub-surface zone which is less affected by changes in surface conditions and maintains approximately uniform and constant moisture content. The proposed model therefore assumes that the diffusion coefficient is time dependent, with high early life values representing the absorption component of chloride ingress and the lower late life values representing the diffusion controlled part of the process at greater depth (Bamforth, P.B.,

1993). Substantial data (several hundred results) have been obtained from the literature, representing structures world-wide and these have been used to derive values for apparent diffusion coefficients and their relationship with time of exposure and with water-binder ratio for a range of concrete mix types including Portland cement, pc, and blends of pc with pulverized fuel ash (pfa, also called fly ash), ground granulated blastfurance slag (ggbs), and micro-silica,ms (silica fume). This model is described and predictions are compared with P.B. Bamforth’s model which was compared with field observations.

3.2 Modeling using a time dependent chloride diffusion coefficient

While it is acknowledged that chloride ingress involves a complex interaction of mechanisms, it is commonly assumed that it can be approximated to a diffusion process. This is because, in many conditions, the shape of the observed chloride profile can be fitted using diffusion theory. The most common approach is to apply the error function solution to Fick’s second law of diffusion to derive values of an “apparent diffusion coefficient”. Fick’s First Law, which, in the one-dimensional situation normally considered, states:

$$J = -D_{eff} \frac{dC}{dx} \quad (3.1)$$

where J is the flux of chloride ions, D_{eff} is the effective diffusion coefficient (see below), C is the concentration of chloride ions and x is a position variable. In practical terms, this equation is only useful after steady-state conditions have been reached, i.e. there is no change in concentration with time. It can be used, however, to derive the relevant equation for non-steady conditions (when concentrations are changing), often referred to as Fick’s Second Law:

$$\frac{\partial C}{\partial t} = D_{eff} \frac{\partial^2 C}{\partial x^2} \quad (3.2)$$

which includes the effect of changing concentration with time (t). This has been solved using the boundary condition $C_{(x=0, t>0)} = C_0$ (the surface concentration is constant at C_0), the initial condition $C_{(x>0, t=0)} = 0$ (the initial concentration in the concrete is 0) and the infinite point condition $C_{(x=\infty, t>0)} = 0$ (far enough away from the surface, the concentration will always be 0).

The error function equation is –

$$C_x = C_{sn} \left(1 - erf \frac{x}{2\sqrt{D_{ca}t}}\right) \quad (3.3)$$

Where,

$D_{ca^{tm}}$ = Apparent diffusion coefficient (m^2/s) at time $t(s)$

t = Expected Time Period

t_m (sec) = 20 years -as the graph is plotted with data of 20 yrs

n = The age factor

x = Depth (mm)

C_{sn} (%) = Notional surface level of chloride (% wt. of Con.)

C_x = Chloride Content(%) at depth, $X(m)$, at time, $t(s)$

erf is the error function

For concrete, there are some factors that interfere with simple interpretation of diffusion data. First of all, the chloride ions are not diffusing through a homogeneous solution. Concrete is a porous matrix that has both solid and liquid components. The diffusion through the solid portion of the matrix is negligible when compared to the rate of diffusion through the pore structure. The rate of diffusion is thus controlled not only by the diffusion coefficient through the pore solution but by the physical characteristics of the capillary pore structure. This effect is normally considered implicitly, however, and the effective diffusion coefficient of the chlorides into the concrete as a whole is considered, called here D_{eff} . Other influences are discussed later.

The form of relationship between D_{ca} and period of exposure was developed by P. B. Bamforth for different concrete mix types. It can be expressed by an equation of the form.

$$D_{ca} = D_{ca(tm)} \left(\frac{t}{t_m} \right)^n \quad (3.4)$$

$D_{ca(tm)}$ is the value of the apparent diffusion coefficient derived at time t_m . Combining equations (3.3) and (3.4) leads to the following expression for predicting chloride levels based on a time dependent apparent diffusion coefficient –

$$C_x = C_{sn} \left(1 - \operatorname{erf} \frac{x}{\sqrt[2]{D_{ca(tm)} \left(\frac{t}{t_m} \right)^n t}} \right) \quad (3.5)$$

To predict the rate of chloride ingress and the time to the onset of corrosion the following values must be quantified:

- ✓ The notional surface chloride level, C_{sn}
- ✓ The effective chloride diffusion coefficient, $D_{ce(t_m)}$ at time, t_m
- ✓ The age factor, n
- ✓ The chloride threshold level, C_t

3.3 Properties of the Concrete that Affect the Chloride Penetration Rate

The rate of ingress of chlorides into concrete depends on the pore structure of the concrete, which is affected by factors including materials, construction practices, and age. The penetrability of concrete is obviously related to the pore structure of the cement paste matrix. This will be influenced by the water-cement ratio of the concrete, the inclusion of supplementary cementing materials which serve to subdivide the pore structure [McGrath, 1996], and the degree of hydration of the concrete. The older the concrete, the greater amount of hydration that has occurred and thus the more highly developed will be the pore structure. This is especially true for concrete containing slower reacting supplementary cementing materials such as fly ash that require a longer time to hydrate [Tang and Nilsson, 1992; Bamforth, 1995]. Another influence on the pore structure is the temperature that is experienced at the time of casting. High-temperature curing accelerates the curing process so that at young concrete ages, a high temperature cured concrete will be more mature and thus have a better resistance to chloride ion penetration than a normally-cured, otherwise identical, concrete at the same age. However, at later ages when the normally-cured concrete has a chance to hydrate more fully, it will have a lower chloride ion diffusion coefficient than the high-temperature-cured concrete [Detwiler, et al., 1991; Cao and Detwiler, 1996]. This finding has been attributed to the coarse initial structure that is developed in the high-temperature-cured concrete due to its initial rapid rate of hydration as well as the possible development of internal microcracking. The rate of chloride penetration into concrete is affected by the chloride binding capacity of the concrete. Concrete is not inert relative to the chlorides in the pore solution. A portion of the chloride ions reacts with the concrete matrix becoming either chemically or physically bound, and this binding reduces the rate of diffusion. However, if the diffusion coefficient is measured after steady-state conditions have been reached, then all the binding can be presumed to have taken place and this effect will not then be observed. If a steady state condition has not been reached, then not all the binding will have occurred and this will affect the results. The chloride binding capacity is controlled by the cementing materials used in the concrete. The inclusion of supplementary cementing materials affects binding, though the exact influence is unclear [Byfors, 1986; Rasheeduzafar, et al., 1992; Sandberg and Larsson, 1993; Thomas, et al., 1995]. Also, the C_3A content of the cement influences its binding capacity, with

increased C₃A content leading to increased binding [Holden, et al., 1983; Midgely and Illston, 1984; Hansson and Sorenson, 1990].

3.4 Input Parameters

3.4.1 Surface Chloride Levels

The surface chloride level is determined primarily by the proximity to the source of the chlorides. For example, surface chloride levels tend to reduce with increased height above sea level and with distance from the coast. It is also a function of the type of concrete used and tends to be higher for mixes which have a high chloride binding capacity and a high resistance to chloride penetration. For such concretes the chlorides captured on the surface find it more difficult to escape, for example, under wash-down conditions. A comprehensive review of surface chloride levels has been carried out by P. B. Bamforth (Bamforth, P.B., 1996) and values have been recommended for predictive and design purposes. These are given in Table 3.1. The designer can select which values to use in combination with assumption about the other input parameters and the acceptable level of risk.

3.4.2 Apparent Diffusion Coefficients

Two of the principal factors influencing the apparent chloride diffusion coefficient, D_{ca} , are the chemistry of the cementing materials (binder) and the water-binder ratio. The chemistry influences not only the initial value of D_{ca} , but also how it varies with age. The relationship defining the change in D_{ca} with time is given as Equation 3.4. The age factors, n , have been derived from a comprehensive analysis of published data for a range of mix types which has been widely published (Bamforth, P.B., 1996) and proposed design values are in Table 3.2

Relationship between D_{ca} and water-binder ratio are shown in Figure 3-1. The results have been normalized using the above age factors to represent values expected after 20 years of exposure. In the spreadsheet, equations defining the curves in Fig 3-1 are used to derive 20 year values for D_{ca} for the

different mix types. Having defined one point on the time axis, the age factor is then used to generate the full curve.

3.4.3 Chloride Threshold Levels

While most data on chloride ingress is presented by weight of concrete, the threshold level for chloride activation is most commonly presented as percent weight of cementitious material. To convert predict values of chloride level from percent weight of concrete to percent weight of binder it is necessary to apply a multiplier which is the ratio of concrete density to binder content. If there are no details of the mix, a default value of 6.7 is proposed. This assumes a density of 2350 kg/m^3 and a cementitious content of 350 kg/m^3 . In general, structures to be exposed to resist chloride ingress will have a cement content greater than 350 kg/m^3 and the default value is, therefore, likely to be conservative.

While considerable research has been carried out in an attempt to define threshold levels, it has become increasingly clear that there is no single value which represents the wide range of concentrating materials and exposure conditions. However, in a comprehensive review of published data Glass (Glass, G K., and Beunfeld, N.R., 1995) concluded that- “At present, the chloride threshold level is best considered in terms of corrosion risk”. This approach was suggested earlier by Browne (Browne, R.D., 1982) who proposed the risk classification Table 3.3.

These recommendations (listed in Table 3.3) are broadly consistent with data from UK bridges (Nilsson, L-O., Poulsen, E., Sandberg, P., Sorensen, H.E., Klinghoffer, o., 1996). Over 450 results were obtained which demonstrated that below 0.2% Cl (% wt of cement) the risk was very low. Above this level the proportion of corroding steel associated with each level of chloride increased in a way which was consistent with a normal distribution of threshold levels.

Many factors appear to influence the threshold level. Some of these are environmental. For example Sandberg et al (1995) proposed that threshold levels are lower under conditions of wet/dry cycling. The mix proportions may also be influential with lower threshold levels being achieved in mixes with high w/c ratios. The commonly used value of 0.4% (wt of cement) appears to be most applicable in conditions of wet/dry cycling of high (>0.6) w/c ratio mixes (Sandberg, P., 1995) while in mixes of low w/c ratios, under more stable moisture conditions, higher values may be tolerated. Blended cements tend to exhibit threshold levels which are similar to, or lower than, that of PC concrete and corrosion inhibitors will increase the tolerance to chlorides. To determine the time to corrosion activation based on achieving a chloride threshold level, the appropriate threshold level must be defined. The Chloride content (% wt of cement) and the risk of corrosion is provided in Table 3.3

3.4.4 Corrosion Rate

There is a relationship between chloride content and corrosion rate (Bamforth, P.B and Chapman-Andrews, J., 1994) in Fig 3.2. Within the constraints of the study the corrosion rate, CR, appeared to be independent of the cover depth and can be expressed by an equation of the form-

$$CR = a.e^{b.C_x} \quad (3.6)$$

The values of “a” and “b” are given in the Table 3-4. Assuming that corrosion is negligible for values of CR < 1.2 microns/year, the best fit exponential curve indicates a threshold value of 0.46% Cl (wt of cement). This value is consistent with average values derived from the relationship between chloride and half cell potential measurements using the same data (Bamforth, P.B and Chapman-Andrews, J., 1994) and close to the results of gravimetric test by Thomas (1996).

The rate of corrosion is influenced by many factors in addition to the chloride content by weight of cement. In particular, the general climatic conditions (i.e., temperature, humidity, time of wetness) and the microclimate (proximity to moisture, orientation) will influence the internal moisture state of the concrete and the temperature will influence both the rate of chloride migration and the threshold level. The spreadsheet calculates the values of CR based on the average value of C_x within each time increment and multiplies this value of CR by the time period within the increment.

3.5 The Chloride Diffusion Spreadsheet Model

3.5.1 Description

The chloride diffusion spreadsheet model first predicts the level of chloride at different cover depths and time increments using Equation 3.5 . A second stage predicts the rate of corrosion within each defined time increment based on the predicted chloride content at the cover depth and increments of corrosion are then summated to determine the total corrosion with time. Predictions are based on limited knowledge of the concrete and the environment, but requires the following input-

- ✓ Cement type and content
- ✓ Water-binder ratio
- ✓ Surface chloride level

The values of apparent diffusion coefficient, D_{ca} , and the age factor, n , can be computed within the model using background data. Table A-1 in the spreadsheet (See Appendix) generates a series of chloride profiles at defined time at different depths. Table A-2 in the spreadsheet (See Appendix) uses condition statements to identify in which depth and time increments a defined threshold level is exceeded and then uses a linear interpolation within the selected increment to calculate the predicted time to activation. This is shown in the final column of the second table. Further tables are used for the corrosion rate within each depth and time increment and the cumulative corrosion. Condition statements are again used to calculate the increments within a defined corrosion rate or amount of corrosion exceeded. Hence the service life can be predicted in relation to either a chloride threshold level, or a corrosion rate, or an amount of corrosion.

3.5.2 Validation of the Model

The model is checked with the P.M Bamforth's model, and the result from this model coincides with P.M Bamforth's model result which is related with Thomas's result of measurement (1996)

3.5.3 Limitations and Developments

The spreadsheet is currently limited in many respects and needs further development. For example, it was developed for temperature climates and has no facility for investigating the effects of temperature. This can be addressed however, by adjusting the effective diffusion coefficient, the threshold level are effected by many other parameters, including the cement type, the water-cement ratio, the cover depth and the state of the steel/ concrete interface. A sub-routine to deal with these factors must be developed. In addition the corrosion required to cause cracking will be influenced by cover depth, bar size and spacing, and the porosity of the concrete and a separate routine must be introduced to take into account these factors. Nevertheless, the model is very quick to use and yields results which are, in general consistent with the observed performance of structure.

3. 6 Summary

A chloride diffusion spreadsheet model has been developed to predict both the rate of chloride ingress into concrete and subsequent corrosion rates of the reinforcement. The model is based on the use of time

dependent chlorides diffusion coefficients, which themselves are derived from empirical relationships with water-binder ratio for different binder types. The rate of corrosion is calculated using predetermined relationships with chloride content for different exposure conditions. Where data was available against which to compare predictions, the outputs from the model were within the normal scatter of results, indicating that, despite the many simplifying assumptions, the model reflects field behaviour.

Further work is needed to broaden the area of application and to take account of the distribution of steel when predicting the time to cracking. In addition, ways of incorporating into the model the influence of various measures for enhancing durability, such as coatings and surface treatments applied to the concrete or the steel are to be investigated.

Table 3-1: Recommended surface chloride levels (% weight of concrete) for use in design (Bamforth, P.B, 1996)

Mix Type	Typical	Upper 90%	Upper 95%
pc concrete	0.36%	0.70%	0.79%
blended cement mixes(pfa,ggbs)	0.51%	0.85%	0.94%

Table 3-2: The age factor, n (Bamforth, P.B, 1996)

Portland Cement Concretes	-0.264
pfa concretes	-0.699
ggbs concretes	-0.621

Table 3-3: Chloride threshold level (Bamforth, P.B., 1998)

Chloride (% wt of cement)	Risk of Corrosion
< 0.4	Negligible
0.4 to 1.0	Possible
1.0 to 2.0	Probable
>2.0	Certain

Table 3-4: Value of "a" and "b" according to Exposure Condition

Exponential relationship between corrosion rate CR and chloride content Cx, $CR=a*e^{(b*Cx)}$		
Exposure Condition	a	b
Wet,rarely dry	0.84	0.64
Splash(cyclic wet/dry) or airborne seawater	0.53	1.56
Tidal	0.46	1.84

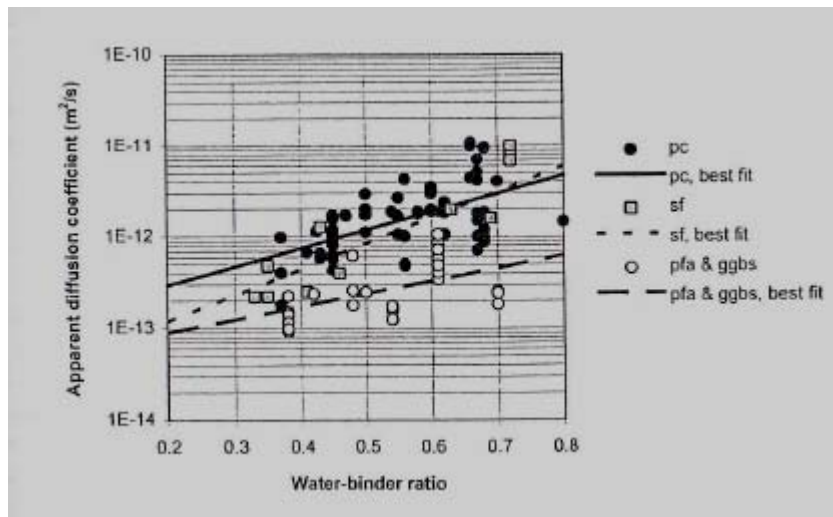


Figure 3-1: The relationship between D_{ca} (normalized for 20 years of exposure) and water-binder ratio (Bamforth, P.B, 1996)

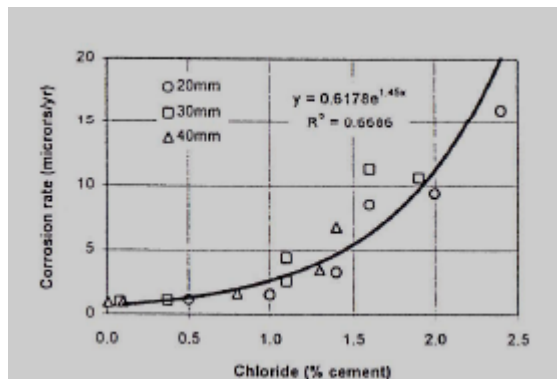


Figure 3-2: The influence of Chloride content on the corrosion rate (Bamforth, P.B. and Chapman-Andrews, J., 1994)

Chapter 4

Model Comparison

4.1 Introduction

The chloride diffusion spreadsheet model is compared with the model Life-365 (Life-365). Both models are based on the diffusion mechanism, Fick's law. Both models predict the chloride concentration and determine the time to start corrosion. Both have some limitations and advantages also.

4.2 Input Comparison

- a) For age factor n , the chloride diffusion spreadsheet model used the formula $D_{ca} * (t/t_m)^n$, so The chloride diffusion spreadsheet model is using the value -0.264 values etc. whereas the Life-365 is using the formula $D_{ca} * (t_m/t)^n$, so Life-365 is using values +0.264 etc as inputs.
- b) Regarding apparent diffusion coefficient, the chloride diffusion spreadsheet model used the coefficient for 20 years as default value, but Life-365 is using the coefficient for 28 days. The Life-365 considers the diffusion coefficient constant after 25 years, but chloride diffusion spreadsheet model does not.
- c) Regarding Cl threshold level, the Life-365 uses the threshold value 0.05% (by mass of concrete), whereas the chloride diffusion spreadsheet model's threshold value is 0.4% (by mass of cement).
- d) Life-365 uses some default values as a factor of safety in case of different cases, like rural highway bridges, urban highway bridges, bridges near sea, etc. But the chloride diffusion spreadsheet model's factor of safety is user defined according to the Table 3.1. So, the chloride diffusion spreadsheet model may show some different values than Life-365 does.
- e) In Life-365 temperature variation is considered, whereas the chloride diffusion spreadsheet model does not include any temperature consideration. So, the chloride diffusion spreadsheet model can get some different values than life 365 for this.

4.3 Result Comparison

Figures 4.1 through 4.4 show the result comparison between Life-365 and Chloride diffusion spreadsheet model. Both results are very close. For comparison the unit of the chloride diffusion spreadsheet model's output is changed to % of concrete.

Advantages

The chloride diffusion spreadsheet model has some advantages over Life-365. For example,

- a) The model can predict the corrosion rate for years.
- b) The user can use his/her own calculated factor of safety as required
- c) The model can be used for three different units of threshold values: (i) % of cement, (ii) % of concrete, (iii) corrosion rate in Microns/ Year.

Limitations

The chloride diffusion spreadsheet model has some limitations.

- a) The present model is independent of temperature variation.
- b) The chloride diffusion spreadsheet model cannot perform cost analysis with time
- c) The chloride diffusion spreadsheet model should include more environmental conditions.

4.4 Summary

Though the present chloride diffusion spreadsheet model has some limitations, but its output result is very close to Life-365. More modification of the model will be done in future.

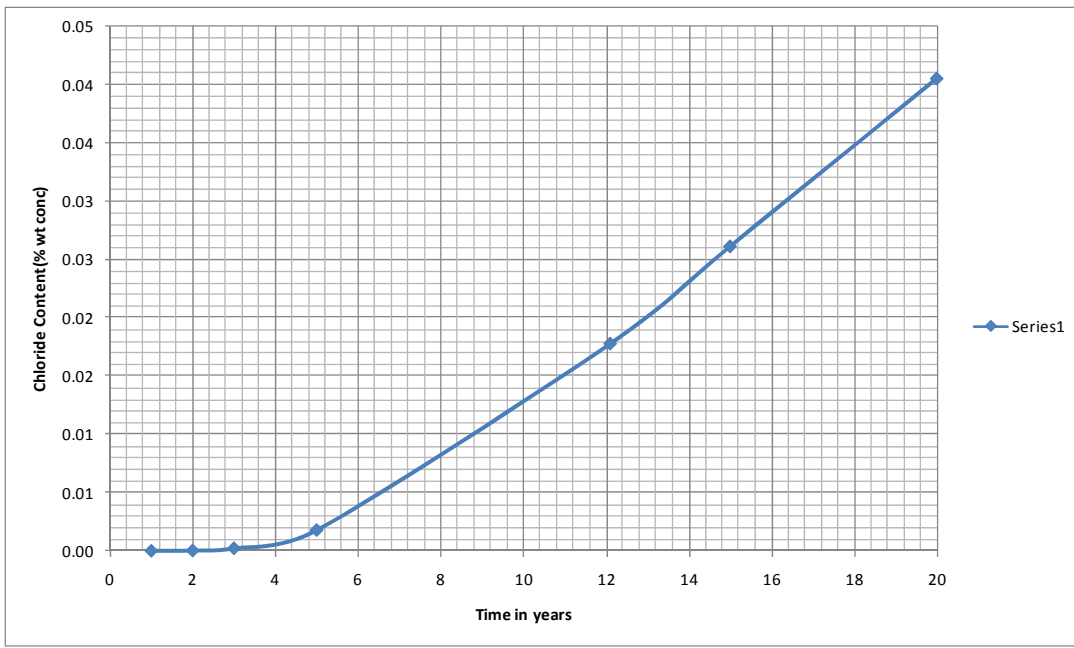


Figure 4-1: At 60 mm depth, The predicted Chloride Content (% of concrete) Vs years. (Chloride diffusion spreadsheet Model)

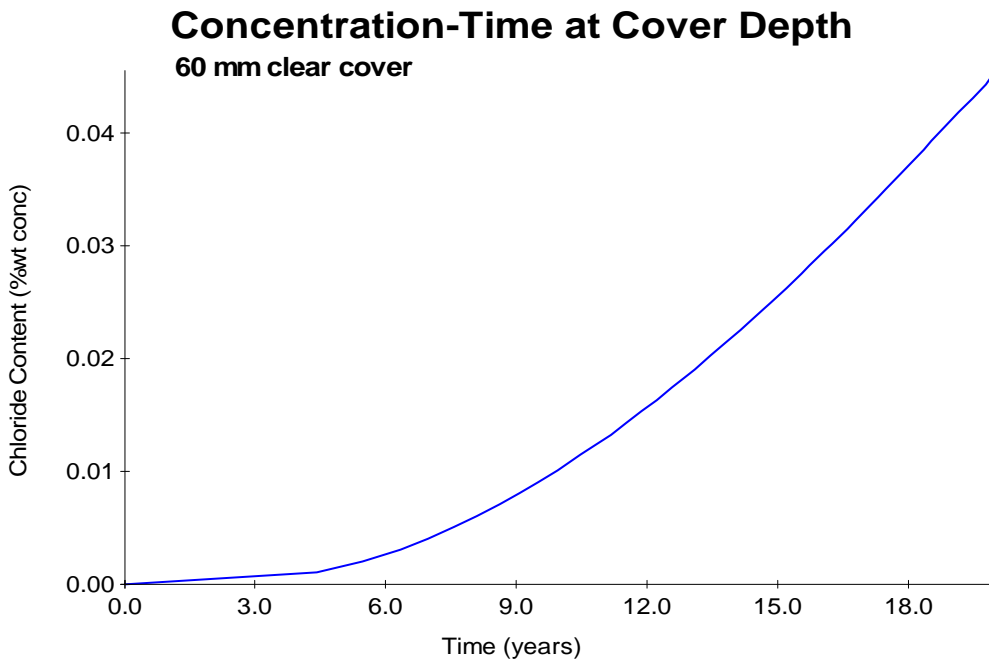


Figure 4-2: At 60 mm depth, The predicted Chloride Content (% of concrete) Vs years. (Life-365 Model)

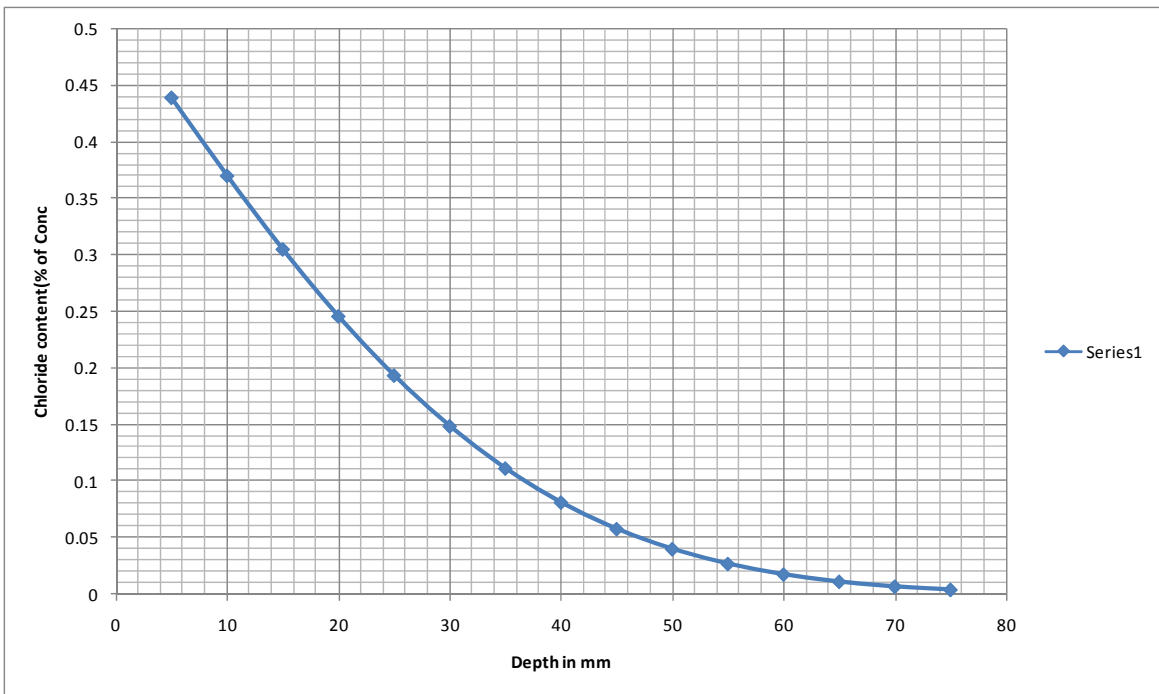


Figure 4-3: At 12.1 year, The predicted Chloride Content (% of concrete) Vs Depth (Chloride diffusion spreadsheet Model)

Concentration-Depth

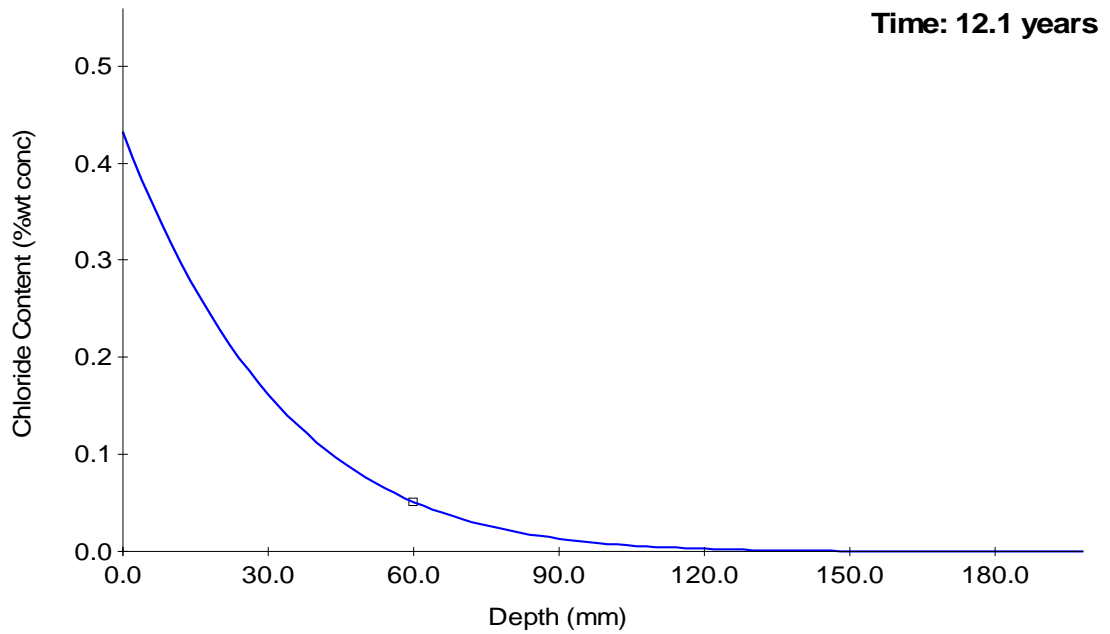


Figure 4-4: At 12.1 year, The predicted Chloride Content (% of concrete) Vs Depth (Life-365 Model)

Chapter 5

Correlation Between the Chloride Diffusion Spreadsheet Model and the State Practice Methods

5.1 Application

Based on the chloride diffusion test, Maryland sets the threshold value of the chloride at 2 lb/yd³ which is equivalent to 0.35 (% wt of cement). The calculation is below.

$$1 \text{ lb/ yd}^3 = 0.59 \text{ kg/ m}^3$$

$$2 \text{ lb/ yd}^3 \Rightarrow 2 * 0.59 = 1.18 \text{ kg/ m}^3$$

According to P.B.Bamforth (1998), the structure of cement content greater than 350 kg/ m³ can resist chloride ingress. The commonly used value of 0.35 cl (%wt. of cement) appears to be most applicable. So, $0.35 * 1 / 100 * 350 = 1.22 \text{ kg / m}^3$ which is close to 1.18 kg/m³. Table A-3 and Table A-4 from the chloride diffusion spreadsheet model will provide the time in years when the corrosion starts based on depth. For the following data, the users can determine when they need to take action to avoid corrosion for further damage.

Cement content = 400 (kg/m³) (24.97 lb/ft³)

Density = 2400 (kg/m³) (149.83lb/ft³)

W/C ratio = 0.45

Cl threshold level = 0.40 (% wt of cement)

Age Factor, n = -0.264 (Portland cement concrete) (Table 3-2)

C_{sn}(% concrete) = 0.7 (Portland cement concrete considering upper 90%) (Table 3-1)

Estimated D_{ce} = 9.41E-13 (at 20 years) (Figure 3-1)

Splash(cyclic wet/dry) or air bone seawater a = 0.53, b=1.56 (Table 3-4)

Table 5-1 and Figure 5-1 are the final results of the above inputs. The user can take the decision considering Table 5-1 & Figure 5-1. For example, at 30 mm (1.18 inch) depth the corrosion will start after 4 years, accordingly at 40 mm (1.57 inch) after 8 years, at 55 mm (2.16 inch) after 12 years and at 60 mm (2.36 inch) after 23 years. So, at present the user can determine when it is needed to remove the surface concrete of 1 inch depth or 1.5 inch depth or 2 inch depth, and the need to be filled with new concrete to stop further ingress of chloride and protect rebar to avoid huge damage. This decision can save not only property loss but also human lives. The user will get the complete profile of the chloride ingress and corrosion rate in future by using this model.

5.2 How to Use Chloride Diffusion Spreadsheet

To use spreadsheet is very easy and there is no need for lots of inputs. According to the inputs there will be generated Table A-1, Table A-2, Table A-3, Table A-4, Figure A-1, Figure A-2 and Figure A-3. (Appendix A.1)

- a) Binding materials: user needs to select the binding materials types from Portland Cement (pc), pulverized fuel ash (pfa), ground granulated blastfurance slag (ggbs), silica fume (sf).
- b) Need to provide cement content (kg/m^3), density(kg/m^3), water cement ratio and threshold value
- c) For age factor, n: select “n” value considering binding materials types.
- d) Regarding surface chloride $.C_{sn}$: The value is % of concrete. The designer can select which values to use in combination with assumptions about the other input parameters and the acceptable level of risk. The user can follow Table 3-1 (Bamforth, P.B, 1996)
- e) Diffusion coefficient, D_{ce} : This value is based on the 20 years data. The user can use Table 3-2 and Figure 3-1 according to the binding materials.
- f) To determine corrosion rate(CR), the user needs to give the value of “a” and “b” from Table 3-4 according to the exposure condition like “ wet/rarely dry”, “ Splash(cyclic wet/dry) or airborne seawater”, “Tidal”.

5.3 Correlation of Recommendations by the Spreadsheet Model and Status Report

The research team has collected some documents of the bridge conditions, specifically Maryland Bridges with Substructures Replaced or Rehabilitated due to Material Problems. There are some examples noted in Table 5-2.

Consideration all the recommendations the research team has summarized six types.

- Full replacement
- Partial replacement
- Rehabilitation for 5", 3" & 2"
- Partial removal/ Spot Rehab
- Minor Repair (Cosmetic)
- No Action

5.4 Summary & Conclusion

From the results of the program the user can conclude that after 12 years 2" of concrete can be removed from the surface and be filled with new concrete. It will stop further penetration of chloride ion; hence corrosion of rebar could be avoided.

For 2", Chloride contents reduced drastically compared to the content on the surface. For more cases or more aged weathered substructures, the chloride penetrates deeper and removal of 3" of concrete may be needed. If the concrete behind rebars shows the sign of chloride penetration, 5" of concrete removal is needed.

Lots of factors depend on repair and replacement schemes. The ways below can be considered whether to be repaired or replaced.

- Structural conditions should be determined, and then the need to set goals.
- The factors involved should be clarified.
- The cost of the repairs and replacement play a vital role.
- The benefits of repair and replacement should be evaluated according to the goals and requirements of all the factors.
- Select the ways of a cost effective scheme in order to perform a further detailed study.

Rehabilitation Systems: Rehabilitation procedures can be classified into the following groups.

- Increment of load carrying capacity, particularly live load.
- Improvement of geometrics
- Correction of mechanical deficiencies
- Others: Concrete, safety, drainage

Replacement Systems: The following issues that need to be taken care of are –

- The cost and the available funding
- Level of service
- Environmental conditions
- Anticipated use
- Delays for permit demands.

The user can see Appendix A.1 for more information regarding the use and result of the model.

Table 5-1: Surface depth vs time to activate corrosion according to the threshold value.

Time (years)	0	0	1	1	2	4	7	8	12	12	15	23	23	30	39
Depth (mm)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75
Depth (inch)	0.19	0.39	0.59	0.78	0.98	1.18	1.37	1.57	1.77	1.96	2.16	2.36	2.56	2.7	3

Table 5-2: Examples of Recommendations (Substructures)

Bridge no.	Recommendation
0021064	Rehabilitation for the piers and abutments of this structure. Remove concrete to a minimum depth of 2 inches or to sound concrete.
0310900	Rehabilitation-Remove concrete to a minimum depth of 5 inches or to sound concrete.
3153	Rehabilitation for the piers and abutments of this structure. Abutments-Minimum depth of 5 inches. Piers-Minimum depth of 3 inches.
3155	Complete removal and replacement for the piers and rehabilitation for the abutments.
0001029	Complete removal & replacement bent 2 column 3 , bent 6 column & arch, Bent 7 all columns and shortest girder, bent 9 & 10 all columns. Remaining bents and piers cosmetic repair.
0001030	No treatment on pier caps.
0001044	Suitable candidate for rehabilitation and need not be replaced.
0001087	WBR-full removal both pier caps, columns and bases; abut sound; EBR-full removal pier 1; full removal northern half pier cap & northern column, pier 2 & northern half east abut wall. Remaining suitable candidates for rehabilitation, not replacement.
0001092	Rehabilitation- for the piers and abutments. Remove the concrete to a minimum depth of 3 inches or to sound concrete.
0001097	Rehabilitation for the piers and abutments. Remove concrete to a minimum depth 3 inches or to sound concrete. Replace with a high density concrete to prevent moisture penetration. The pedestals for the piers and abutments should be repaired.

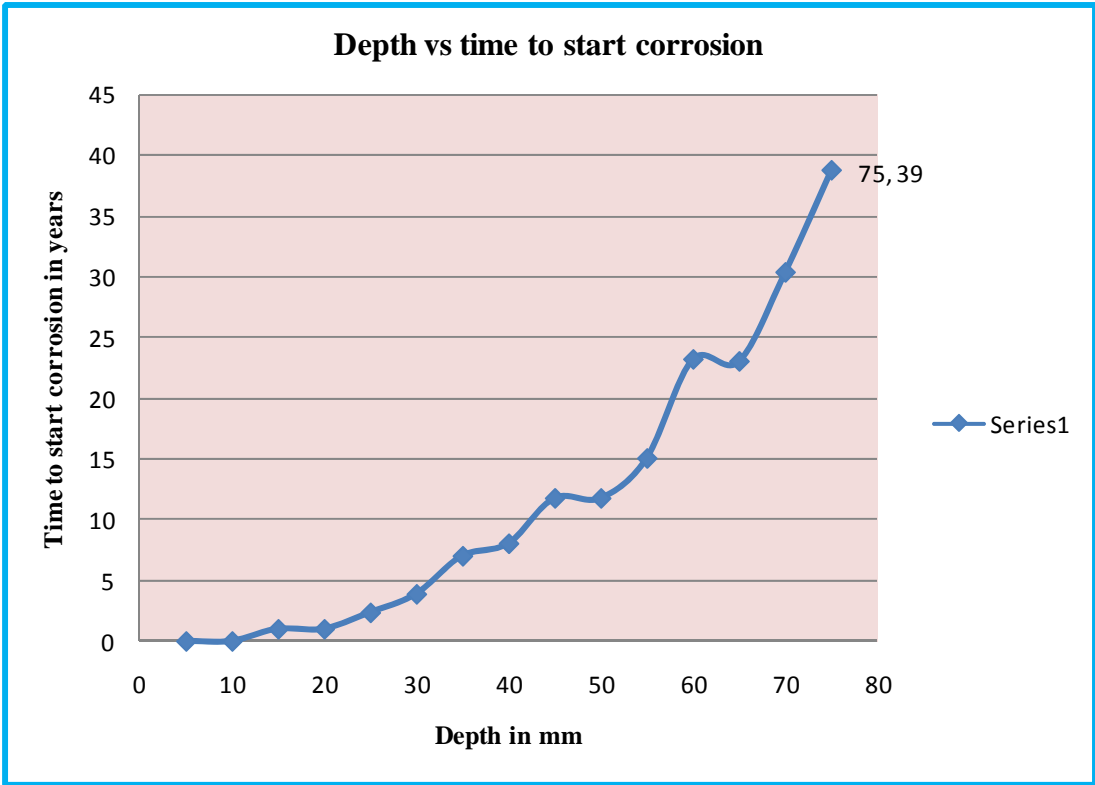


Figure 5-1: Depth vs time to start corrosion (From Chloride diffusion spreadsheet model)

Appendix A

Appendix A.1

The Yellow filled with red color shows input data, and the rest are the output results according to Inputs. The outputs are the Chloride content with respect to certain depth in mm and time in years. Normally, it is assumed that bridge design life is 100 years, so here maximum 100 years are considered to calculate chloride content and 5 mm is considered as minimum depth, whereas 75 mm is the maximum depth from surface. Noted here all the calculated amount of chloride content is percentage of cement. The Chloride diffusion coefficient D_{ca} is calculated in m^2/sec according to the time in years. Threshold value is in the input table as for different states and for different situation it varies. Here 0.4 of % cement is considered. Regarding age factor n , there are three choices as input according to concrete type. The given value for D_{ca} is at 20 years, and has three options to select as inputs according to W/C ratio and material type used. W/c ratio is considered as 0.45 for given D_{ca} , for different W/C ratio follow the Figure 3.1. The recommended surface chloride level which is % weight of concrete varies according to the mix types. The graph below shows the chloride content in % of cement with respect to depth in mm and time of exposure in years taking the value from Table A-1 (see below). In Table A-2 (see below) the CR, Corrosion rate is calculated for different time period in years and depth in mm using the exponential equation where the values of a and b are selected according to the environmental condition. Then the calculated value is compared with the threshold value to get the time to activation of corrosion. The last column shows the time of activation of corrosion.

Case 1 : For this case Portland cement, upper 90% condition for surface chloride (C_{sn}), 0.45 water cement ratio and threshold value 0.4 (% of cement) or 1.2 microns/year are considered.

Mix details & input Data		
Cement type (See below)		
Cement content (kg/m ³)	400	
Density(kg/m ³)	2400	
W/C ratio	0.45	
Cl threshold level	0.4	(% cement)
CR threshold	1.2	(microns/yr)

Age Factor,n	
Portland cement conc	-0.264
Pfa conc	-0.699
ggbs concrete	-0.621
Input for the calculation	-0.264

Csn(% concrete)	Pc concrete	blnd cmt mix.
Typical	0.36%	0.51%
Upper 90%	0.70%	0.85%
Upper 95%	0.79%	0.94%
Input for calculation	0.7	

Estimated Dce at 20 years	w/c	Dce at 20 yrs
Pc	0.45	9.41E-13
pfa & ggbs	0.45	2.04E-13
silica fume	0.45	6.14E-13
Input for calculation	0.45	9.41E-13

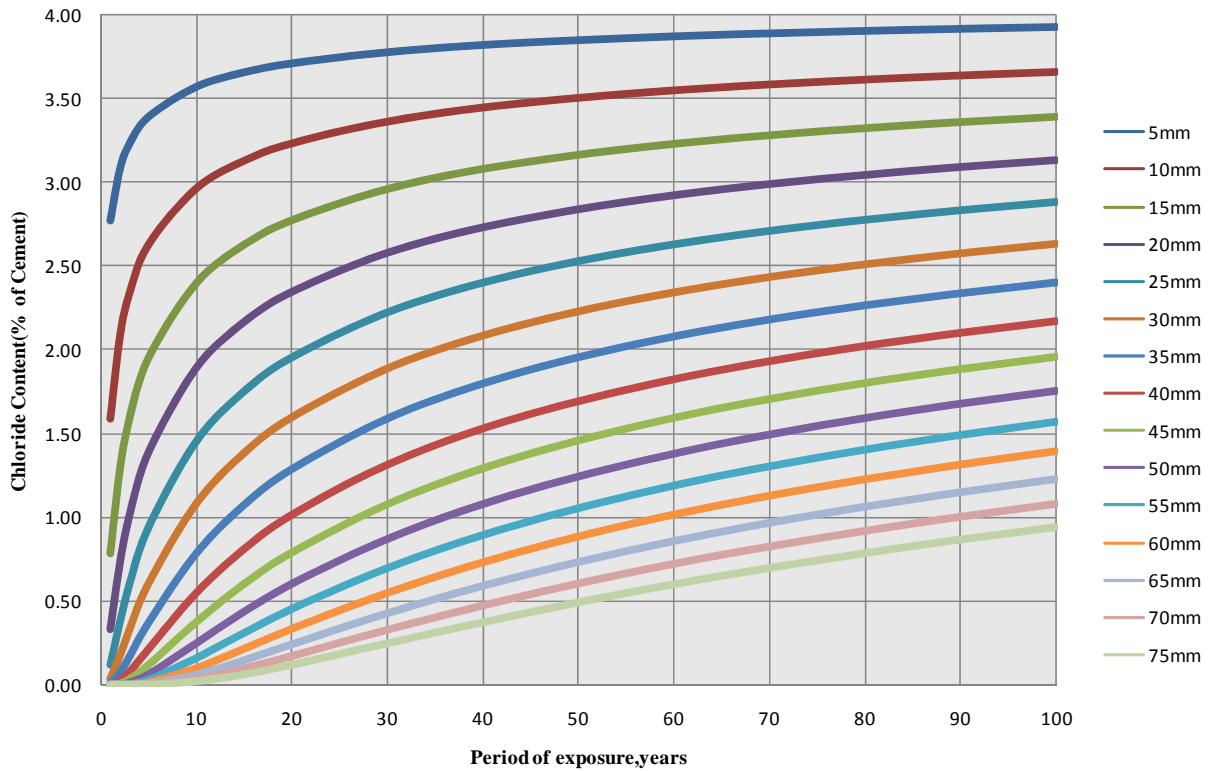
For information	
Cement codes	
Portland Cement	pc
pc/pulverised fuel ash	pfa
pc/ground granulated bfs	ggbs
pc/silica fume	sf
Dce at age of .Yrs	20
Multiplier	6

Table A-1: Calculated chloride contents(% cement) with period of exposure and depth

Depth (mm)	Time(yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
	Dce(m ² /sec)													
	2.075E-12	1.728E-12	1.553E-12	1.357E-12	1.13E-12	1.015E-12	9.41E-13	8.455E-13	7.836E-13	7.388E-13	7.041E-13	6.76E-13	6.422E-13	6.153E-13
Choloride content (% cement)														
5	2.77	3.08	3.23	3.39	3.57	3.66	3.71	3.78	3.82	3.85	3.87	3.89	3.91	3.93
10	1.59	2.08	2.34	2.63	2.97	3.13	3.23	3.37	3.45	3.51	3.55	3.59	3.63	3.66
15	0.78	1.29	1.59	1.95	2.40	2.63	2.78	2.96	3.08	3.17	3.23	3.29	3.35	3.40
20	0.33	0.72	1.01	1.39	1.89	2.17	2.35	2.58	2.73	2.84	2.93	2.99	3.07	3.14
25	0.12	0.37	0.60	0.94	1.45	1.75	1.95	2.22	2.40	2.53	2.63	2.71	2.81	2.88
30	0.03	0.17	0.33	0.61	1.08	1.38	1.60	1.89	2.09	2.23	2.35	2.44	2.55	2.64
35	0.01	0.07	0.17	0.37	0.78	1.07	1.29	1.59	1.80	1.95	2.08	2.18	2.30	2.40
40	0.00	0.03	0.08	0.22	0.55	0.81	1.02	1.32	1.53	1.70	1.83	1.94	2.07	2.18
45	0.00	0.01	0.03	0.12	0.38	0.60	0.79	1.08	1.29	1.46	1.60	1.71	1.85	1.96
50	0.00	0.00	0.01	0.06	0.25	0.44	0.60	0.87	1.08	1.25	1.38	1.50	1.64	1.76
55	0.00	0.00	0.01	0.03	0.16	0.31	0.45	0.70	0.89	1.06	1.19	1.31	1.45	1.57
60	0.00	0.00	0.00	0.01	0.10	0.22	0.33	0.55	0.73	0.88	1.02	1.13	1.27	1.39
65	0.00	0.00	0.00	0.01	0.06	0.15	0.24	0.43	0.59	0.74	0.86	0.97	1.11	1.23
70	0.00	0.00	0.00	0.00	0.03	0.10	0.17	0.33	0.47	0.61	0.72	0.83	0.96	1.08
75	0.00	0.00	0.00	0.00	0.02	0.06	0.12	0.25	0.38	0.49	0.60	0.70	0.83	0.95

Note:
 1) Yellow fill with Red color are Input cells
 2) Rest of are output Cells

Figure A-1: Chloride Ingress with time at depths



Exponential relationship between corrosion rate CR and chloride content Cx, $CR=a \cdot e^{(b \cdot Cx)}$		
Exposure Condition	a	b
Wet,rarely dry	0.84	0.64
Splash(cyclic wet/dry) or airborne seawater	0.53	1.56
Tidal	0.46	1.84
Input here for selected exposure condition	0.53	1.56

Depth (mm)	Time (yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
	5	39.99	64.62	81.73	105.34	139.36	159.43	173.45	192.64	205.71	215.47	134.80	229.53	237.25
10	6.34	13.60	20.37	32.12	54.11	69.97	82.33	100.97	114.81	125.77	134.80	142.45	152.08	160.10
15	1.80	3.94	6.32	11.18	22.42	32.03	40.31	54.00	65.03	74.26	82.19	89.12	98.12	105.84
20	0.89	1.64	2.55	4.61	10.15	15.56	20.63	29.73	37.63	44.60	50.81	56.42	63.92	70.53
25	0.64	0.94	1.34	2.29	5.10	8.13	11.15	16.98	22.38	27.37	31.98	36.27	42.15	47.50
30	0.56	0.69	0.88	1.36	2.87	4.60	6.42	10.12	13.75	17.24	20.57	23.74	28.22	32.40
35	0.54	0.59	0.69	0.95	1.80	2.83	3.94	6.32	8.75	11.18	13.56	15.88	19.23	22.42
40	0.53	0.55	0.60	0.74	1.25	1.89	2.59	4.15	5.79	7.48	9.18	10.86	13.35	15.78
45	0.53	0.54	0.56	0.64	0.95	1.36	1.82	2.86	3.99	5.18	6.39	7.62	9.47	11.30
50	0.53	0.53	0.54	0.58	0.78	1.05	1.36	2.07	2.86	3.71	4.59	5.49	6.86	8.25
55	0.53	0.53	0.53	0.56	0.68	0.86	1.08	1.57	2.14	2.75	3.39	4.06	5.09	6.14
60	0.53	0.53	0.53	0.54	0.62	0.74	0.89	1.25	1.66	2.11	2.58	3.08	3.86	4.67
65	0.53	0.53	0.53	0.54	0.58	0.67	0.77	1.03	1.34	1.67	2.03	2.41	3.00	3.62
70	0.53	0.53	0.53	0.53	0.56	0.62	0.69	0.88	1.11	1.36	1.64	1.93	2.39	2.87
75	0.53	0.53	0.53	0.53	0.55	0.58	0.64	0.78	0.95	1.15	1.36	1.58	1.94	2.32

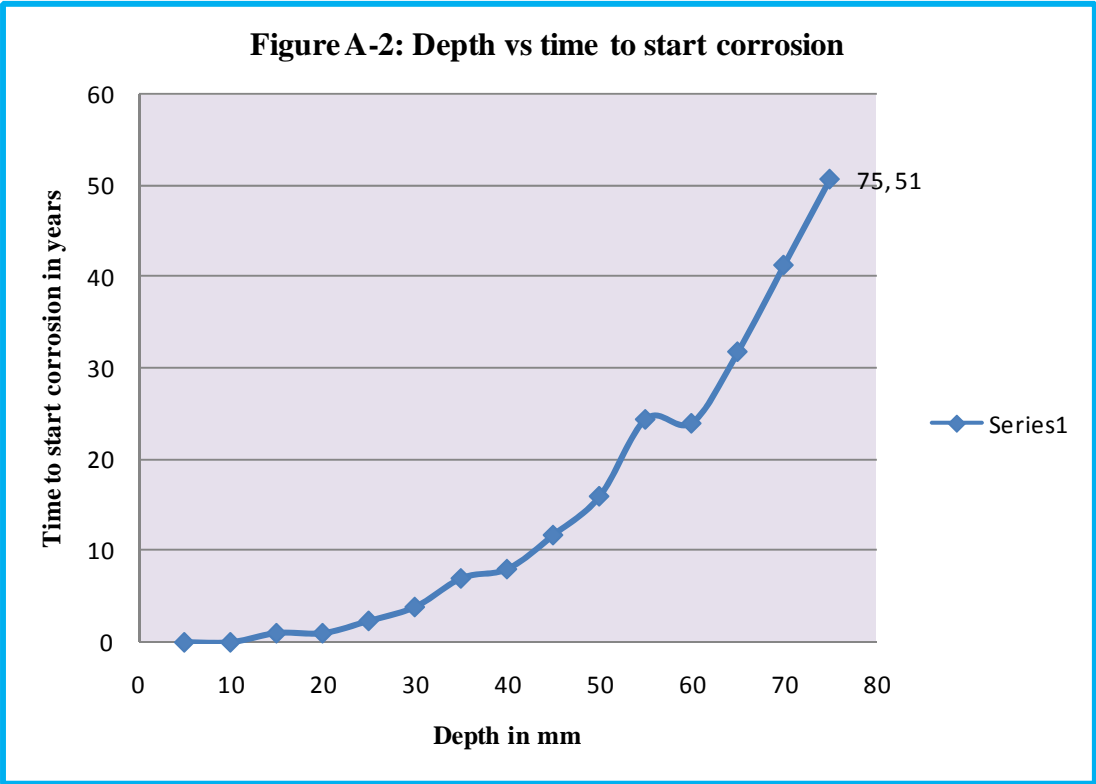
Considering CR Threshold Value= 1.2 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	16	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	24	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	24	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	0	32	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	0	41	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	0	51	0	0	0	0

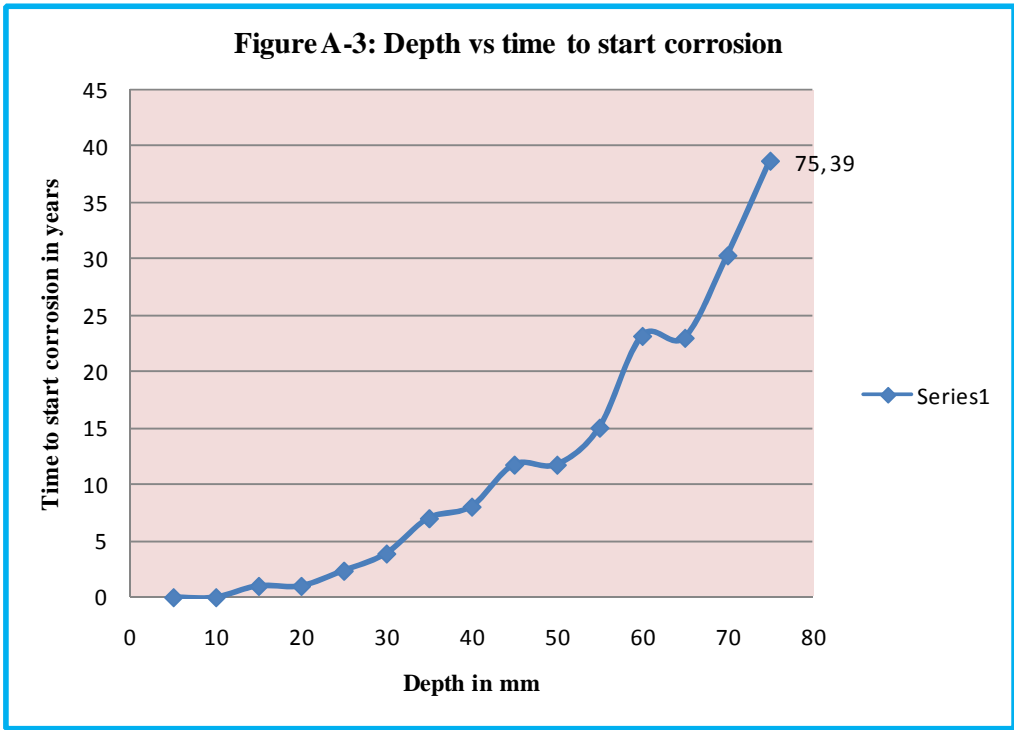
Considering CI Threshold Value = 0.4 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	15	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	23	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	23	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	30	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	39	0	0	0	0	0

Considering CR Threshold Value= 1.2 (microns/yr)



Considering CI Threshold Value = 0.4 (% cement)



Case 2: For this case Portland cement, Typical condition for surface chloride (C_{sn}), 0.45 water cement ratio and threshold value 0.4 (% of cement) or 1.2 microns/year are considered.

Mix details & input Data		
Cement type (See below)		
Cement content (kg/m ³)	400	
Density(kg/m ³)	2400	
W/C ratio	0.45	
Cl threshold level	0.4	(% cement)
CR threshold	1.20	(microns/yr)

Age Factor,n	
Portland cement conc	-0.264
Pfa conc	-0.699
ggbs concrete	-0.621
Input for the calculation	-0.264

Csn(% concrete)	Pc concrete	blnd cmt mix.
Typical	0.36%	0.51%
Upper 90%	0.70%	0.85%
Upper 95%	0.79%	0.94%
Input for calculation	0.36	

Estimated Dce at 20 years	w/c	Dce at 20 yrs
Pc	0.45	9.41E-13
pfa & ggbs	0.45	2.04E-13
silica fume	0.45	6.14E-13
Input for calculation	0.45	9.41E-13

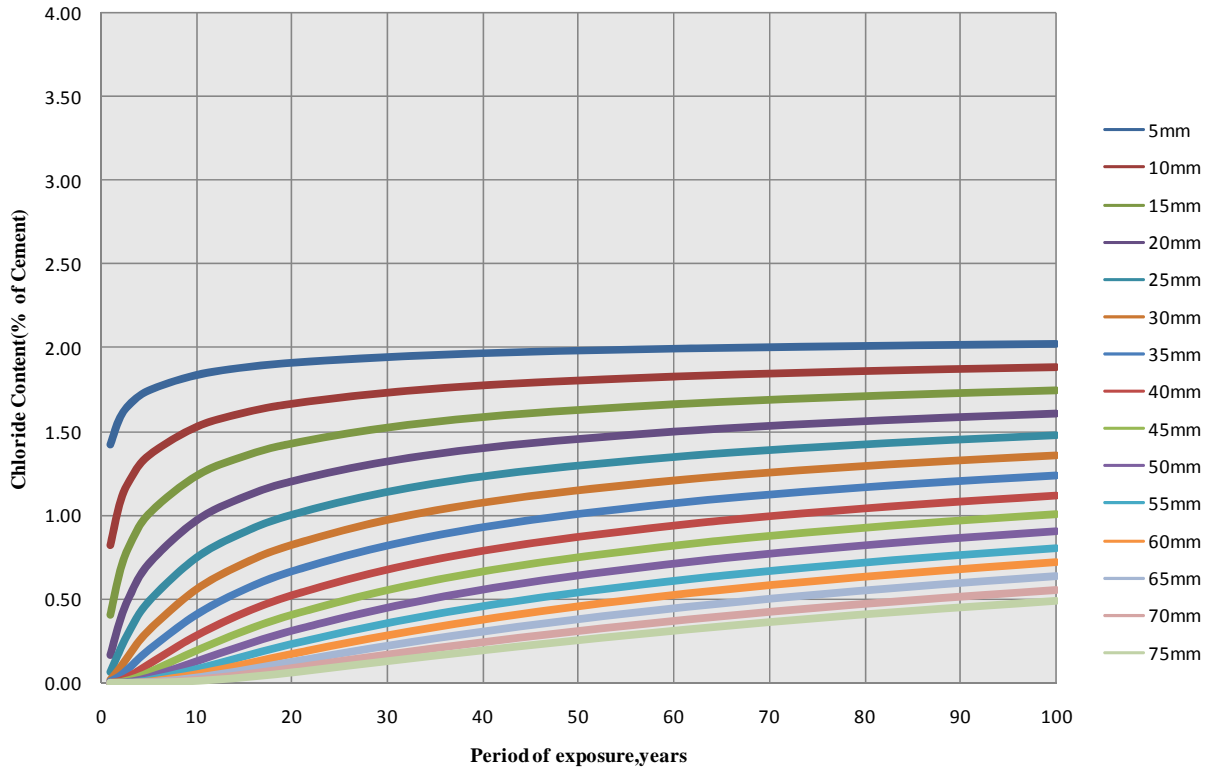
For information	
Cement codes	
Portland Cement	pc
pc/pulverised fuel ash	pfa
pc/ground granulated bfs	ggbs
pc/silica fume	sf
Dce at age of ..Yrs	20
Multiplier	6

Table A-1: Calculated chloride contents(% cement) with period of exposure and depth

Depth (mm)	Time(yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
	Dce(m ² /sec)													
	2.075E-12	1.728E-12	1.553E-12	1.357E-12	1.13E-12	1.015E-12	9.41E-13	8.455E-13	7.836E-13	7.388E-13	7.041E-13	6.761E-13	6.422E-13	6.153E-13
	Choloride content (% cement)													
5	1.43	1.58	1.66	1.74	1.84	1.88	1.91	1.94	1.97	1.98	1.99	2.00	2.01	2.02
10	0.82	1.07	1.20	1.35	1.53	1.61	1.66	1.73	1.77	1.80	1.83	1.84	1.87	1.88
15	0.40	0.66	0.82	1.01	1.23	1.35	1.43	1.52	1.59	1.63	1.66	1.69	1.72	1.75
20	0.17	0.37	0.52	0.71	0.97	1.11	1.21	1.33	1.41	1.46	1.50	1.54	1.58	1.61
25	0.06	0.19	0.31	0.48	0.75	0.90	1.00	1.14	1.23	1.30	1.35	1.39	1.44	1.48
30	0.02	0.09	0.17	0.31	0.56	0.71	0.82	0.97	1.07	1.15	1.21	1.25	1.31	1.36
35	0.00	0.04	0.09	0.19	0.40	0.55	0.66	0.82	0.92	1.01	1.07	1.12	1.18	1.23
40	0.00	0.01	0.04	0.11	0.28	0.42	0.52	0.68	0.79	0.87	0.94	1.00	1.06	1.12
45	0.00	0.00	0.02	0.06	0.19	0.31	0.41	0.56	0.67	0.75	0.82	0.88	0.95	1.01
50	0.00	0.00	0.01	0.03	0.13	0.23	0.31	0.45	0.56	0.64	0.71	0.77	0.84	0.90
55	0.00	0.00	0.00	0.02	0.08	0.16	0.23	0.36	0.46	0.54	0.61	0.67	0.75	0.81
60	0.00	0.00	0.00	0.01	0.05	0.11	0.17	0.28	0.38	0.46	0.52	0.58	0.65	0.72
65	0.00	0.00	0.00	0.00	0.03	0.07	0.12	0.22	0.30	0.38	0.44	0.50	0.57	0.63
70	0.00	0.00	0.00	0.00	0.02	0.05	0.09	0.17	0.24	0.31	0.37	0.43	0.50	0.56
75	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.13	0.19	0.25	0.31	0.36	0.43	0.49

Note:
 1) Yellow fill with Red color are Input cells
 2) Rest of are output Cells

Figure A-1: Chloride Ingress with time at depths



Exponential relationship between corrosion rate CR and chloride content Cx, $CR=a \cdot e^{(b \cdot Cx)}$		
Exposure Condition	a	b
Wet, rarely dry	0.84	0.64
Splash (cyclic wet/dry) or airborne seawater	0.53	1.56
Tidal	0.46	1.84
Input here for selected exposure condition	0.53	1.56

Depth (mm)	Table A-2: Calculation of Corrosion Rate, CR in microns													
	Time (yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
5	4.90	6.27	7.07	8.06	9.31	9.97	10.41	10.99	11.37	11.64	9.15	12.03	12.24	12.40
10	1.90	2.81	3.46	4.38	5.72	6.53	7.10	7.89	8.42	8.83	9.15	9.41	9.73	9.99
15	0.99	1.49	1.90	2.54	3.64	4.37	4.92	5.71	6.29	6.73	7.09	7.39	7.77	8.08
20	0.69	0.95	1.19	1.61	2.42	3.01	3.48	4.20	4.75	5.18	5.54	5.85	6.23	6.56
25	0.58	0.71	0.86	1.13	1.70	2.16	2.54	3.15	3.63	4.03	4.37	4.66	5.03	5.35
30	0.54	0.61	0.69	0.86	1.26	1.61	1.91	2.42	2.83	3.18	3.48	3.75	4.09	4.39
35	0.53	0.56	0.61	0.71	0.99	1.25	1.49	1.90	2.24	2.54	2.81	3.05	3.36	3.64
40	0.53	0.54	0.56	0.63	0.83	1.02	1.20	1.53	1.81	2.07	2.30	2.51	2.79	3.04
45	0.53	0.53	0.54	0.58	0.72	0.86	1.00	1.26	1.50	1.71	1.91	2.09	2.33	2.56
50	0.53	0.53	0.54	0.56	0.65	0.75	0.86	1.07	1.26	1.44	1.61	1.76	1.98	2.17
55	0.53	0.53	0.53	0.54	0.60	0.68	0.76	0.93	1.09	1.24	1.38	1.51	1.70	1.87
60	0.53	0.53	0.53	0.54	0.57	0.63	0.69	0.82	0.95	1.08	1.20	1.31	1.47	1.62
65	0.53	0.53	0.53	0.53	0.56	0.60	0.64	0.75	0.85	0.96	1.06	1.15	1.29	1.42
70	0.53	0.53	0.53	0.53	0.55	0.57	0.61	0.69	0.78	0.86	0.95	1.03	1.15	1.26
75	0.53	0.53	0.53	0.53	0.54	0.56	0.58	0.65	0.72	0.79	0.86	0.93	1.03	1.13

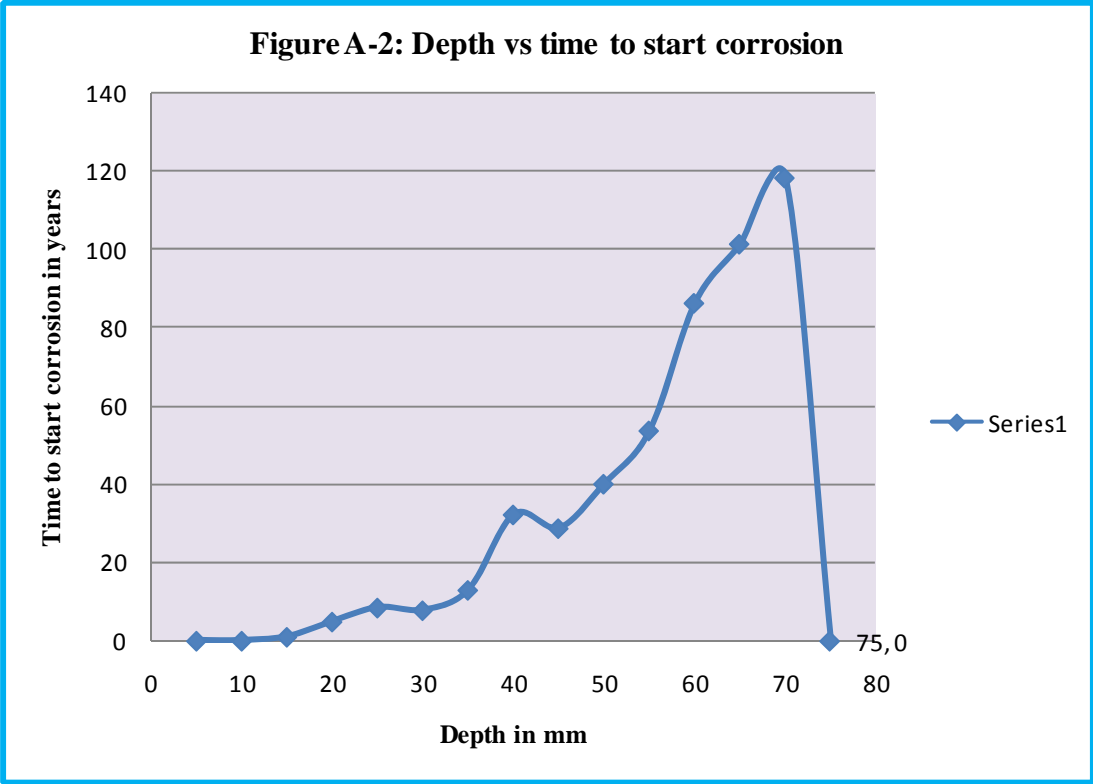
Considering CR Threshold Value= 1.2 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	0	0	5	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	0	13	0	0	0	0	0	0	0	0	0
40	0	0	0	0	0	0	0	32	0	0	0	0	0	0	0
45	0	0	0	0	0	0	0	29	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0	0	40	0	0	0	0	0	0
55	0	0	0	0	0	0	0	0	0	54	0	0	0	0	0
60	0	0	0	0	0	0	0	0	0	0	0	86	0	0	0
65	0	0	0	0	0	0	0	0	0	0	0	0	101	0	0
70	0	0	0	0	0	0	0	0	0	0	0	0	0	118	0
75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

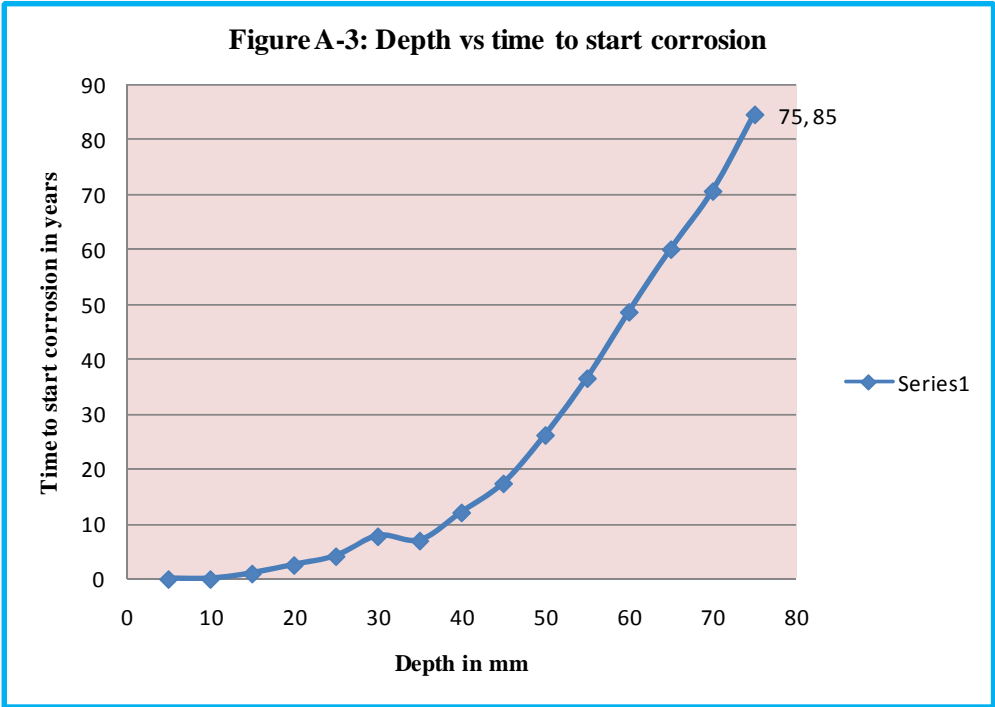
Considering Cl Threshold Value = 0.4 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	0	17	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0	26	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	0	37	0	0	0	0	0	0
60	0	0	0	0	0	0	0	0	0	49	0	0	0	0	0
65	0	0	0	0	0	0	0	0	0	0	60	0	0	0	0
70	0	0	0	0	0	0	0	0	0	0	0	71	0	0	0
75	0	0	0	0	0	0	0	0	0	0	0	0	85	0	0

Considering CR Threshold Value= 1.2 (microns/yr)



Considering CI Threshold Value = 0.4 (% cement)



Case 3: For this case Portland cement, upper 95% condition for surface chloride (C_{sn}), 0.45 water cement ratio, Tidal zone and threshold value 0.4 (% of cement) or 1.2 microns/year are considered.

Mix details & input Data		
Cement type (See below)		
Cement content (kg/m ³)	400	
Density(kg/m ³)	2400	
W/C ratio	0.45	
Cl threshold level	0.4	(% cement)
CR threshold	1.2	(microns/yr)

Age Factor,n	
Portland cement conc	-0.264
Pfa conc	-0.699
ggbs concrete	-0.621
Input for the calculation	-0.264

Csn(% concrete)	Pc concrete	blnd emt mix.
Typical	0.36%	0.51%
Upper 90%	0.70%	0.85%
Upper 95%	0.79%	0.94%
Inout for calculation	0.79	

Estimated Dce at 20 years	w/c	Dce at 20 yrs
Pc	0.45	9.41E-13
pfa & ggbs	0.45	2.04E-13
silica fume	0.45	6.14E-13
Input for calculation	0.45	9.41E-13

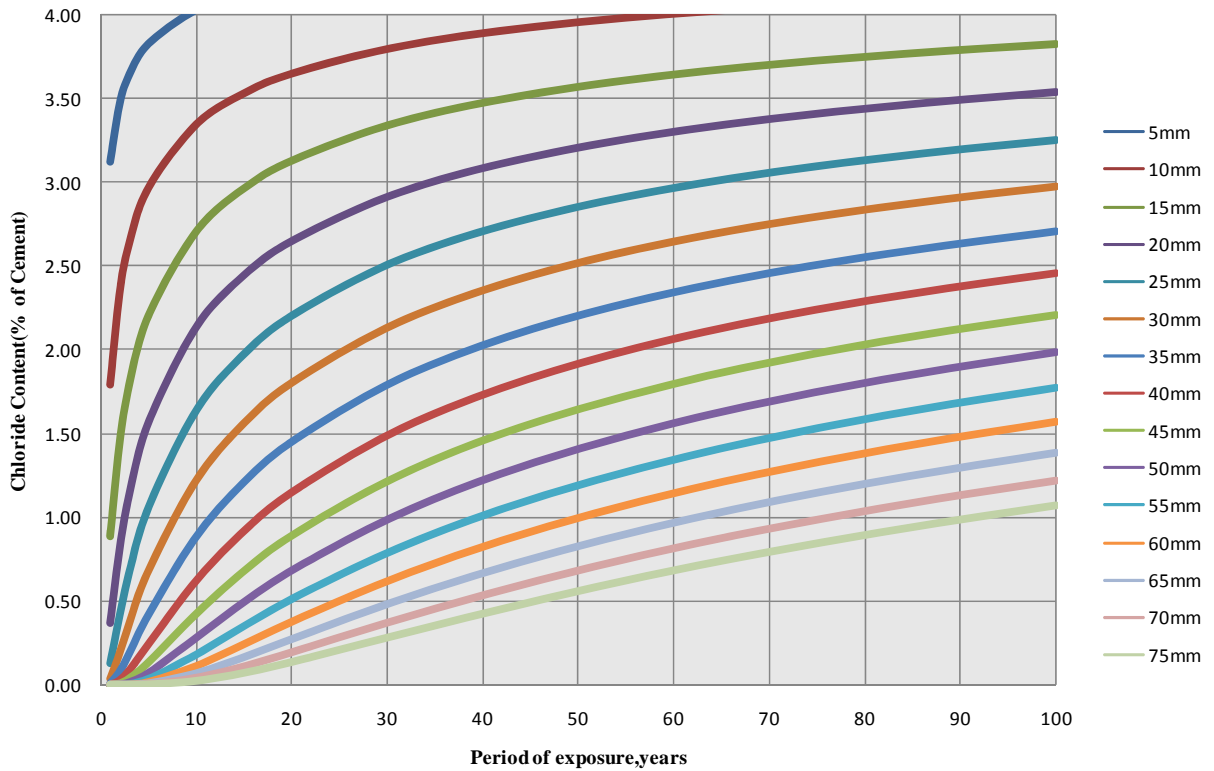
For information	
Cement codes	
Portland Cement	pc
pc/pulverised fuel ash	pfa
pc/ground granulated bfs	ggbs
pc/silica fume	sf
Dce at age of .Yrs	20
Multiplier	6

Table A-1: Calculated chloride contents(% cement) with period of exposure and depth

Depth (mm)	Time(yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
	Dce(m ² /sec)													
	2.075E-12	1.728E-12	1.533E-12	1.357E-12	1.13E-12	1.015E-12	9.41E-13	8.455E-13	7.836E-13	7.388E-13	7.041E-13	6.76E-13	6.422E-13	6.153E-13
	Choloride content (% cement)													
5	3.13	3.47	3.64	3.83	4.03	4.13	4.19	4.27	4.31	4.35	4.37	4.39	4.42	4.43
10	1.80	2.35	2.64	2.97	3.35	3.53	3.65	3.80	3.89	3.96	4.01	4.05	4.09	4.13
15	0.89	1.45	1.79	2.21	2.71	2.97	3.13	3.35	3.48	3.58	3.65	3.71	3.78	3.83
20	0.37	0.82	1.14	1.57	2.14	2.45	2.65	2.91	3.08	3.21	3.30	3.38	3.47	3.54
25	0.13	0.42	0.67	1.06	1.64	1.97	2.20	2.51	2.71	2.85	2.97	3.06	3.17	3.25
30	0.04	0.19	0.37	0.68	1.22	1.56	1.80	2.13	2.36	2.52	2.65	2.75	2.88	2.98
35	0.01	0.08	0.19	0.42	0.89	1.21	1.45	1.79	2.03	2.21	2.35	2.46	2.60	2.71
40	0.00	0.03	0.09	0.24	0.62	0.92	1.15	1.49	1.73	1.92	2.06	2.19	2.33	2.45
45	0.00	0.01	0.04	0.14	0.42	0.68	0.89	1.22	1.46	1.65	1.80	1.93	2.09	2.21
50	0.00	0.00	0.02	0.07	0.28	0.49	0.68	0.99	1.22	1.41	1.56	1.69	1.85	1.99
55	0.00	0.00	0.01	0.04	0.18	0.35	0.51	0.79	1.01	1.19	1.34	1.47	1.64	1.77
60	0.00	0.00	0.00	0.02	0.11	0.24	0.38	0.62	0.83	1.00	1.15	1.27	1.44	1.57
65	0.00	0.00	0.00	0.01	0.07	0.16	0.27	0.48	0.67	0.83	0.97	1.09	1.25	1.39
70	0.00	0.00	0.00	0.00	0.04	0.11	0.19	0.37	0.54	0.68	0.82	0.93	1.09	1.22
75	0.00	0.00	0.00	0.00	0.02	0.07	0.13	0.28	0.42	0.56	0.68	0.79	0.94	1.07

Note:
 1) Yellow fill with Red color are Input cells
 2) Rest of are output Cells

Figure A-1: Chloride Ingress with time at depths



Exponential relationship between corrosion rate CR and chloride content Cx,
 $CR = a * e^{(b * Cx)}$

Exposure Condition	a	b
Wet, rarely dry	0.84	0.64
Splash (cyclic wet/dry) or airborne seawater	0.53	1.56
Tidal	0.46	1.84
Input here for selected exposure condition	0.46	1.84

Table A-2: Calculation of Corrosion Rate, CR in microns

Depth (mm)	Time (yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
5	145.30	275.17	376.20	527.40	765.44	915.60	1024.33	1177.85	1285.38	1367.26	732.30	1487.25	1554.22	1608.85
10	12.52	34.57	59.18	108.51	217.30	305.92	379.89	498.46	591.44	667.73	732.30	788.17	859.85	920.70
15	2.35	6.65	12.46	26.62	67.24	108.13	146.84	216.68	277.53	331.16	379.02	422.18	479.83	530.72
20	0.91	2.07	3.73	8.20	23.42	41.36	60.20	97.90	134.00	167.98	199.84	229.71	271.20	309.21
25	0.59	0.99	1.59	3.23	9.38	17.42	26.55	46.44	67.09	87.70	107.91	127.55	155.84	182.68
30	0.49	0.66	0.91	1.62	4.36	8.16	12.72	23.31	35.07	47.39	59.96	72.58	91.36	109.76
35	0.47	0.53	0.65	0.99	2.35	4.27	6.65	12.46	19.23	26.62	34.42	42.47	54.81	67.26
40	0.46	0.49	0.54	0.72	1.45	2.49	3.81	7.11	11.10	15.60	20.48	25.63	33.73	42.12
45	0.46	0.47	0.49	0.59	1.01	1.61	2.38	4.34	6.76	9.56	12.66	15.99	21.35	27.02
50	0.46	0.46	0.47	0.52	0.77	1.14	1.61	2.83	4.35	6.13	8.14	10.33	13.91	17.77
55	0.46	0.46	0.46	0.49	0.64	0.88	1.18	1.96	2.95	4.12	5.44	6.91	9.34	12.00
60	0.46	0.46	0.46	0.47	0.57	0.72	0.92	1.44	2.10	2.89	3.79	4.80	6.47	8.33
65	0.46	0.46	0.46	0.47	0.52	0.62	0.76	1.12	1.57	2.12	2.75	3.45	4.63	5.94
70	0.46	0.46	0.46	0.46	0.49	0.56	0.66	0.91	1.23	1.62	2.06	2.56	3.41	4.36
75	0.46	0.46	0.46	0.46	0.48	0.52	0.59	0.77	1.00	1.28	1.61	1.97	2.59	3.28

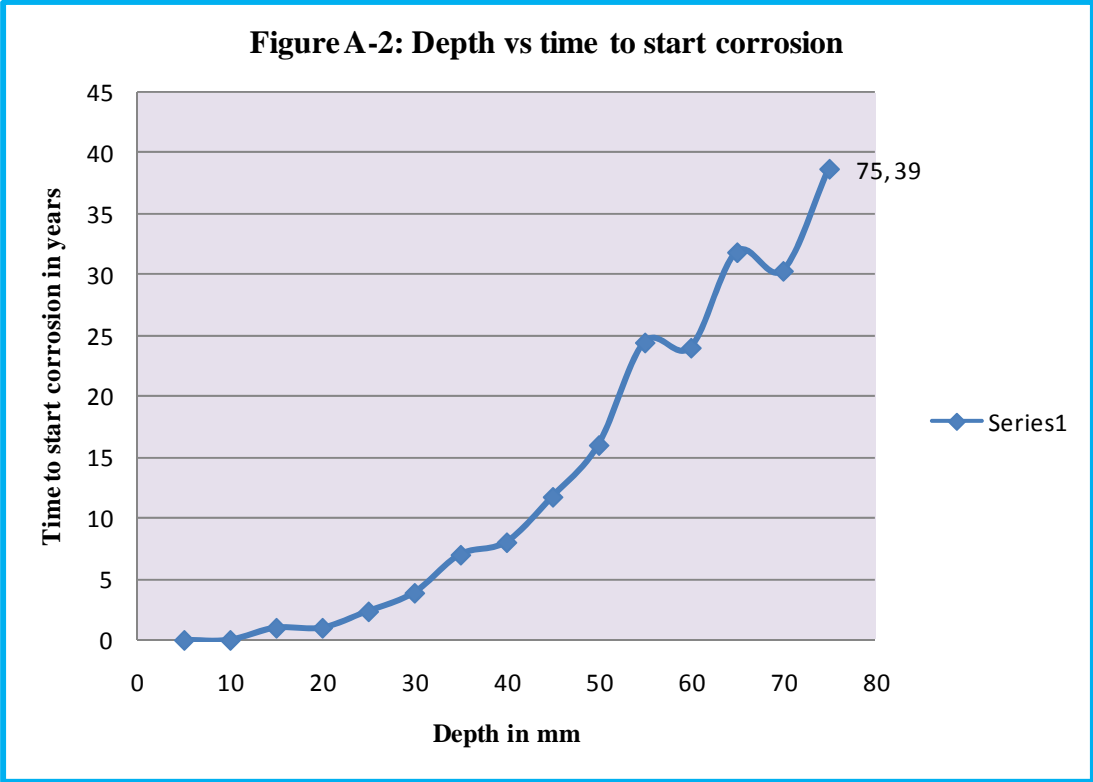
Considering CR Threshold Value= 1.2 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	16	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	24	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	24	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	0	32	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	30	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	39	0	0	0	0	0

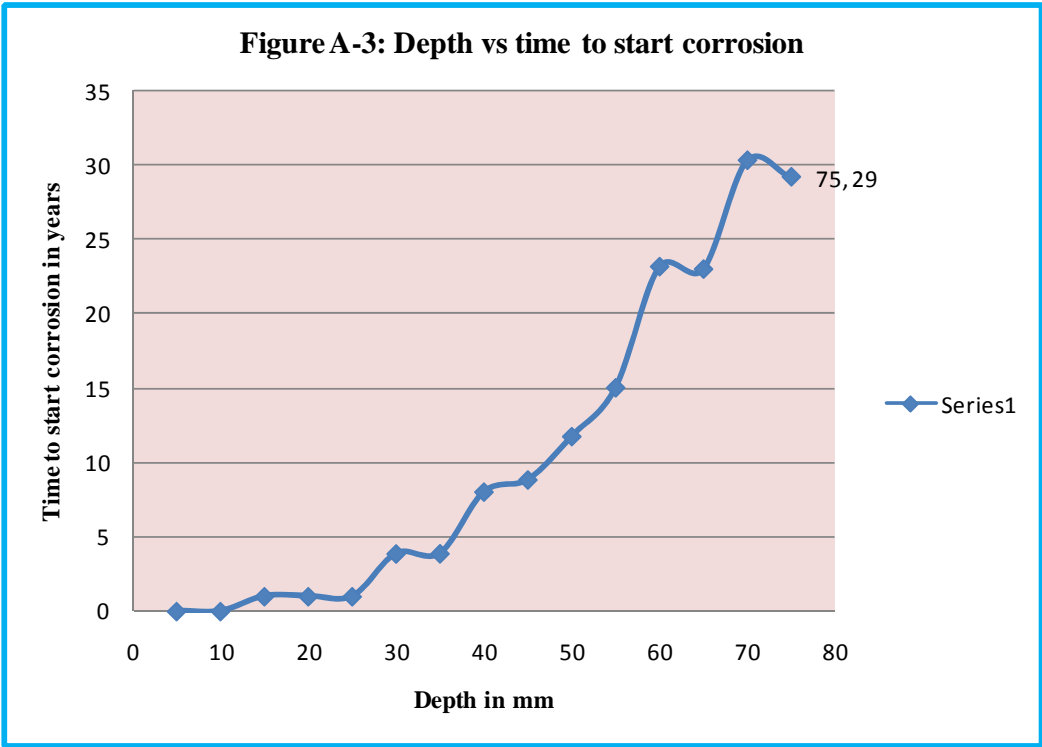
Considering CI Threshold Value = 0.4 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	9	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	15	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	23	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	23	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	30	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	29	0	0	0	0	0

Considering CR Threshold Value= 1.2 (microns/yr)



Considering CI Threshold Value = 0.4 (% cement)



Case 4: For this case pfa concrete, upper 95% condition for surface chloride (C_{sn}), 0.45 water cement ratio and threshold value 0.4 (% of cement) or 1.2 microns/year are considered.

Mix details & input Data		
Cement type (See below)		
Cement content (kg/m ³)	400	
Density(kg/m ³)	2400	
W/C ratio	0.45	
Cl threshold level	0.4	(% cement)
CR threshold	1.2	(microns/yr)

Age Factor,n	
Portland cement conc	-0.264
Pfa conc	-0.699
ggbs concrete	-0.621
Input for the calculation	-0.699

Csn(% concrete)	Pc concrete	blnd cmt mix.
Typical	0.36%	0.51%
Upper 90%	0.70%	0.85%
Upper 95%	0.79%	0.94%
Input for calculation	0.94	

Estimated Dce at 20 years	w/c	Dce at 20 yrs
Pc	0.45	9.41E-13
pfa & ggbs	0.45	2.04E-13
silica fume	0.45	6.14E-13
Input for calculation	0.45	2.04E-13

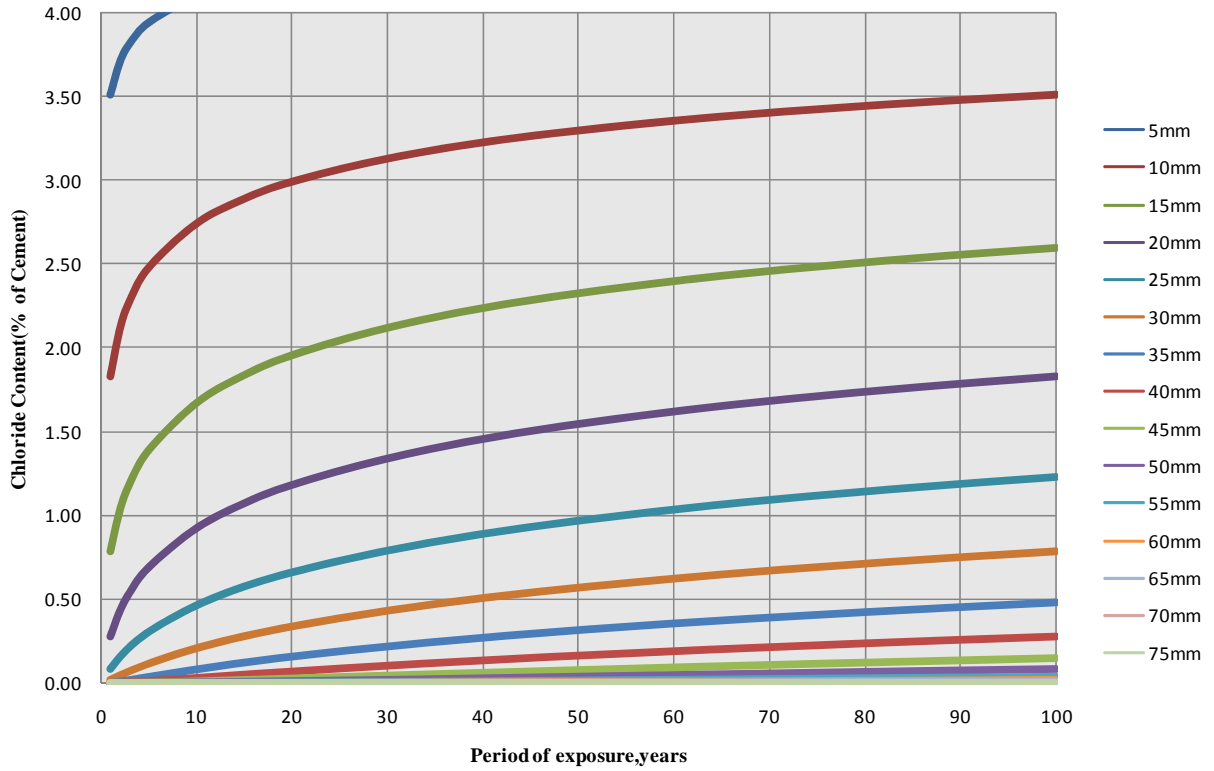
For information	
Cement codes	
Portland Cement	pc
pc/pulverised fuel ash	pfa
pc/ground granulated bfs	ggbs
pc/silica fume	sf
Dce at age of .Yrs	20
Multiplier	6

Table A-1: Calculated chloride contents(% cement) with period of exposure and depth

Depth (mm)	Time(yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
	Dce(m ² /sec)													
	1.656E-12	1.02E-12	7.683E-13	5.376E-13	3.312E-13	2.494E-13	2.04E-13	1.537E-13	1.257E-13	1.075E-13	9.465E-14	8.498E-14	7.42E-14	6.623E-14
Choloride content (% cement)														
5	3.51	3.71	3.81	3.94	4.10	4.19	4.25	4.33	4.38	4.42	4.46	4.48	4.52	4.54
10	1.83	2.11	2.28	2.48	2.74	2.89	2.99	3.13	3.22	3.30	3.35	3.40	3.46	3.51
15	0.79	1.03	1.19	1.39	1.67	1.84	1.95	2.12	2.24	2.32	2.40	2.46	2.53	2.59
20	0.28	0.43	0.54	0.69	0.92	1.07	1.18	1.34	1.46	1.55	1.62	1.68	1.76	1.83
25	0.08	0.15	0.21	0.30	0.46	0.57	0.66	0.79	0.89	0.97	1.03	1.09	1.17	1.23
30	0.02	0.04	0.07	0.11	0.21	0.28	0.34	0.43	0.51	0.57	0.62	0.67	0.73	0.79
35	0.00	0.01	0.02	0.04	0.08	0.12	0.16	0.22	0.27	0.31	0.35	0.39	0.44	0.48
40	0.00	0.00	0.00	0.01	0.03	0.05	0.07	0.10	0.13	0.16	0.19	0.21	0.25	0.28
45	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.04	0.06	0.08	0.09	0.11	0.13	0.15
50	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02	0.03	0.04	0.04	0.05	0.07	0.08
55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.02	0.02	0.03	0.04
60	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.02
65	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Note:
 1) Yellow fill with Red color are Input cells
 2) Rest of are output Cells

Figure A-1: Chloride Ingress with time at depths



Exponential relationship between corrosion rate CR and chloride content Cx, $CR=a^*e^{(b^*Cx)}$		
Exposure Condition	a	b
Wet,rarely dry	0.84	0.64
Splash(cyclic wet/dry) or airborne seawater	0.53	1.56
Tidal	0.46	1.84
Input here for selected exposure condition	0.53	1.56

Depth (mm)	Table A-2: Calculation of Corrosion Rate, CR in microns													
	Time (yrs)													
	1	2	3	5	10	15	20	30	40	50	60	70	85	100
5	126.49	171.94	203.41	248.46	319.45	366.26	401.79	455.07	495.06	527.29	99.24	577.79	607.93	633.70
10	9.21	14.33	18.48	25.29	38.13	48.01	56.27	69.88	81.08	90.71	99.24	106.93	117.25	126.46
15	1.81	2.65	3.37	4.62	7.17	9.29	11.17	14.45	17.32	19.90	22.27	24.46	27.51	30.31
20	0.81	1.03	1.22	1.55	2.23	2.82	3.34	4.29	5.13	5.91	6.64	7.33	8.30	9.21
25	0.60	0.67	0.73	0.85	1.09	1.29	1.48	1.81	2.12	2.40	2.66	2.91	3.27	3.61
30	0.54	0.57	0.59	0.63	0.73	0.82	0.90	1.04	1.17	1.29	1.40	1.51	1.66	1.81
35	0.53	0.54	0.55	0.56	0.60	0.64	0.68	0.75	0.81	0.86	0.92	0.97	1.05	1.12
40	0.53	0.53	0.53	0.54	0.56	0.57	0.59	0.62	0.65	0.68	0.71	0.74	0.78	0.81
45	0.53	0.53	0.53	0.53	0.54	0.55	0.55	0.57	0.58	0.60	0.61	0.63	0.65	0.67
50	0.53	0.53	0.53	0.53	0.53	0.53	0.54	0.54	0.55	0.56	0.57	0.58	0.59	0.60
55	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.54	0.54	0.54	0.55	0.55	0.56	0.56
60	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.54	0.54	0.54	0.54
65	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.54
70	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53
75	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53

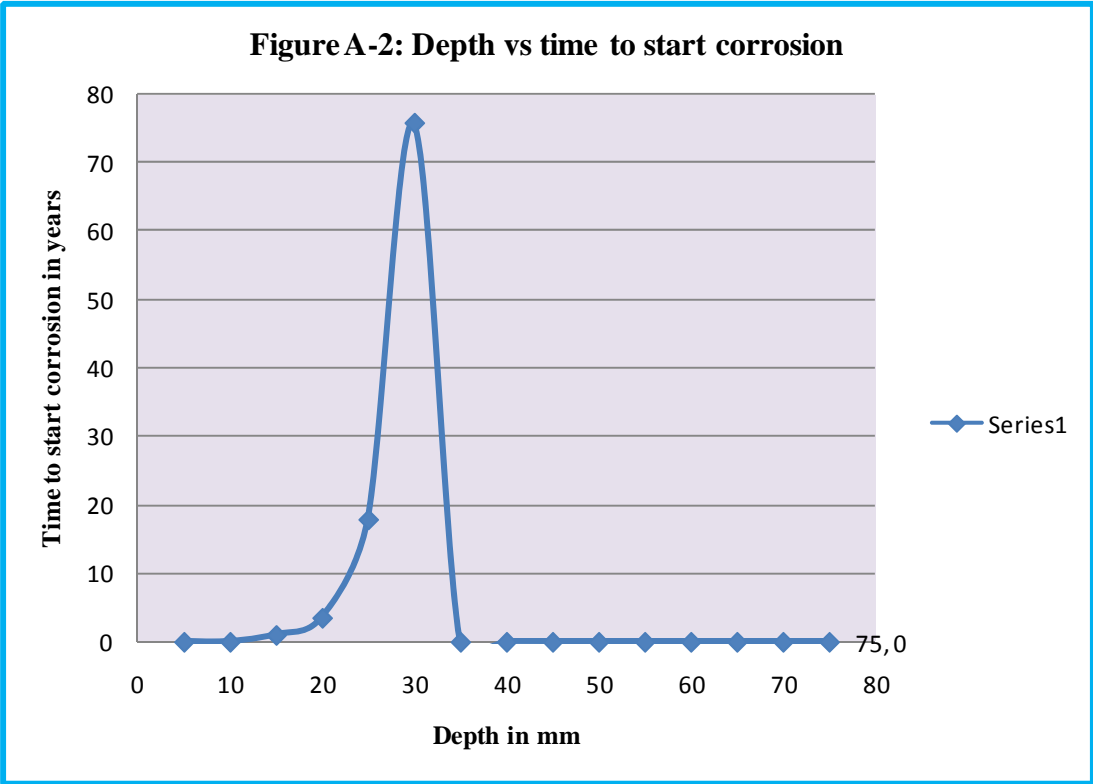
Considering CR Threshold Value= 1.2 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	0	4	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	0	0	18	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	76	0	0	0	0	0
35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

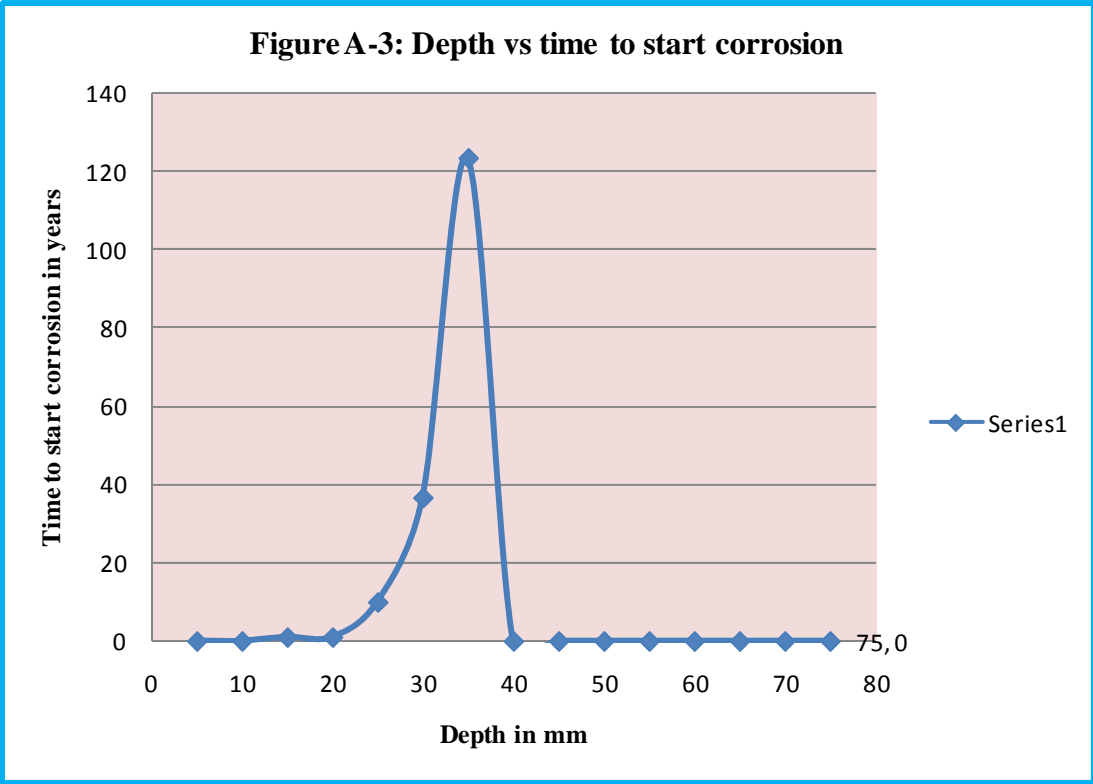
Considering CI Threshold Value = 0.4 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	0	10	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	37	0	0	0	0	0	0	0
35	0	0	0	0	0	0	0	0	0	0	0	0	123	0	0
40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Considering CR Threshold Value= 1.2 (microns/yr)



Considering CI Threshold Value = 0.4 (% cement)



Case 5: Similar to Case1 except threshold value of CR 1.0 microns/year and CI threshold level 0.2 (% wt of cement)

Considering CR Threshold Value= 1.00 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0
40	0	0	0	0	8	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	15	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	24	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	23	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	30	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	39	0	0	0	0	0

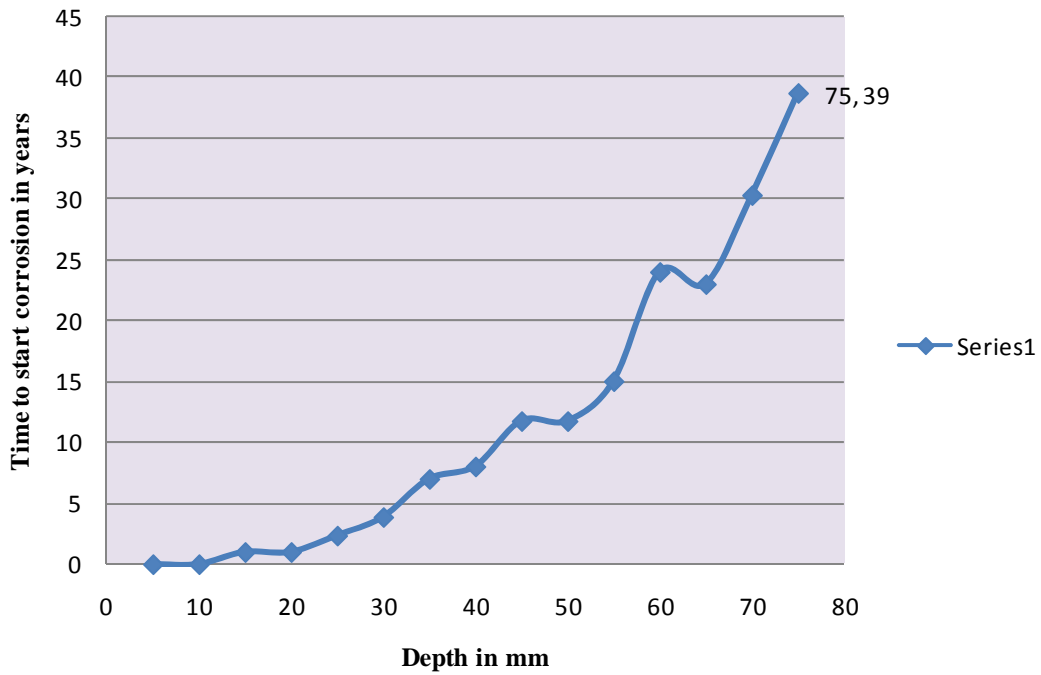
Considering CI Threshold Value = 0.2 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
	Time to activation(Years)														
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
40	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	9	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	10	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	12	0	0	0	0	0	0	0	0	0
60	0	0	0	0	0	13	0	0	0	0	0	0	0	0	0
65	0	0	0	0	0	0	14	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	22	0	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	22	0	0	0	0	0	0

Considering CR Threshold Value= 1

(microns/yr)

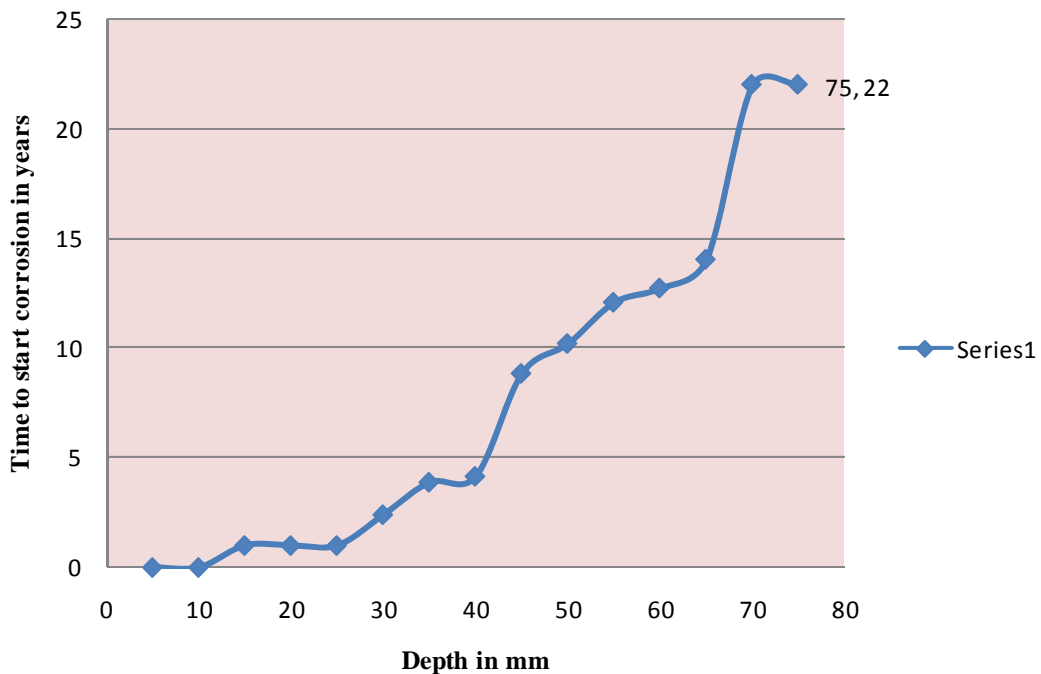
Figure A-2: Depth vs time to start corrosion



Considering CI Threshold Value = 0.2

(% cement)

Figure A-3: Depth vs time to start corrosion



Case 6: Similar to Case1 except threshold value of CR 0.4 microns/year and Cl threshold level 0.05 (% wt of cement)

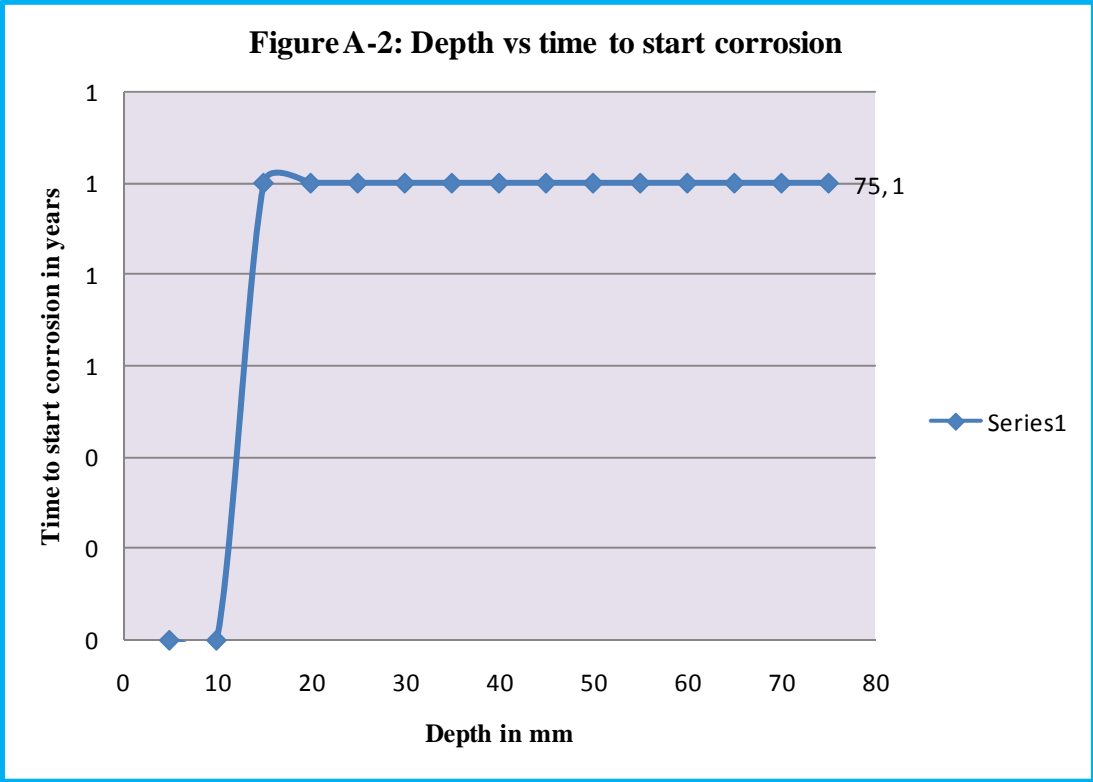
Considering CR Threshold Value= 0.40 (microns/yr)

Depth (mm)	Table A-3: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
35	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
45	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
50	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
65	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
70	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
75	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0

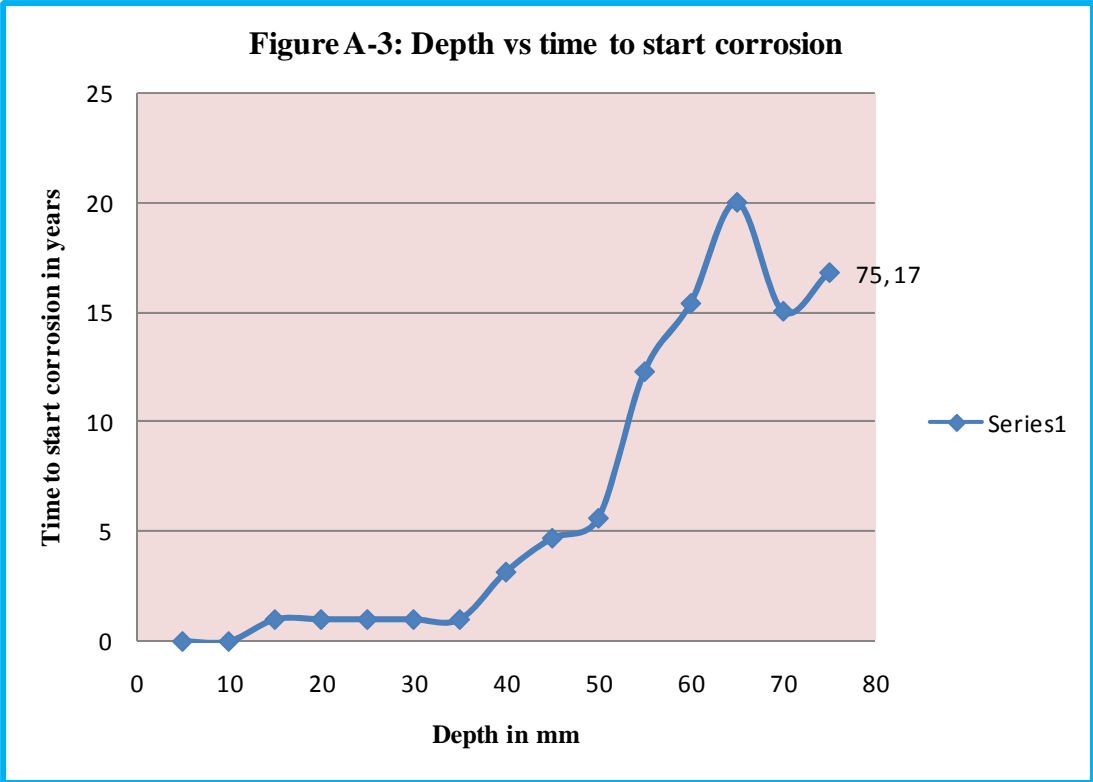
Considering Cl Threshold Value = 0.05 (% cement)

Depth (mm)	Table A-4: calculation of time to corrosion activation based on a defined threshold														Time to activation
	Time (yrs)														
	1	2	3	5	10	15	20	30	40	50	60	70	85	100	
Time to activation(Years)															
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
35	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
40	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	5	0	0	0	0	0	0	0	0	0	0	0
50	0	0	0	6	0	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	12	0	0	0	0	0	0	0	0	0	0
60	0	0	0	0	15	0	0	0	0	0	0	0	0	0	0
65	0	0	0	0	20	0	0	0	0	0	0	0	0	0	0
70	0	0	0	0	0	15	0	0	0	0	0	0	0	0	0
75	0	0	0	0	0	17	0	0	0	0	0	0	0	0	0

Considering CR Threshold Value= 0.4 (microns/yr)



Considering CI Threshold Value = 0.05 (% cement)



REFERENCES

1. *2004 Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance.* FHWA ,Report to Congress,2004
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