## INFLUENCE OF CORROSION OF REINFORCING BARS ON THE BOND BETWEEN STEEL AND CONCRETE

by

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July 1996



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A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfilment of the requirements for the degree of Master of Engineering

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ISBN 0-612-19860-X

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To my husband Bahjat and our daughter Aishah

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### Abstract

Tension tests have been carried out for a preliminary study of the influence of the steel reinforcement corrosion on bond behaviour. The bond strength was studied through both transverse and longitudinal splitting cracks and a relative bond effectiveness of the corroded bars was determined from the crack spacing. Different stages of the steel reinforcement corrosion were established to study their relative bond behaviour, ranging from no corrosion at all to complete corrosion at the steel-concrete interface. These stages of corrosion have been achieved by using an accelerated corrosion method. An electrochemical method was used to accelerate the corrosion within the specimens. Direct current was applied for increasing periods of time to the reinforcing bar embedded in the tension specimens, immersed in a concentrated sodium chloride solution (5% NaCl by weight of water). The reinforcing bar served as the anode, while a bare steel bar was located in the water to serve as the cathode. The chloride content of the concrete plays an important role in the rate of the reinforcement corrosion. The chloride content was obtained for each tension specimen by chemical analysis using the Volhard Method [British Standard (1988), Part 124]. The bond strength decreases rapidly with an increase in the corrosion level, especially in the case of any severe localized corrosion. It has been found that the first level of corrosion, which is 4 percent weight loss due to corrosion, resulted in a 9 percent decrease of the nominal bond stress, while the sixth level of corrosion with a 17.5 percent weight loss (the case of severe localized corrosion) due to corrosion, resulted in a 92 percent loss of the nominal bond stress. The bond behaviour is influenced by the deterioration of the reinforcing bar ribs, and by the reduced adhesion and cohesion of the reinforcing bar due to the widening of the longitudinal splitting crack resulting from corrosion.

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### Résumé

Des essais de traction ont été effectués pour une étude préliminaire de l'effet de la corrosion de l'armature en acier sur le comportement en adhérance. La résistance en adhérance a été étudiée à travers les fissures longitudinales et transversales, et une mesure relative de l'efficacité d'adhérance des barres corrodées a été determinée à partir de l'espacement des fissures. Différentes étapes de corrosion de l'armature en acier ont été établies afin d'étudier leur comportement en adhérance relative, variant d'une corrosion nulle à une corrosion complète à l'interface acier-béton. Ces étapes de corrosion ont été accomplies en utilisant une méthode de corrosion accélérée. Une méthode électrochimique a été réalisée à l'intérieur des specimens. Un courant direct a été appliqué durant des périodes de temps croissantes sur les armatures en acier des specimens en traction immersés dans une solution concentrée de chlorure de sodium (5% NaCl par poids d'eau). Les armatures en acier ont servi d'anode, tandis qu'une plaque en acier a été placée dans l'eau pour servir de cathode. Le contenu en chlorure du béton joue un rôle important dans la vitesse de corrosion de l'armature. Le contenu en chlorure a été obtenu pour chaque specimen en traction par une analyse chimique en utilisant la méthode de Volhard [British Standard (1988), part 124]. La résistance en adhérance décroit rapidement avec l'augmentation du niveau de corrosion, particulièrement dans le cas d'une corrosion sévère et localisée. Il a été déduit que le premier niveau de corrosion qui est de 4 pourcent corrosion a entrainé 9 pourcent de réduction des contraintes nominales d'adhérance, alors que le sixième niveau de corrosion qui est de 17,5 pourcent corrosion (le cas d'une corrosion sévère et localisée) a entrainé une perte de 92 pourcent des contraintes nominales d'adhérance. Le comportement en adhérance est affecté par la détérioration des barres déformées de renforcement, et par l'adhésion et la cohésion réduites à cause de l'élargissement des fissures longitudinales résultant de la corrosion.

## Acknowledgements

The author would like to express her deepest gratitude to her excellent teacher and thesis supervisor, Professor M.S. Mirza, for his valuable suggestions, constant encouragement and support through the course of this research program. The author is deeply grateful to him for all his assistance, constructive criticism and guidance.

Thanks are extended to the Jamieson Structures Laboratory staff, especially Mr. Ron Sheppard and Marek Przykorski for their assistance in casting and testing the specimens. The author would like to thank Chongku Yi, Zeid Salah with the testing of the specimens, and David Byrne for his asristance with the drawings.

The author also extends her thanks to Mozza Diwani, Dr. Arshad A. Khan, Dr. Homayoun H. Abrishami, for their stimulating discussions and helpful suggestions. The French translation of the abstract by Nourhene Kharouf is very much appreciated. Thanks are extended to all those who encouraged and helped during the course of this project.

The author is grateful to her parents, her sister Asina for their encouragement and love. Most of all, the author is grateful to her companion and husband Bahjat and their daughter Aishah, because without their unconditional love, patience, invaluable encouragement, and understanding, the completion of this project would not have been possible.

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# List of Symbols

$A_{\chi}$		area of steel reinforcement
а	÷-	rib height
С	÷	distance between two ribs
d	=	distance from extreme compressive fiber to centroid of tension reinforcement
$d_{h}$	=	bar diameter
$d_c$	=	distance from extreme tension fiber to center of closest bar
Ecorr	=	corrosion potential
E <sub>a</sub>	=	anodic equilibrium potentials
E <sub>c</sub>	=	cathodic equilibrium potentials
Ε,	=	modulus of elasticity of the reinforcing bar
$f_b$	=	bearing stress
$f_{\rm c}$	=	concrete stress
f <sub>cr</sub>	=	tensile strength of concrete
$f_{\rm c}^*$	=	compressive strength of concrete (from a standard cylinder test)
f,	=	modulus of rupture
$f_{sp}$	=	splitting tensile strength of concrete
$f_y$	=:	specified yield strength of steel reinforcement
i	=	current density
iourr		corrosion current
k	=	constant
I	=	embedment length
I <sub>d</sub>	=	development length
L	=	length of reinforcement
Т	=	applied tensile load on the specimen
S <sub>o</sub>	=	minimum crack spacing
S <sub>m</sub>	=	mean crack spacing

τ,

$T_{cr}$	=	eracking load in a tension specimen
u	=	bond stress
v <sub>a</sub>	=	shear stress developed by adhesion
r <sub>c</sub>	=	shear stress acting on the cylindrical concrete
$w_m$	=	average crack width
Wmax	=	maximum crack width
α	=	rib face angle
<b>6</b> ,	=	strain in the concrete
€,	=	strain in the steel
<b>e</b> <sub>m</sub>	=	average strain
€ <sub>cm</sub>	=	average residual surface strain
η	=	measured polarization
$\eta_a$	=	anodic overpotential
$\eta_c$	=	cathodic overpotential
t	=	time of exposure
x	=	carbonated cover depth

## Introduction

#### **1.1 Background Information**

Corrosion of reinforced concrete was first recognized early in the twentieth century, but it has become worse in recent years with the widespread use of de-icing salts on highways and bridge decks. Usually concrete provides an ideal protective environment for the reinforcing steel. However, when salts (chlorides or sulphates) penetrate the concrete and reach the steel rebars, corrosion normally commences. The corrosion products of the steel reinforcement will swell up to seven times its original size, developing pressures as high as 34.5 MPa (5000 psi) within the concrete, which cause cracking and spalling of the concrete cover and expose the rebar to further corrosion activity. Corrosion of reinforcing steel in concrete has caused catastrophic failures in some specific cases, resulting in injury and death, such as the collapse of the Berlin Congress Hall as shown in Fig. 1.1 [Isecke (1982)] and of a parking garage in Minnesota in Fig. 1.2 [Borgard *et al.* (1990)].

Chloride-induced corrosion of reinforcing steel in concrete bridge decks, parking garage slabs and marine structures has been identified as the primary cause of concrete deterioration. The distress in concrete is caused basically by several interactive factors and characterized mainly by severe environment, unsuitable materials, inadequate construction practices and specifications in conjunction with structural weakness. The Ontario Ministry of Transportation will be spending approximately \$700 million on the repair and rehabilitation of its bridges. The Province of Quebec has committed to spend about \$65 million over the next 5 years on the

repair and rehabilitation of the Montreal area bridges, and another \$334 million on the repair of the area roads. The Federal Government has spent recently \$150 million on the repair and rehabilitation of the Champlain Bridge in Montreal.

Based on the information provided by the Strategic Highway Research Program (SHRP), it is estimated that the cost of the corrosion damage in the United States transportation system now stands at over \$20 billion, and it is increasing at the rate of \$500 million per year. From a survey of collapsed buildings in England from 1974 to 1978, Mehta and Monteiro (1992) showed that eight concrete building structures collapsed because of the corrosion of the steel reinforcement. In 1975, the U.S. Interstate Highway System alone reported the need for US \$6 billion for repair and replacement of reinforced concrete bridge decks. In addition, it was reported that at least 4800 of the 25000 bridges in the State of Pennsylvania were found to be in dire need of repair [Mehta and Monteiro (1992)]. The repair and maintenance of reinforced concrete structures is becoming increasingly important and extensive. In order to increase the reliability of the structure and to reduce maintenance costs, eliminating or at worst impeding the corrosion problem is very important. Also, to design new concrete structures and to repair existing deteriorated concrete structures requires an understanding of the various causes and mechanisms of corrosion of reinforcing and prestressing steel.

The concrete cover acts as a physical barrier to the access of aggressive agents because of its hardness and resistance to wear and tear, and to permeation of fluids containing harmful compounds. The high alkalinity of concrete normally provides excellent protection to the reinforcing steel. Despite the "interest" in the protective qualities of concrete, corrosion of steel is the most common cause of distress in concrete structures. Use of deicing salts in cold climate countries aggravate this situation. Concrete structures subjected to sea water spray in the zone in marine stuctures and carbonation of concrete in industial environments also lead to the depassivation of the protective oxide layer on the reinforcing steel. These distresses have also occurred from errors in concrete mixes, lack of quality control in mixing, placing, consolidating and curing of concrete resulting in permeable concrete. In addition, incorrect use of the different types of cements, supplementary cementitious materials, superplasticizers and other additives available commercially and used without a full understanding of their properties have also resulted in deterioration of concrete structures because of steel corrosion.

Corrosion of the reinforcing steel causes a decrease in the bar diameter which affects adversely the mechanical properties of the steel bar in terms of its ultimate strength, yield strength, ductility, etc. Furthermore, when reinforcement corrodes, the corrosion products occupy a much larger volume than the original steel, and eventually exerts a large force on the concrete surrounding it to cause cracks which grow slowly as the reinforcement continues to corrode followed by spalling of the concrete cover. Also, corrosion of the reinforcing steel causes changes in the surface conditions of the reinforcement steel, and layer of the corrosion products causes loss of cohesion and adhesion at the steel-concrete interface. As corrosion continues, it finally leads to changes in the profile of the bar rib. Eventually, all of the concrete around the steel bar is forced off by the growing corrosion products, and the reinforcement looses not only any remaining protection against corrosion, but also looses a significant part of the bond resistance to transfer the force from the reinforcing steel to the surrounding concrete, and vice versa.

Corrosion is very serious in the case of post-tensioned prestressed concrete due to the fact that the reinforcement is deliberately stressed in tension prior to any loading on the structure. In addition, the post-tensioning tendons are free to move within the concrete over the beam length as they may be anchored only at the ends; such movements are prevented in grouted tendons. Therefore, rusting of tendons may take place without any visible sign on the concrete surface causing a sudden structural failure without any advance warning.

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It should be emphasized that the reinforcing steel is provided in reinforced concrete to resist the tensile forces, and to produce controlled cracking within that zone. However, corrosion not only deteriorates the steel bar and its function of transferring the tensile stresses, but it deteriorates the concrete by spalling of the cover. Therefore, corrosion of the reinforcement has a strong influence on the bond behaviour at the interface between the steel reinforcement and concrete. As corrosion of the reinforcing steel progresses, the bond strength

between the reinforcing steel and concrete diminishes progressively, and major repairs or replacement is needed. While much has been written about the problem, and numerous reports have appeared which discuss how this corrosion can be controlled, only limited data are available about its influence on the bond behaviour at the steel-concrete interface.

### 1.2 Research Significance

A study of the influence of corrosion and cracking on the bond behaviour of reinforced concrete aimed at understanding of how are the bond stresses transferred from the corroded steel to the surrounding concrete (steel-concrete interface) would be quite useful to the progressive deterioration of bond with increasing levels of corrosion. The correlation between corrosion, bond strength and cracking of the reinforced concrete needs urgent attention.

It is anticipated that the findings of this research program and its application will result in a better understanding of the corrosion problem, its seriousness, and its influence on the bond between the steel and concrete.

#### **1.3 Summary of Previous Research**

The following does not attempt a total review of all of the tests on corrosion of the embedded steel in the concrete, but rather the results of some tests that have an impact on corrosion of embedded steel in concrete arc reviewed. Special attention is given to the research that has investigated the influence of corrosion on bond behaviour.

Concrete in sea water is a topic which has been in the literature for generations [Gjorv (1975)]. Some papers were presented as early as 1909 in the International Association for Testing Materials in Copenhagen, which reported on corrosion mechanisms, procedures to reduce or prevent corrosion in the early investigation based on the results of the various experiments and observations. Information on the chloride content and the onset and progress

of corrosion process has been incorporated in the provisions of the various codes of practice presently used in construction in different parts of the world, but research is still needed to explain fully the corrosion phenomena. The ACI Committee 222R (1989: 1994) reports the state-of-the art of the corrosion of metals in concrete. This basic information is also incorporated in the CSA Standards S6, S413 and S474.

A considerable amount of research has been undertaken to study the bond characteristics of deformed bars in concrete. It has been reported by Abrams (1951) that the earliest published tests on bond of reinforced concrete with " iron bars" was carried out by Hyatt in 1877. The ACI Committee 408 (1966; 1991) and the CEB Task Group VI (1981) have summarized some of the major developments in the study of the bond characteristics over the last century.

Several researchers have investigated the bond characteristics of steel bars in reinforced concrete by studying the behaviour of reinforced concrete tension elements. Houde and Mirza (1979) have obtained new and basic information on the bond-slip characteristics of deformed bars at the various load levels using two widely different types of specimens which are anchorage and transfer type of specimens, along with the variation of the concrete strength from 20.5 MPa to 44.0 MPa (2980 psi to 6390 psi). Sixty-two concentric tensile specimens with No. 4, No. 6 and No. 8 bars, with twelve tests on specimens with internally instrumented with No.8 bars, along with another six beam-end tests with similar internal instrumention. The specimens were sliced and examined after testing, and it was found that slips have resulted from gradual deterioration of the concrete keys in front of the ribs. The influence of the concrete cover in restraining the bar was also examined. They found that the slip of the reinforcing bars increases almost linearly with an increase in the steel stresses.

The tensile behaviour of concrete members reinforced with a single reinforcing bar and the influence of transverse cracks and splitting cracks on tension stiffening has also been studied by several researchers such as Abrishami et al. (1995). Fracture mechanics techniques have been developed to model the influence of bond and cracking on the tensile response of reinforced concrete members [Bazant (1992) and Ouyang and Shah (1994)]. However, little research has been undertaken to evaluate the effect of corrosion of the bar surface changes on steel-concrete bond. Therefore, more research is needed in order to gain a better understanding of the nature of the corrosion influence on bond, and the parameters which affect the failure of bond due to corrosion.

Page et al. (1978) conducted research on the effect of mix characteristics and steel surface conditions on the bond between the steel reinforcement and different mortars. It was found that the changes in the properties of the steel oxide film influence the bond strength. In addition, the relationship between the bond strength and the potential resembles an electrocapillary curve.

Sakamoto and Iwasaki (1982) studied the influence of sodium chloride (NaCl) on bond between the reinforcing steel and the concrete using accelerated corrosion tests on 150x150x150 mm cubes, reinforced with a 16 mm diameter reinforcing bar, in accordance with the ASTM Standard C234. The deformed and plain reinforcing bars were galvanised in accordance with the Japanease Industrial Standard (JIS) H8641. The molten zine was maintained at a temperature of 460 C and the immersion time was 30 seconds. The amounts of NaCl (dissolved in the mixing water - 188 kg/m<sup>3</sup> of the concrete) added to the concrete were 0, 0.1, 0.3, 1, 2, and 5 percent by weight of sand (794 kg/m<sup>3</sup> of concrete) in oven dry condition. The curing temperatures used were 20° C, 40° C and 50° C and the ages at testing were 7 and 28 days. Two specimens were tested for each condition.

The bars removed from the specimens, which showed deterioration of bond strength under conditions with high NaCl concentration and elevated temperature, showed corrosion products on the surface. The reinforcing bars in such specimens were affected by corrosion even in the case where rust was not evident. In case of the specimens with galvanized bars and subjected to similar conditions, the galvanized bars showed no signs of corrosion, and there was only some discoloration on the bar surface. Sakamato and Iwasaki (1982) also noted that the layer of the corrosion products becomes dense in its structure because of the constraint of the surrounding concrete. This decreases the rate of consumption of zinc and preserves the bond strength over a longer period, because the layer of corrosion products is prevented from penetrating on to the zine surface. They concluded that zine coating is an effective means to protect the reinforcing steel from the attack if chlorides are present in the concrete.

Al-Sulaimani *et al.* (1990) studied the influence of reinforcing bar corrosion and the associated longitudinal cracking on the steel-concrete interface bond behaviour using the standard pullout and beam tests. They used pullout tests to simulate severe local corrosion conditions, and the beam tests to simulate relatively uniform corrosion conditions along the bar length. The pullout specimens, 150 mm side cubes, were reinforced with 10, 14 and 20mm diameter bars cast centrally to give cover-diameter ratios of 7.50, 5.36 and 3.75, respectively. The effective embedment length-diameter ratio provided was 4.0, with the concrete and the steel strengths being 30 MPa (water-cement ratio = 0.55) and 450 MPa, respectively. Polypropylene fibres were added (0.2 percent by volume) to the concrete for the second test series. The beams, 150 x 1000 mm in size, were reinforced with two 10 mm diameter top bars, one 12 mm diameter bottom bar isolated from the 6 mm diameter closed stirrups at 50 mm centres. The embedment lengths provided were 144 mm in one series and 300 mm in the second series, the latter being required by the ACI 318-83. The bond behaviour at the steel-concrete interface was examined at four different stages of corrosion:

- No corrosion stage
- Precracking stage
- Cracking stage
- Postcracking stage

These stages of corrosion were achieved by impressing a direct current with a density of 2 mA/cm<sup>2</sup> for increasing periods on the reinforcing bar embedded in the pullout, or the beam specimens located in water. The circuitry was so arranged that the steel bar served as the anode, while a stainless steel plate situated in the water acted as the cathode.

Analysis of the test data showed that the bond strength increased with corrosion up to

a certain level (about 1% less of bar weight) because of the increased bar surface roughness with the growth of a firm layer of corrosion products on the bar surface, which enhanced the bond strength at the steel-concrete interface. However, with a progressive increase in the level of corrosion, the bond strength decreased rapidly for the pullout test, but at a much lower rate than for beams. Al-Sulaimani *et al.* (1990) noted that for pullout tests, the bond strength becomes negligible at corrosion level of 7.5 percent weight loss for the 14-mm diameter bar. They noted that the calculated average bond stress was 1.5 times the permissible bond stress calculated using the provisions of ACI 318-83 even after a corrosion level with 5 percent loss of bar weight for the beam tests. This bond determination was attributed to the severe deterioration of the bar lugs or deformations, because of the severe localized corrosion in the pullout specimens, which generated flaky products of corrosion on the bar surface, which along with the widening of the longitudinal crack resulting from corrosion, seriously deteriorated the bond strength at the steel-concrete interface.

Al-Sulaimani *et al.* (1990) observed that introduction of 0.2 percent polypropylene fibres by volume into the concrete improves the bond strength at the steel-concrete interface, particularly during the postcracking stage. This is basically due to the lower level of damage at the bar surface and the contribution of the fibres in improving the confinement and holding capacity of the concrete surrounding the bar.

Cabrera and Ghoddoussi (1992) undertook a laboratory investigation in the influence of reinforcement corrosion on bond strength of deformed bars, using beam and pullout specimens made with ordinary Portland cement (opc), and pulverised fly ash (pfa) and cured in a simulated hot dry environments (35°C and 45% relative humidity). To achieve the different levels of corrosion, a voltage of 3 volts versus saturated calomel electrode was impressed to accelerate the corrosion process. The experimental results were used to determine the relationships between bond stress and the corrosion rate. Similarly, the ultimate bond stress was related to the crack width. They discussed the influence of the cement type on the rate of corrosion and their effect on the bond strength. The concrete with pfa exhibited better resistance to corrosion damage than opc concrete basically because of its higher electrical resistivity.

#### Previous Research At McGill University

Two detailed investigations dealing with accelerated electrochemical corrosion of reinforced concrete have been completed at McGill University [ Palumbo (1991), Farah (1993)]. They developed the test set-up and a detailed procedure for accelerated corrosion testing using a lollipop specimen, with the period to complete corrosion in most specimens not exceeding 45 days. The effects of the clear concrete cover thickness, deformed steel bars with and without an epoxy-coating, and the effectiveness of the surface sealants were studied.

Fazio (1996) has undertaken a similar study of the flexural behaviour of reinforced concrete beams subjected to a similar accelerated corrosion regime as Amleh (1996). He has tested five beams, and he will be testing another two beams, with very extensive corrosion in one of them. The results of this experimental work will be presented in the spring of 1996.

#### 1.4 Scope and Objectives of the Present Investigation

The investigations of the influence of corrosion on the bond behaviour between the reinforcing steel and the concrete reported here were carried out under direct tension testing relative to three stages of corrosion: no corosion, corrosion corresponding to cracking, and postcracking levels. The appearance of the first visible crack was defined as first level of corrosion; in total, seven levels of corrosion were produced, ranging from no corrosion, uncorroded 0.0 percent, to complete corrosion, 17.5 percent weight loss due to corrosion, with a 9 mm longitudinal crack caused by the mechanical pressure due to the volume expansion of corrosion products. Corrosion is measured as the loss of metal of the reinforcing steel bar relative to the original reinforcing steel bar weight.

Tension tests have been carried out for a preliminary study of the influence of the steel reinforcement corrosion on the bond behaviour. The bond strength was studied through both transverse and splitting cracks, and a relative bond effectiveness of the corroded bars was determined from the crack spacing.

The primary objective of this research program is to simulate the prevalent conditions under severe local corrosion which causes significant changes on the surface conditions of the steel reinforcing bar, and to study the effect of the various levels of corrosion on the response of a standard tension specimen with a single reinforcing steel bar and the effectiveness of bond between the reinforcing steel and the concrete. The corrosion rate is evaluated for the different levels of corrosion along with the determination of the chloride-ion penetration.

#### 1.5 Outline of the Thesis

Chapter 1 addresses the goals of this research program, besides presenting an overview of the research undertaken in the general area of bond of corroded bars in reinforced concrete where they have been the subject of intensive study and research during the recent years. Chapter 2 presents some of the latest literature review of the mechanisms of reinforcement corrosion, which have been the subject of intensive research over the past few decades. It discusses how the concrete environments protects the embedded steel. Also, it discusses the characteristics of the concrete of relevance to the corrosion of the reinforcing steel and presents the basic principles of corrosion. Chapter 3 reviews the basic mechanics of bond transfer between the concrete which causes cracking. The stresses and deformations in the concrete, caused by the bonding forces are also presented. This chapter also summerizes the program planned to study the parameters which are influenced by corrosion, to evaluate the bond characteristics in the corroded reinforcement in the tension specimens used in this study. The parameters studied are:

i) Load-deflection response



- ii) Stress-strain relationship
- iii) The crack spacing and crack width

Chapter 4 describes the experimental program of the accelerated corrosion testing to simulate the corrosion conditions in the twelve specimens that underwent the accelerated testing. Chapter 5 describes the experimental program of the direct tension testing of the specimens that was performed after achieving the required corrosion conditions. Chapter 6 reports the results obtained from both experimental programs. Chapter 7 presents the discussion and analysis of the results that were obtained from the two experimental programs. A brief summary of the experimental and analytical work as well as the conclusions are presented in Chapter 8, and recommendations for further research and development on the influence of corrosion on bond behaviour are included.



Fig. 1.1: Collapse of the Berlin Congress Hall, [Isecke (1982)].



Fig. 1.2: Collapse of a salt-damaged parking garage, [Engineering News Record (1984)].

## Chapter 2

# Mechanisms of Corrosion of Embedded Steel in Concrete

The corrosion of steel reinforcement is of the greatest concern in the deterioration of the infrastructure around the world, and therefore, this chapter will discuss the factors that cause and control corrosion of steel in concrete, as several metals will corrode under certain conditions when embedded in concrete. Factors influencing the electrochemical process are also discussed, and some protective measures that can be utilized are presented.

### 2.1 Introduction

Corrosion is the process of the transformation of a metal to its "native" form which is the natural ore state, often as oxides, chlorides or sulphates. This transformation occurs because the compounds such as the oxides "involve" less energy than pure metals, and hence they are more stable thermodynamically. The corrosion process does not take place directly but rather as a series of electrochemical reactions with the passage of an electric current. Corrosion also depends on the type and nature of the metal, the immediate environment, temperature and other related factors. The corrosion may be defined as the destructive attack of a metal by chemical or electrochemical reaction with its environment.

Steel in concrete is normally immune from corrosion because of the high alkalinity of the concrete; the pH of the pore water can be greater than 12.5, which protects the embedded steel against corrosion. This alkalinity of concrete causes passivation of the embedded

reinforcing bars. A microscopic oxide layer which is the 'passive' film, forms on the steel surface due to the high pH, which prevents the dissolution of iron. Furthermore, the concretes made using low water-cement ratios and good curing practices, have a low permeability which minimizes the penetration of the corrosion inducing ingredients. In addition, low permeability is believed to increase the electrical resistivity of the concrete to some degree which helps in reducing the rate of corrosion by retarding the flow of electrical currents within the concrete that accompany the electrochemical corrosion. Consequently, corrosion of the embedded steel requires the breakdown of its passivity.

The steel reinforcement in a majority of concrete structures or concrete elements does not corrode because of these inherent protective characteristics, as long as there is a suitable quality of concrete and proper design of the structure for the intended environmental exposure which does not change during the life of the structure. However, corrosion of the steel in the concrete may result when the conditions mentioned above are not met in reinforced or prestressed concrete. In places of very severe exposure, such as pilings in sea water or bridge decks exposed to deicing salts, the use of other protective measures, i.e. corrosion inhibitors, coatings on steel or sealing of the concrete surface, or cathodic protection may be utilized.

#### 2.2 Concrete as an Electrolyte

An understanding of some of the elements of concrete structure is essential for discussing the factors influencing the corrosion. Concrete consists of a cement paste and coarse and fine aggregates, and the aggregates usually do not play a significant role in the electro-chemical corrosion process. Basically, the cement paste phase of the concrete acts as the electrolyte for the transportation of ions and the ionic current.

Mehta and Monteiro (1992) and others have described the composition of the hydrated cement paste, which consists of two phases - the hydrated minerals and the pore solution (liquid phase). The basic parameters which control steel corrosion are:

- (1) The volume and composition of the pore solution.
- (2) The size and distribution of pores.
- (3) The presence of  $Ca(OH)_2$  in the hardened paste.

#### 2.2.1 Pore Solution

The pore solution composition has a great influence on whether the steel embedded in the concrete will remain passivated, or it will commence to corrode. Recent research has shown that ingress of soluble sodium and potassium compounds can lead to pH values of greater than 13 in the pore solution in ordinary portland cements, however, the pH of the pore solution of blended cement concretes has been observed to be lower. Considerable research has also been undertaken to determine the influence of the cement composition on the degree of binding of the chloride ions, because it is the free chloride ions in the pore solution which are available for attacking the steel and initiating corrosion.

#### 2.2.2 Distribution of Pores

The hydrated cement paste contains several types of voids which have an important influence on its properties. Mainly, there are three types of voids or pores:

- (1) Interlayer space in the calcium silicate hydrated phase pores.
- (2) Capillary pores.
- (3) Air pores.

The permeation of moisture and oxygen to the reinforcing steel surface depends on the permeability of concrete in general, and on the permeability of cement paste in particular, which is governed by the pore size and distribution and their interconnectivity. It should be noted that the capillary pore system is basically responsible for the diffusion and permeation processes, and consequently, they are important for the corrosion of the embedded reinforcing and prestressing steel. While the availability of moisture and oxygen are essential for the formation and maintenance of the passive film on the steel surface, they also control the rate of permeation of chlorides and/or carbon dioxide to the steel surface. The resulting chemical reaction destroys this passive protective layer on the steel surface, and the iron then reacts with the moisture and the oxygen to form oxides and hydroxides of iron, which constitute the corrosion process.

The amount and significance of the paste pore size and their distribution can be determined using a number of techniques such as the mercury intrusion porosimetry, vapour adsorption, vapour desorption, small angle neutron scattering and low-temperature calorimetry.

#### 2.2.3 Calcium Hydroxide

The volume of solids in the hydrated cement paste consists of 20 to 25 percent of calcium hydroxide,  $Ca(OH)_2$ , crystals. Detailed studies of the hydrated cement paste have shown limited solubility of  $Ca(OH)_2$  in aqueous solutions and it remains as a solid substance distributed in the hydrated cement paste. While tricalcium silicate,  $C_3S$ , and dicalcium silicate,  $C_2S$ , have been known to contribute to the strength of the hydrated cement paste, the contribution of calcium hydroxide,  $Ca(OH)_2$ , because of its considerably lower surface area, is much lower. However,  $Ca(OH)_2$  helps maintain a high pH level in the hydrated cement paste and provides a useful pH buffer for the pore solution. In addition,  $Ca(OH)_2$  tends to form large crystals with a distinctive hexagonal-prism morphology, where it precipitates as a coating over the steel surface, the pore walls, the mold walls, which provide physical protection to the steel bars. This explains why the various reinforcement corrosion tests cannot be extrapolated to the behaviour of the steel embedded in the concrete in "real" structures. It should also be noted that the higher the concentration of  $Ca(OH)_2$  in the hydrated cement paste, the longer will be the time for the carbonation front to penetrate the concrete cover thickness and reach the steel surface.

#### 2.3 Principles of Corrosion

The corrosion of iron may occur by several mechanisms. Corrosion may initiate by bacterial action, certain bacteria (desulphovibrio desulphuricans) [Berkely and Pathmanaban (1990)], by direct oxidation (burning), or by acid attack, and by chemical attacks. Such corrosion is of little concern in the concrete. Indirect oxidation (electrochemical corrosion) as a result of dissimilar or non-uniform metals or dissimilar environments is of paramount importance in the deterioration of most concrete structures. Also, corrosion of the reinforcing steel in concrete by electrolysis due to "stray electrical currents" and hydrogen embrittlement due to "stress corrosion" are also relevant. In summary, the corrosion of the reinforcing steel in concrete occurs generally by an electrochemical reaction in the presence of moisture and oxygen, which is believed to be the essential cause for all of the corrosion distress that occurs.

#### 2.3.1 Electrochemical Process

Corrosion of steel by galvanic actions where the highly alkaline environment and the accompanying passivating effect may be destroyed, occurs because of three principle causes: the carbonation of concrete, the ingress of chloride ions into concrete to destroy the passive layer of oxides on the steel bar surface, and the leaching of the alkalies by the streaming water. It must be emphasized that corrosion is possible only if sufficient moisture and oxygen are available. This electrochemical process causing corrosion of the reinforcing steel in the concrete is similar to the action that takes place in a flashlight battery which involves an anodic reaction, consisting of oxidation of the iron, and a cathodic reaction where this reaction consumes any electrons produced during oxidation of the iron, an electrical conductor, and an electrolyte.

The dissolution of the metal in the concrete occurs at the anodic sections where ions are generated in the solution in the form of hydrated anions (the negative pole) by the half-cell reaction where the iron is oxidized to ferrous ions such as:

$$Fe \to Fe^{2+} + 2e \tag{2.1}$$

Corrosion occurs at the anode and results in a loss of the metal from the section, which adheres loosely to the metal section as an oxide. This oxide product is lost eventually from the section under continued corrosion. The Fe<sup>2+</sup> is changed to oxides of iron by a different number of complex reactions, and the volume of the reaction products is several times the volume of the iron. Rosenberg *et al.* (1991) reported on a research done by Nielsen who found that when concrete with actively corroding reinforcement is broken open, a light-green semisolid reaction product, which may be a mixture of Fe(OH)<sub>2</sub> and Fe<sub>3</sub>O<sub>4</sub>, has been observed near the steel, which on exposure to air, turns black (Fe<sub>3</sub>O<sub>4</sub>) and subsequently rust red (Fe(OH)<sub>3</sub>.*n*H<sub>2</sub>O). Figure 2.1 illustrates the relative volumes of iron and its corrosion reactions [Nielsen (1985)].

The electrons released at the anode where the reduction takes place move towards the cathodic regions of the surface where they are assimilated by the atoms of the dissolved oxygen or hydrogen ions and consumed by the cathodic half-cell reaction. When the pore solution is alkaline and has ready access to the air, the reduction of dissolved oxygen takes the form:

$$2H_{2}O + O_{2} + 4e^{-} - 4OH^{-}$$
 (2.2)

The electrical circuit is completed through the electrolyte solution in which the hydrated ions move through cement paste pore solution in the concrete as mentioned above. Figure 2.2 presents the schematic representation of the corrosion of steel in concrete.

#### 2.3.2 Concept of Electrochemical Potential

Anodic and cathodic sections form on the surface of the metal where they are of different electrochemical potential due to the existence of heterogeneities in the corroding system. These heterogeneities in the same metal occur due to metallurgical segregation, different grain orientation or due to the local differences in the electrolyte. This can also occur when two different metals are connected and immersed in the same electrolyte, where one will act as an anode and the other as a cathode, depending on the nobility of the metals involved.

The driving force for the reactions is the difference in electrochemical potential between the anode and the cathode. These potentials may be defined as a measure of the ease of transfer across the steel-concrete interface and the ease of ionization of the dissolved oxygen, respectively. It is not possible to determine the absolute value of an electrochemical potential. Therefore, the potential difference between the anode ( or cathode ) and a reference electrode is taken as a measure of the actual potential. It is quoted in volts relative to the particular reference electrode used. A fixed difference in potential is always established between a metal and a solution containing its ions at an activity level of unity. This potential difference is arbitrarily taken to be zero for the hydrogen electrode which is used as a reference for all metal potentials.

The difference  $(E_c - E_a)$ , between the potentials of the cathode,  $E_c$ , and the anode,  $E_a$ , can be very small if the anode and the cathode are quite close to each other, and the electrolyte conductivity is high and can attain a voltage of several hundreds of millivolts when the resistance of the medium is high. It should be noted that the potential of the local anode is always less than that of the cathode in the corrosion pair.

When the solution in contact with the metal does not contain its ions, or if their activity is not unity, the Nerest equation allows the actual potential, E, to be determined as :

$$E = E^{\circ} - (RT/nF)\ln K \qquad (2.3)$$

where R is the gas constant, n the valence, and K the equilibrium constant for the ions present in the solution. Therefore, the actual potential of a corroding iron piece (known as its open
circuit potential, rest potential, or corrosion potential,  $E_{corr}$ ) is dependent on a variety of factors, including:

- . The equilibrium potentials of the anodic and cathodic half-cell reaction,
- . The composition of the surrounding electrolyte.
- . The temperature.
- . The polarization of the half cells, and
- . The existence of passivity.

# 2.3.3 Pourbaix Diagrams

The electrical potential versus pH diagrams, more commonly known as Pourbaix diagrams, are named after the originator of the concept, who devised a compact summary of the thermodynamic data for metals in aqueous environment in the form of these diagrams [Pourbaix (1974)]. Figure 2.3 shows a simplified Pourbaix diagram for iron. Based on equilibrium thermodynamics, these diagrams showing potential-pH plots define three main regions:

(1) Immunity, where the metal is thermodynamically stable and is immune to corrosion.

(2) Corrosion, where the ions of the metal are thermodynamically stable, and corrosion occurs at a rate which cannot be predicted thermodynamically.

(3) Passivity, where the compounds of the metal are thermodynamically stable, and may protect the substrate from further reactions with the environment.

The dotted lines of Fig. 2.3 represents the thermodynamically stable region of oxygen, which is above line a, of water between lines a and b, and of hydrogen which is below line b. The iron is in the passive state at a pH in the range of 8-13 as illustrated in Fig 2.3, however, it also shows that corrosion may begin if the pH is more than 13, where a soluble ferrite,  $HFeO_2^-$ , forms. However, the occurrence of this phenomenon in the concrete has not been confirmed [ACI 222R (1994)].

#### 2.3.4 Polarization of the Half Cells

Polarization is the shift of the measured potential away from the reversible potential when a current flows through the half-cell. This polarization represents an overpotential defined as [Rosenberg *et al.* (1991)]:

$$\eta_a = E_{corr} - E_a \tag{2.4a}$$

$$\eta_c = E_c - E_{corr} \tag{2.4b}$$

where  $\eta_a$ ,  $\eta_c$  are the overpotentials and  $E_a$ ,  $E_c$  are the equilibrium potentials for the anode and the cathode, respectively.

The polarization process will influence the shapes of the anodic and cathodic branches of the polarization curve. There are three kinds of polarization where they may act separately or simultaneously:

(1) Concentration polarization, which happens when the concentration of the electrolyte changes in the vicinity of the electrode, such as depletion of oxygen at the cathode.

(2) Ohmic polarization occurs because of the ohmic resistance of the electrolyte and of any films on the electrode surface. This produces an ohmic potential drop in accordance with the Ohm's law.

(3) Activation polarization occurs due to kinetic hindrance of the rate-controlling step of the electrode reaction. Tafel (1905) showed experimentally that the measured polarization is directly proportional to the logarithm of the current density, i, for large currents in the absence of concentration and ohmic polarization:

$$\eta = a + b \log i \tag{2.5}$$

where a is constant parameter known as the Tafel intercept

b is constant parameter known as the Tafel slope

These parameters can be obtained empirically by plotting  $\eta$  versus *i* on semilogarithmic scales. The parameter *a* is related to the exchange current  $i_{os}$  and is a measure of the reversibility of the reaction, while parameter *b* gives an insight into the mechanism of the electrode reaction.

## 2.3.5 Morphology of Corrosion Process

The mrophology of the corrosion process depends on the distribution of anodes and cathodes on the steel bar surface and on their relative areas. If the anodes and the cathodes are irregularly distributed on the steel surface and change their position during the corrosion process, the attack will be more or less uniform. However, if the anodes are located at fixed point and the anodes/cathode area ratio is very small, localized attack will develop.

The corrosion rate is considered to be cathodically controlled when the cathodic process is slower. Figure 2.4 represents the log of the absolute current, *I*, versus potential, *E*, by polarizing each half-cell and demonstrates the cathodic control as the cathode has larger polarization. The corrosion rate is considered to be anodically controlled when the anodic process is slower and is shown in Fig. 2.5. There are two types of corrosion-rate controlling mechanisms [ACI Committee 222 Report, (1994)]:

(1) The cathodic diffusion, where the rate of oxygen diffusion through the concrete determines the rate of corrosion.

(2) The development of a high resistance path, when there is a large distance between

the anodic and cathodic areas such as several feet apart, and hence, the resistance of the concrete may be of great importance.

#### 2.3.6 Passive Layer on Reinforcing Steel

The term "passivity" predates modern understanding of the protective film crystalline structures in solids. Passivity is provided by an insoluble layer formed on the metal surface which protects the metal against corrosion. It is made of chemical combinations of oxygen, though the exact composition has been difficult to determine. Cornet *et al.* (1968) by using the Pourbaix diagrams, explained how concrete with a pH of approximately 12.8, protects steei from corrosion.

The alkaline nature of the pore solution, and the oxygen within it, result in the formation of insoluble and adhering corrosion product which forms an extremely thin microfilm on the reinforcing steel bar surface. The environmental condition in the paste pore solution at the bar surface promote the stability of this passive oxide film. It should be noted that at this stage, the corrosion does not stop, however, it is severely slowed down by the ohmic resistance of the film. Rosenberg, Andrade (1978) and Hansson (1984) refer to their earlier works, and suggest that the passive corrosion rate of the steel embedded in the concrete is basically equivalent to the oxidation of approximately 0.1  $\mu$ m per year from the steel surface. This is negligibly small when compared with the anticipated service life of the concrete structures.

A relatively high pH and the presence of oxygen and moisture are essential for the maintenance of the passivity of the reinforcing steel embedded in the concrete. Ingress of chloride ions or carbon dioxide into the concrete causes a reduction in pH which in turn would lead to active corrosion. While the elimination of moisture inhibits or stops the corrosion process, the effect of elimination of oxygen is not straightforward. If oxygen is absent in the pore solution at the steel-concrete interface and if its pH is greater than 9, the corrosion process will continue, however, it will result in the evolution of hydrogen, instead of the

reduction of oxygen at the cathode [Rosenberg et al. (1989)]:

$$2H_{2}O + 2e^{-} - H_{2} + 2OH^{-}$$
 (2.6)

The corrosion rate under these conditions is an order higher ( about 1  $\mu$ m per year at the steel surface ) than for the passive conditions with the reduction of oxygen, however, these values are still extremely low and acceptable for most concrete structures. However, the evolution of hydrogen can lead to the embrittlement of the prestressing steel in both pretensioned and post-tensioned structures and result in their sudden and catastrophic failures. When passivity is lost, the corrosion rate of the steel reinforcement is inversely proportional to the resistivity of the concrete [Gjorv (1982) and Gonzalez and Andrade (1982)].

# 2.3.7 Availability of Oxygen

As mentioned earlier, the presence of oxygen is an essential factor for the corrosion of iron in concrete with the addition of chlorides and a reduced alkalinity. Also, the effect of elimination of oxygen is not straightforward. In the case of concrete saturated in water, the diffusion of oxygen is strongly effected by the degree of saturation. This effect is best demonstrated in the work of Griffin and Henry (1963) as shown in Fig. 2.6. The level of corrosion increased as the sodium chloride concentration increased until a maximum concentration was reached, beyond which the rate of corrosion decreased despite the increased chloride concentration. This is due to the reduced solubility and hence the availability of oxygen to sustain the corrosion process. Also, investigations by Shalon and Raphael (1959) indicate that the rate of steel corrosion is very slow even though chlorides are present if the concrete is continuously water-saturated.

# 2.3.8 Corrosion Initiation

The presence of chloride ions either in the concrete mix or due to ingress from the immediate environment, and a decrease of the hydrated cement paste pore solution pH because of the concrete carbonation are two known major causes of the breakdown of the passive layer on surface of the steel bar embedded in the concrete. These two effects can often be present simultaneously and have a stronger degrading effect on the reinforcing steel passivity. The consequences of the steel corrosion are manifested as a decrease in the bar diameter, deterioration of the mechanical properties of the reinforcing steel (e.g., the change from the normal ductile response of low carbon steel bars to a relatively brittle response in bars damaged by pitting corrosion), cracking and spalling of the concrete by the expansive iron oxides and hydroxides, and a noticeable decrease in the bond at the steel-concrete interface.

#### 2.3.8.1 Penetration of Chlorides

Stratful *et al.* (1975). Brown (1980) and several other researchers have shown that the presence of chloride ions in the concrete can cause corrosion of the reinforcing steel provided if the corrosion reaction can be sustained by an adequate supply of oxygen and moisture. Chloride ions have been known to be introduced in the concrete through the use of calcium chloride as an accelerating agent for the hydration of portland cements in several countries. Some water-reducing admixtures contain small amounts of calcium chloride to offset their set retarding properties. Use of seawater, or water containing chlorides for concrete mixing and the use of aggregate exposed to seawater can introduce considerable quantities of chlorides in the concrete mix. In addition, porous aggregates are more harmful as they can retain a significant amount of chlorides as compared with the non-porous aggregates.

The most important source of chlorides in the concrete in cold climate countries is basically from the use of deicing salts on pavements and bridge decks during the winter. The ice melt-salt mixture readily penetrates more or less dry concrete by diffusion through totally or partially water-filled pores to the interior of the concrete, or in the most rapid capillary suction which can cause penetration to the extent of a few millimetres in a few hours. This salt penetration occurs more slowly in wet or highly moist concrete by diffusion of the chloride ions through the hydrated cement paste pore solution which results from the gradient of the chloride ion concentration at the concrete surface and inside the concrete. The penetration rate depends on the chloride concentration gradient. Page *et al.* (1981) estimated the typical diffusion rates in fully saturated hydrated cement paste to be about  $10^{-12}$  meter<sup>2</sup> per second, which is so small that it would require several months for the chloride ions to penetrate a 10 mm thick layer of hydrated cement paste, showing the importance of a concrete cover of appropriate thickness and quality. Thus, permeability, water penetration depth and the cover thickness of the concrete are of great importance to the rate of penetration of chlorides.

The rate of corrosion is strongly influenced by the rate at which the chloride ions reach a critical concentration at the steel surface and the degree of binding of the chloride in the concrete [Rosenberg et al. (1989)], which influences the rate at which the chlorides reach the steel surface. It should be noted that not all the chlorides can attack the steel; only the freely dissolved chlorides in the cement pore solution can be involved in the corrosion process. It is generally believed that some chloride ions can react chemically with the calcium aluminate minerals in the cement gel to form "Friedel's salt" and therefore, these will not be available to attack the steel. Therefore, the tricalcium aluminate  $(C_1A)$  content of the cement has a strong influence on the amount of free chlorides remaining in the hydrated coment paste pore solution and readily available for the corrosion process. Byfors et al. (1986) and recent research have shown additional factors influencing the degree of chloride binding include the proportion of cement in the concrete, water-cement ratio, pore solution pH, presence of sulphate ions [Holden et al. (1986)] and the specific surface of the cement gel. Figure 2.7 shows the gradient of total chloride concentration which depends on whether chemical reaction occurs with the cement [Verbec (1975)]. Figure 2.8 shows that the amount of chlorides required for corrosion initiation increases as the pH at the iron-liquid interface increases [Erlin and Verbeck (1987)].

# 2.3.8.2 Mechanism of Chloride attack

The structure of the passive film on the steel bar surface and the mechanism of its breakdown are not fully understood and considerable research on this subject is underway presently. It is agreed that in general the chloride ions replace some of the oxygen in the passive film, thereby increasing both its conductivity and solubility. This results in the loss of the protective character of the film. Also, the "initial" high potential difference across the film cannot be maintained any more, and this potential at the steel surfaces decreases considerably to the value for an iron electrode.

Research and field experience has shown that chloride ions are rarely distributed uniformly over the steel surface. Similarly, the imperfections in the passive film which permit easy incorporation of the chloride ions are also nonuniformaly distributed, resulting in local breakdowns of the film and creation of macrogalvanic cells (Fig.2.2). These local active areas act as anodes where iron dissolves quite readily at a relatively low potential, while the remaining passive areas act as cathodes, where oxygen reduction takes place at a considerably higher potential (Rosenberg *et al.* 1989). They noted that in addition to the chloride ion concentration and availability of the oxygen, the rate of iron dissolution in the galvanic cells is dependent on the ratio of the cathode-anode areas and the electrical resistivity of the concrete between the cells.

The pattern of corrosion of the reinforcing steel is influenced strongly by the separation of the anodic and cathodic areas, resulting from the localized nature of the chloride attack. The concentration of positive iron ions increases in the vicinity of the anodic area along with a drop in pH at the anode, while negative hydroxyl ions are produced in the cathodic region and its vicinity. The pH decrease results in the chloride at the anode which is soluble and diffuses away from the anode, and results in the continuation of corrosion. At some distance from the anode, both the pH and the concentration of dissolved oxygen are higher than their values at the anode. This causes the iron chloride to breakdown and to form iron hydroxide with the release of the chloride ions which are free to react further with the iron ions at the anode. This process continues as the iron ions migrate further from the steel and combine with oxygen to form higher oxides or hydroxides. As a result, the corrosion process focusses at the local anodic area instead of spreading along the bar and results in the formation of deep pits and substantial local loss of cross sectional area. As this process continues, eventually the bar breaks down completely leading to its severance. The threshold chloride content below which there is low probability of corrosion is dependent on several factors [Rosenberg *et al.* (1989)]:

- . Concrete mix details.
- . Type of cement and its specific surface area.
- . Water-cement ratio.
- . Sulphate content.
- . Curing condition, age and environmental history of the concrete.
- . Carbonation of concrete.
- . Concrete environment ( temperature and relative humidity ).
- . Roughness and cleanliness of the reinforcing steel bar.

The CEB (1985) has reviewed the influence of some of these factors on the threshold chloride level which are presented in Fig. 2.9. Depending on the local experience, different threshold limits have been adapted by the different national standards. The CEB and RILEM have considered the value of 0.4 percent by weight of cement to be appropriate. The Federal Highway Administration of the U.S.A. consider a chloride ion limit of 0.15 percent by weight of cement to be tolerable, while a limit of 0.3 percent is considered dangerous.

# 2.3.8.3 Carbonation of Concrete

The carbon dioxide,  $CO_2$ , in the air can penetrates slowly from the surface of the concrete through the pores of the concrete to the interior. This carbon dioxide can react with the hydration products in the hydrated cement paste (hcp), especially with Ca(OH)<sub>2</sub> which is the cause of high pH in the concrete, and with other ions in the pore solution as follows :

$$Ca(OH)_2 + CO_2 - CaCO_3 + H_2O$$
 (2.7a)

$$2NaOH + CO_2 - Na_2CO_3 + H_2O$$
 (2.7b)

The CaCO<sub>3</sub> and Na<sub>2</sub>CO<sub>3</sub> precipitate in the pore of the hep, thereby decreasing its permeability and also reducing pH of the pore solution. It should be noted that the permeability of the concrete plays an important role with respect to the rate of diffusion of CO<sub>2</sub> as it can travel through the air-filled pores of the concrete, and hence a totally saturated concrete will not carbonate. This reaction continues until the pore solution is neutralized. If carbonation continues beyond this stage, then carbon dioxide reacts with the other hydration products resulting in the formation of amorphous SiO<sub>2</sub>, Al(OH)<sub>3</sub>, CaSO<sub>4</sub>, CaCO<sub>3</sub> and water.

It should be noted that the change in the value of pH due to carbonation is very sudden and it appears as a narrow zone or carbonation front separating the two zones - one towards the concrete surface with pH values less than \$, and the other into the concrete core with pH values larger than 12. The carbonation rate is dependent on the cement type, water-cement ratio, cement proportion in the concrete mix, and others- basically constituting the concrete cover quality or the cover permeability. In most cases, the rate of carbonation or the depth of carbonation from the concrete surface, x, is proportional to the square root of exposure time, t, i.e.

$$x = k\sqrt{t} \tag{2.8}$$

The water saturation level of the pores in the hcp also influence the carbonation rate because  $CO_2$  can permeate into the concrete rapidly in the gaseous phase, however, the carbonation reaction takes place in the liquid phase (Rosenberg *et al.* 1989). Carbon dioxide cannot react with  $Ca(OH)_2$  in a completely dry concrete, because of the absence of water in the pores. By contrast, in a completely water saturated concrete, carbon dioxide must first dissolve in the solution and then diffuse through the pores to reach the alkaline substance before the carbonation reaction can occur. However, it should be emphasized that at intermediate humidity levels when pores have moisture on the walls but are not completely saturated with relative humidity levels of 50 to 80 percent, carbon dioxide can rapidly reach the pore walls and have enough water present for the carbonation reaction to proceed.

The carbonation front advances with time through the concrete cover and it reaches the level of the reinforcement. The passive film becomes unstable and active corrosion commences. This corrosion is generalized and homogeneous, and it will result in a reduction in the cross-sectional area and formation of a considerable amount of oxides which can either crack the concrete cover, or it may diffuse to the concrete surface through the pores. Occurrence of alternate semidry and wet cycles constitutes the most aggressive environment related to the neutralization of the concrete. The carbonation front advances through the semidry period, while the steel corrodes during the wet periods. If the carbonation front reaches the steel in a permanently dry environment, the steel will get depassivated, however, no significant corrosion will occur. By contrast, no carbonation will occur in a permanently wet condition and the steel will remain passive provided that no other depassivating agent, such as chloride ions, is present.

Field examinations have shown that progress of the carbonation front is slow in sound and dense of concrete, and the expected service life gets reduced because of the cover being too thin and the existence of cracks. The concrete surfaces at the crack carbonate leading to depassivation of the steel in the vicinity of the crack. With crack width larger than 0.4 mm, no correlation has been observed between the crack width and the amount of corrosion. The risk of corrosion is a function of the cover quality (thickness and impermeability) and of the cover/bar diameter ratio. In general, a corrosion risk is always present, however, for crack width less than 0.4 mm, the risk is independent of the crack width.

# 2.3.9 Stray Current Corrosion

Stray current corrosion is caused by the discharge of stray direct current from the surface of a metal, thus they are brought into action by the flow of external current from the environment. The origin of this current could be natural, by a variation in the earth's magnetic field, or manmade sources such as electric railways, electroplating plants and cathodic protection systems. Stray electrical currants can greatly accelerate the corrosion of reinforcing steel.

## 2.3.10 Stress Corrosion Cracking and Hydrogen Embrittlement

Stress corrosion is caused by the combination of stress and corrosion. Prestressing steel cables are susceptible to this type of attack and it usually occurs in nonalkaline environments in which the cathodic half-cell reaction results in the development of hydrogen which rapidly penetrates and embrittles the steel. A small imperfection caused by corrosion in stressed steel can lead to a serious loss in its tensile strength, and especially the stress corrosion and hydrogen embrittlement attacks are characterized by their unexpected and often catastrophic consequence.



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Fig. 2.1: The relative volumes of iron and its corrosion reaction products, [Nielsen (1985)]







Fig. 2.3 : Pourbaix diagram (potential-pH) for iron [Pourbaix (1974)].







Fig. 2.5: Potential vs current plots for systems under anodic control.



Fig. 2.6 : Effect of concentration of sodium chloride on corrosion rate [Griffin and Henry (1963)]



Fig. 2.7: Chloride content vs pH, [Verbeck (1975)]



Fig. 2.8: Chloride concentration vs depth of cover, [Erlin and Verbeck (1987)]



Fig. 2.9: The critical chloride content [CEB recommendations (1985)].

# Chapter 3

# **Bond Mechanisms**

The behaviour of a reinforced concrete member is influenced to a considerable degree by the bond between the concrete and the reinforcing steel, and cracking. This chapter presents some basic information on bond between the concrete and the reinforcing steel, and the associated slip and cracking in reinforced concrete. The mechanics of slip of deformed bars versus plain bars in concrete is discussed. The stress redistribution and deformations in concrete associated with the bond are also presented. A summery of the program planned to study the parameters which are influenced by corrosion is also presented.

# 3.1 Introduction

The external load is very seldom applied directly to the reinforcing steel which receives its share of the load through the surrounding concrete. Thus, an effective reinforced concrete member must have a positive interaction between the steel bar and the surrounding concrete in order to obtain a transfer of stress between the two materials. This phenomenon is fundamental because it influences many aspects of the behaviour of reinforced concrete such as cracking, deformability, instability and others. The transfer of the load between the steel and the concrete is affected by the phenomenon of bond at the steel-concrete interface, which ensures secure gripping of the reinforcement, and the working of the reinforcing steel in conjunction with the concrete, to form a reliable structural element, capable of withstanding both tension and compression forces. The bond is affected by many factors such as change in temperature, variation in the loading of a member, creep in the concrete, corrosion, etc., however, the bond must be capable of adjusting to any alterations of the above influencing phenomena. As control of crack width and deflection is one of the most important requirements for serviceability and the quality of structures, considerable research and development has been undertaken on bond, tension stiffening and crack width control. However, little has been done to evaluate the effect of corrosion on the bond behaviour and tension stiffening. Corrosion in concrete reduces the durability of concrete structures. The consequence of corrosion is probably more serious in the deterioration of bond between the concrete and the reinforcing steel due to rusting than in a reduction of the load carrying capacity of the steel bars due to a decrease in the cross-sectional area, unless the steel bars are very small.

As concrete is relatively weak and brittle in tension, cracking is expected when significant tensile stress is induced in a reinforced member. Cracking is an important phenomenon specific to reinforced concrete, and it can have a great influence on the durability of a structure. The load carried by concrete prior to cracking is transferred to the reinforcement crossing the crack, and the load carried across the crack by the reinforcement is gradually transferred by bond to the concrete on each side of the crack. Thus, the original specimen transforms into blocks of varying lengths separated by tension cracks, and linked by the reinforcement. Figure 3.1 illustrates the formation of the cracks, and the load sharing between the reinforcement and the concrete.

# **3.2 Fundamentals of Bond**

# **3.2.1 Basic Definition**

As defined earlier, "bond stress" is the shear stress at the bar-concrete interface which modifies the steel stress along the length of the bar by transferring the load between bar and the surrounding concrete [ACI Committee 408 (1966)]. Bond stress is calculated as the nominal shear force per unit area of the bar surface. There are two important aspects to the development of bond stresses which are the anchorage or development type, and the change of bar force along its length due to a change in the bending moment along the member (flexural bond in flexural members). Actually, the bond forces are measured by the rate of change in the force of reinforcing bars.

# **3.2.2 Bond Resistance**

Bond of plain bars depends on the steel surface-to-concrete bond which consists of chemical adhesion and friction between the mortar and the bar surface. The resisting mechanism for small loads and stresses is the chemical adhesion, however, even low stresses will cause a slip capable of breaking the adhesion between the concrete and the reinforcing steel. Bond is also developed through friction and by mechanical interlocking due to the roughness of the bar surface. Bond failure with plain bars is commonly due to the bar simply being pulled out of the concrete with little resulting damage.

However, bond capacity increases significantly with the use of deformed bars basically due to the interlocking of the ribs with the surrounding concrete. The transfer of the force from the bar to the surrounding concrete occurs by the mechanical interaction between the two materials, which is the primary bond mechanism. There are three mechanisms for the bond at the steel-concrete interface. The bond strength developed between two ribs of a deformed bar, as shown in Figure 3.2, is a combination of the following stresses:

(a) Shear stress,  $v_{a}$ , due to adhesion around the surface of the bar.

(b) Shear stress,  $v_c$  in the tangential direction resulting from the radial compressive stresses due to bearing of the lugs which is acting on the cylindrical concrete surface between adjacent ribs.

(c) Bearing stresses,  $f_b$ , against the face of the rib.

#### **3.2.3 Bond Failure Modes**

Typically, there are two types of failure under monotonically increasing loads. The first mode is a direct pullout of the bar, although, in tension tests where adequate embedment length with an adequate concrete cover and standard deformed bars are provided, it is not possible to produce a bond pullout failure, rather the bar will fracture at its loaded end. In a pullout failure, the surrounding concrete fails due to the shearing between the ribs, thus this failure depends mainly on the concrete strength and the profile and geometry of the ribs. The second mode of failure is a splitting of the concrete cover when the surrounding concrete cannot sustain the circumferential tensile stresses. The actions at the bar lugs consist of compressive forces normal to the deformation surfaces which in turn cause tensile forces on the concrete whose radial components are similar to the bursting pressures in pipes which cause longitudinal tensile splitting of the concrete cover and the component parallel to the bar axis assists with the transfer of force between the concrete and the reinforcing steel.

# 3.2.4 Bond Mechanisms

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With deformed bars, bearing against the lugs is considered to be the primary load transfer mechanism at high loads. This does not mean friction and adhesion are negligible with deformed bars, but they are secondary.

For relatively small loads, the basic resisting mechanism is the chemical adhesion, however, as the load is increased the chemical adhesion along the bar surface is lost quickly. Actually, it has been assumed that adhesion can break down due to the action of the service loads, or due to shrinkage of the concrete. After the chemical adhesion is destroyed, some frictional slip occurs before the full bearing capacity at the ribs is mobilized. Based on the work of Treece and Jirsa (1989), the ACI Committee 408 (1994) suggested that friction can contribute up to 35% of the ultimate strength governed by the splitting of the concrete cover.

The other force transfer mechanism is due to the mechanical interlocking between the ribs and the concrete keys. As the ultimate bond strength is reached, shear cracks begin to form in the concrete between the ribs as the interlocking forces induce large bearing stresses around the ribs, and slip occurs. Therefore, the bar ribs restrain the slip movement by bearing against the concrete keys. The slip of a deformed bar may occur in two ways, either through pushing the concrete away from the bar by the ribs, i.e. wedging action, or through crushing of the concrete by the ribs.

## 3.2.5 Bar Profile Geometry

Due to the importance of the mechanical interlocking for superior bond characteristics, the geometry of the lugs or the bar rib is of a great importance to the wedging action of these ribs. Based on Rehm's research work (1968), Park and Paulay (1975) have shown that the best performance of a bar embedded in concrete over a short length 'c' which is the rib spacing, occurs for a value of the ratio of the bearing area to the shearing area, a/c, where a is the rib height (see Fig. 3.2), equal to 0.065. Figure 3.3 illustrates the two types of failure mechanism, associated with the geometric shape and size of the ribs. The deformation requirements of ASTM A 615-72 (1972), give the following range of values for the a/c ratio:

# 0.057 < a/c < 0.072.

With reference to the steep face angle in Fig. 3.3, Lutz and Gergely (1967), stated that the bond of deformed bars is developed mainly by the bearing pressure of the bar ribs against the concrete. Pullout tests by Rehm (1957) and Lutz (1966) showed that for bars with steep rib face angle  $\alpha$ , (larger than about 40 degrees with the bar axis) slip occurs only by the compression of the concrete in front of the bar rib, while in bars with flat ribs, i.e., the angle  $\alpha$  is small, slip occurs with the ribs sliding relative to the concrete as the rib tends to push the concrete away from the bar. This wedging action can be a major cause of longitudinal splitting along the bar. For  $45^\circ < \alpha > 70^\circ$ , the deformations must reverse in direction on each side of the bar, [ASTM A 615-72 (1972)].

#### 3.2.6 Cracking Behaviour

Cracking occurs in the concrete when the tensile stress at a given location exceeds its tensile strength and it is manifested by a separation of the concrete at this location. In plain bars, this separation between steel and concrete leads to complete loss of bond in the vicinity of the crack. Mathey and Watstein (1959) found that the crack width at the bar is nearly the same as the width at the surface of the concrete. However, in the case of deformed reinforcing bars, separation does not produce complete unloading and bond forces are transmitted solely by the rib bearing in the vicinity of a main crack, as demonstrated in Fig. 3.4, on the study of the nature of cracking around a deformed bar Goto (1971). In referring to Goto's study (1971), Beeby (1979) suggested that in the development of a crack, crack forms with a minimal width at the bar surface initially. Further loading causes loss of adhesion adjacent to the crack. Further loading causes more internal cracks to form at successively greater distances from the main crack. Steel stresses will reach a local peak at the crack, but between the cracks, the steel stress is lower due to the concrete contribution. The transfer of forces produces bond stresses.

The circumferential tensile stresses in the concrete at the crack which are similar to the bursting forces in the concrete expand the circumference of the concrete. These stresses may lead to longitudinal cracks which can cause bond failure. Using finite element studies of experimental specimens, Lutz and Gergely (1967) determined the approximate magnitudes of these stresses. Tepfers (1973) found that the inclined concrete compressive forces or 'struts' that resist the bar movement are equilibrated by hoop tension within the surrounding concrete. As mentioned earlier, bond failure occurs when the hoop tension exceeds the tensile capacity of the concrete. When this happens, longitudinal cracking develops and, since the force in the 'struts' can no longer be balanced, failure occurs, with the splitting of the concrete cover and the bar pulls out (See Fig. 3.5).

#### 3.2.6.1 Crack Spacing

The stress in the concrete at the crack surface immediately after it is formed is zero. The surface stresses in the concrete increase with an increasing distance from the crack until the redistribution of the stresses is complete at some distance equal to the transfer length,  $S_{\mu\nu}$ , where the stress distribution is not influenced by the crack. Within the distance  $\pm S_{\mu\nu}$  of the crack, the concrete surface stress is reduced below the tensile strength of the concrete, thus the next crack must form outside this region. Thus, the minimum crack spacing is  $S_{\mu\nu}$ , and the maximum spacing would be twice that distance. Therefore, the mean crack spacing,  $S_{\mu\nu}$ , is normally in the range:

$$S_o \leq S_m \leq 2S_o \tag{3.1}$$

# 3.2.6.2 Crack Width

The elongation of the concrete after cracking is due the widening of the crack and the formation of new cracks. Hence, the mean crack,  $w_{m}$ , width is given by the mean crack spacing multiplied by the mean strain,  $C_{m}$ , minus the mean residual surface strain,  $C_{cm}$ , in the concrete between cracks:

$$w_m = S_m(\epsilon_m - \epsilon_{cm}) \tag{3.2a}$$

The small elastic strain in the concrete between the cracks,  $C_{em}$ , is usually ignored. Therefore, the relationship of the crack width and the strain of the member results in:

$$w_m = S_m \epsilon_m$$
 (3.2b)

Base et al. (1966) have stated that bond failure or slip will cause a further reduction in stress, increasing the value of  $S_o$ . Thus, the mean crack width increases as the mean spacing

increases. Goto (1971) showed that the effect of the loss of adhesion and the formation of internal cracks is to reduce the rate at which force is transferred from the reinforcement to the surrounding concrete, and hence it increases the distance from the crack over which the surface stresses are reduced. Therefore, the spacing  $S_{\mu}$  is very much influenced by the internal failure of the surrounding concrete, and therefore it influences the mean crack width.

# 3.3 Precracking Bond Response

Figure 3.6 illustrates a loaded tension member with a single bar and the distribution of stresses before cracking, steel stress,  $f_s$ , bond stress, u, and the concrete stress,  $f_c$ . The bond forces across the ribs need to be transferred to the surrounding concrete to enable the full strength of the bar to be developed. Bond performance is influenced by the fluctuation of the stress in the surrounding concrete, and hence bond stress is zero within the 'B' region of Fig. 3.6. At low load levels, before any tensile cracks develop, the axial stresses are uniformly distributed over the cross section except at the end sections, where it is labelled the 'C' region, where there is a disturbed flow of stresses. Also the strain of the concrete and the steel is the same and constant over the length. As described earlier at very small loads, the bar adheres chemically to the concrete within the 'C' regions which is the tension adhesion region. This chemical adhesion mechanism acts until the initial movement of the steel bar occurs.

# 3.4 Postcracking Bond Response

As the applied load is increased, the deformed bar tends to move or slip in the region 'C'. The disturbed flow of stresses, and the stresses in the surrounding concrete lead to cracking and deformations in the concrete in the 'B' region, where the bond stress is zero. As soon as a main crack forms, the bond stresses at the crack reduces immediately to zero and the tensile force that existed in the concrete at the cracked section is transferred to the steel. Figure 3.7

illustrates the redistribution of stresses after cracking occurs in a reinforced member, as steel stress,  $f_s$ , bond stress, u, and concrete stress,  $f_c$ . This unloading, that caused the interfacial tensile stresses between the steel and the concrete, will break down the adhesion mechanism and tends to pull the concrete away from the steel allowing the crack to open. The surrounding concrete separates, and internal secondary cracks form at this stage. With increasing loads, the concrete cracks when the tensile force applied to the concrete section at a certain distance from the ends, reaches a level where the concrete is overstressed in tension. However, new cracks keep developing as long as the length of the concrete specimen between these two newly formed cracks is longer than the stable crack spacing.

Figure 3.8 demonstrate the two types of cracking in the tension specimen. The longitudinal splitting crack, which passes through a plane through the axis of the reinforcing bar, occurs in the 'C' region and the transverse tensile crack that crosses the axis of the reinforcing bar occurs in the 'B' region. When transverse crack forms, the stresses redistribute forming new 'C' and 'B' regions. When a longitudinal splitting crack occurs, the bond stress decreases and the transfer length in the 'C' region increases and extends into the 'B' region.

The type of cracking in a reinforced concrete member depends on the bond characteristics between the concrete and the steel reinforcement, and the concrete tensile strength.

In light of the above discussion, it can be stated that the bond between the concrete and the reinforcing steel would be quite significantly affected by the corrosion of the steel reinforcement. Therefore, all of the parameters which influence corrosion would need to be considered in any evaluation of bond of the corroded reinforcing steel. Nevertheless, the study of the parameters which are influenced by corrosion is determined in the following manner:

# 3.5 Response of Concrete Tension Specimens

#### 3.5.1 Purpose of Tension Tests

In a number of modern codes, such as the CEB-MC90 (1990), the crack width is controlled by defining an effective tension area. In addition, the bond stress is important in the response of the concrete tension specimens. The present research has focussed on investigation of pure tension in reinforced concrete structures, to study the bond and the interaction between the reinforcement and the concrete under tensile stresses.

Many experimental research programs have been undertaken using the tension specimen, for example, by Broms and Raab (1961), Houde and Mirza (1972). Goto and Otsuka(1979) and others. It should be noted that cracks forms in a tension specimen with increasing tensile stresses; this action will be reflected realistically in concrete beams, except for the conditions at the beam end, where the force in the bar is transferred to the concrete as in the pull-out tests. In a pull-out test the concrete is in compression at the loaded end while the steel is in tension, which eliminates the transverse tension cracking. In the case of pure tension tests, both the concrete and the reinforcing steel are in tension, and thus the pure tension specimen represents a simplified model of the tension zone of a reinforced concrete beam.

In order to study the effect of corrosion on bond behaviour, the basic responses of uncorroded specimens and corroded bars in tension specimens will be compared. The purpose of this investigation is to study the influence of various stages of corrosion on tension stiffening and cracking, and the influence of the pre-existing splitting due to corrosion. This kind of longitudinal splitting in uncorroded reinforced concrete is critical as it causes bond failure, unless an adequate confinement is provided. Both splitting cracks and transverse tensile cracks are studied.

The complete response of this series of tests will be achieved by studying the following:

- (a) load-deflection response
- (b) stress-strain relationship
- (c) cracking behaviour

#### 3.5.2 Load-Deflection Responses

The bond behaviour between the concrete and the reinforcing steel in a reinforced concrete member can be predicted if the load-deformation relationship of that element is known.

Unlike plain concrete, reinforced concrete can still resist some tensile stresses after a full crack is formed. The development of the stresses in the concrete between the cracks is dependent on the bond characteristics between the reinforcing steel and the concrete. The phenomenon of concrete contribution between cracks to the net stiffness of a member is called "tension stiffening" and plays a significant role in reducing the postcracking deformation in reinforced concrete structures. Figure 3.9 illustrates the tension stiffening through the presence of tensile stresses in the concrete that reduces the average axial member deformation, by considering the relationship between the load and the average strain in the concrete both before and after cracking. The ascending part 1 represents the uncracked stage, the horizontal part 2 represents the crack formation stage, and the ascending part 3 represents the stabilized cracking stage.

#### 3.5.3 Steel Stress-Strain Relationship

In order to help the understanding and interpretation of the results from the load-deflection responses, and to formulate the tension stiffening effect, it is necessary to develop a modified stress-strain relationship of the specimen.

A brief review of the change in the bar size and its cross-sectional area due to corrosion is presented here. From the load-deflection response curves, the yield load for each specimen is compared with the yield load of the control specimen, to determine an equivalent crosssectional area using the equation:

$$T_{(uc)s} / A_{(uc)s} = T_{(c)s} / A_{(c)s}$$
(3.3)

where

 $T_{toris}$  = yield load of the control specimen.  $A_{turos}$  = area of the steel bar in the specimen (20M bar)  $T_{tors}$  = yield load of the corroded specimen  $A_{tors}$  = new (equivalent) area of the bar after corrosion

where the subscript 'uc' relates to the uncorroded specimen and the subscript 'c' stands for the corroded specimen.

After obtaining the new (equivalent) cross-sectional area of the corroded steel, stressstrain relationships are obtained for the corroded rebars for each level of corrosion. Also, a control specimen is included in each stress-strain curve for comparison purposes. The difference in the elongation between the bare bar and the "real" intermediate behaviour represents the real contribution of the concrete in tension.

# 3.5.4 Cracking Behaviour

There are variations in the spacing of the cracks, mainly due to the bond characteristics of the reinforcing bar, the difference in the tensile strength of the concrete, and the proximity of previous primary cracks, where the local tensile stress tends to decrease. The ACI Committee 224.2R-3 (1994) report suggests that for the normal range of concrete covers, 30 to 75 mm, the average crack spacing will not reach the limiting value of twice the cover until the reinforcement stress reaches a value of 138 to 200 MPa. Because of the variability in the concrete tensile strength along the length of a tension member, cracks do not all form at the

same stress level. Also, the bar ribs tend to control the crack width by limiting the slip between the concrete and the steel.

The maximum crack widths versus the steel stress at the crack location for each specimen are obtained and compared with the control specimen, along with the crack spacings.

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(c) Cracks fully developed  $T_3 > T_2$ 





Fig.3.2: The stresses between two ribs of a deformed bar [Park and Paulay (1975)].



Fig.3.3: Failure mechanisms at the ribs of deformed bars [Park and Paulay (1975)].



Fig.3.4: Formation of internal cracks [Goto (1971)].



Fig.3.5: Tensile stress rings [Tepfers (1973)].



Figure 3.6: Stresses acting on concrete and steel before cracking, [Abrishami (1994)]



Figure 3.7: Distribution of steel stress, bond stress and concrete stress in a tension specimen, [CEB Manual (1985)]



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Figure 3.8: Splitting and transverse crack propagation in a tension specimen



Figure 3.9: Influence of tension in concrete on load-deformation response, [Collins and Mitchell (1991)]


# Chapter 4

# Accelerated Corrosion Testing

This chapter describes the test program to simulate the prevalent conditions of severe local corrosion conditions which cause significant changes in the surface conditions of the bar and the heights of the ribs. The criteria to establish the different levels of corrosion for different bar diameters, the method adopted to evaluate the corrosion rate, and the determination of the chloride-ion concentration along the concrete cover depth are summarised.

# 4.1 Introduction

The various service and field corrosion tests normally require long exposure durations. Therefore, for design of structural components for durability against corrosion, and selection of suitable materials and their protective systems, it is essential to use accelerated corrosion tests to obtain qualitative information on their corrosion behaviour. This data is especially useful in situations where the member or system service life is endangered by the severe corrosive environment. Several electrochemical test procedures have been developed, including some which have been standardized, and are available in the literature. Some of these methods such as the anodic polarization tests utilize closed circuit testing while others rely on the chemical exposure in an open circuit. Accelerated corrosion test have been used successfully to determine the susceptibility of the reinforcing and other forms of structural steel to localized attacks such as pitting corrosion, stress corrosion and other general forms of corrosion. These tests provide qualitative data which provides the designer with relative material performance indicators for the various localized forms of corrosion.

It is important that the accelerated testing conditions do not differ significantly from the service environment except for the time scale, otherwise the corrosion products from the accelerated tests could differ considerably from the products developed during the normal corrosion activity. Also, the accelerated testing could lead to misleading results [Biczok, (1972)]. Acceleration of corrosion activity in these tests can be achieved by suitable increases in temperature, applied voltage or current, and an increased concentration of the corrosive agent.

As mentioned earlier, an essential requirement of the accelerated corrosion testing is to simulate the corrosion conditions in the system under the "normal" environmental conditions. As mentioned earlier, corrosion of the reinforcing steel embedded in the concrete occurs by the electrochemical process, similar to that in a flashlight battery, with the formation of an anode where electrochemical oxidation takes place, and a cathode where electrochemical reduction occurs, and an electrolyte capable of conducting the ionic current.

## 4.2 Experimental Procedure

### 4.2.1 Test Apparatus

The test apparatus consisted of the following:

- 1. One tank, 2m in length by 1m in width, and 1m in height.
- 2. Power supply, 10 Volt DC.
- 3. Voltmeter (HICKOK Digital System Model DP100).
- 4. Ammeter (MICRONTA 10mA range).
- 5. A reference saturated calomel electrodes (SCE).
- 6. Electric wires and clips.
- 7. Half-Cell potential probe.

#### 4.2.2 Specimen Preparation

Fourteen concrete cylindrical specimens, 1000 mm long by 100 mm diameter, each specimen reinforced with a pre-weighed single 20M reinforcing deformed bar and with a concrete cover of approximately 40 mm were cast. Figure 4.1 shows the geometry and instrumentation for a typical tension specimen.

The concrete specimens were cast in specially designed plastic molds which had two sockets affixed to each top and bottom cover to allow the steel bars to pass. The reinforcing bar extended 150 mm outside the ends of the concrete. Just outside the concrete, an electrical wire (about 1 m in length) was connected to the steel bar by means of clips on each of the 12 specimens. The remaining two specimens were used as control specimens (uncorroded conditions). The extensions of the bars plus another length of 20 mm near the specimen end, were epoxy coated and then covered with two layers of tape, first with an electrical tape followed by a duct tape, in an attempt to protect this part of steel from corrosion. A silicon layer was added on top of the electrical wire and the clip for the same reason. Each specimen was marked with a number and a grid, 10 mm x 10 mm, was drawn on the spacing to be used for the measurements prior to immersion.

The tension specimens along with the accompanying auxiliary specimens, cylinders and flexure beam specimens were air cured in the plastic molds for 48 hours, after which they were removed from the mold and cured for 28 days before positioning them in the exposure tank. It was suggested by Biczok, (1972) that the concrete exposed at an early age has less resistance to chemical attack than the concrete hardened over an extended period. Therefore, the concrete specimens, cylinders and fluxure beams were cured for 28 days to simulate the natural corrosion process.

#### 4.2.3 Concrete Mix Parameters

The concrete mixture was provided by a local redy-mixed concrete supplier; the cement used

was ordinary Type 10 Portland cement. The coarse aggregate was 20 mm maximum-size crushed limestone with a bulk specific gravity of 2.69 and an average absorption coefficient of 0.95 percent. The fine aggregate was St Gabrial sand with a specific gravity of 2.68 and an average absorption coefficient of 0.58 percent. A fine-to-coarse aggregate ratio of 1.0 and a water/cement ratio of 0.45 without any chloride additives were used. A sieve analysis of both the fine and coarse aggregates, and gradation limits were provided by the concrete company. These tests were conducted according to the CSA Standard A23.2-2A Sieve Analysis of Fine and Coarse Aggregate.

The six standard cylinders, with a height of 300 mm and a diameter of 150 mm, were used to obtain the concrete compressive strength,  $f_{c}$  and the splitting tensile strength,  $f_{sp}$ . Flexural tests on three specimens of 100 mm by 100 mm by 400 mm long beams subjected to the thirdpoint loading over a span of 300 mm were used to obtain the modulus of rupture,  $f_r$ . The procedure used and the test results are presented in Chapter 5.

#### 4.2.4 Experimental set-up

As described previously, two detailed investigations dealing with accelerated electrochemical corrosion of reinforced concrete were completed at McGill University (Palumbo 1991, Farah 1993). They developed the test set-up and the detailed procedure for the accelerated corrosion testing using small lollipop specimens. The same electrochemical method was adopted, and after a few trials, a corrosion cell was developed with some modifications, which was different due to the size of specimens.

Once the tension specimens were properly cast and cured, 12 specimens were placed in the tank. Figure 4.2 illustrates the schematic drawing of the set-up. The tank was filled with an electrolytic solution [5% sodium chloride (NaCl) by weight of water]. The tank was filled to cover the top of the specimens by approximately 50 mm. The electrolyte solution was changed on a weekly basis to eliminate any change in the concentration of the NaCl.

A copper electrical wire was soldered and connected by stainless steel anchors to the reinforcing bar of each specimen. These wires were permitted to protrude from the specimen surface and were used to make electrical connections to the circuit of the corrosion cell. Once the specimens were properly cured, they were immersed in the tank, and placed at a clear distance of 50 mm apart from each other by means of wooden spacers. The wires were then connected to a power supply with an in-built voltmeter within the electrical circuit required to impress the necessary voltage. To close the circuit, a steel bar was placed in the tank, and a reference saturated calomel electrode (SCE) was also placed in the electrolyte solution to determine the potentials of both electrodes. The SCE consisted of a mercury/mercury chloride reference element surrounded by saturated potassium chloride (KCl) contained in the outer tube. When immersed in the NaCl solution, electrical contact was made between the sample and the electrolyte solution at the opening located at the end of the electrode. This opening formed a conductive bridge between the reference electrode, the sample, and the indicating electrode. A refill hole is a small circular opening located at the top of the reference electrode. where the spout of the re-fill bottle is inserted. The electrode was periodically refilled with KCI solution to ensure that the electrolyte always covered the tip of the internal element, as the level of the KCl electrolyte had to be maintained above the surface of the aqueous solution, to ensure that the NaCl solution will not flow back into the electrode.

The electrodes and the tension specimens were placed under water and the direction of the current was arranged so that the reinforcing bar served as the anode while the steel bar served as the cathode. A constant voltage of 1 V was applied as in the previous research programs to stimulate corrosion, (Palumbo 1991, and Farah 1993) in lollipop samples. It was soon discovered that one volt was not enough, therefore, after six weeks, the voltage was increased to 2 V. After a few weeks, the current and the potential readings did not change. Consequently, the voltage was changed again to 5 V, and the voltage and the current were monitored on a daily basis and the readings of both the current and the potential were recorded.

For connection, a jumper for each specimen wire was used to facilitate the connection

and measurements. The jumper was connected to the positive terminal of a direct current power supply and it was used as a switch, to open or close the electrical circuit. The negative terminal was connected to the bare bar electrode by a different electrical wire; the schematic drawing of the arrangement is shown in Fig. 4.3.

### 4.3 Measurements

### 4.3.1 Current and potential

The current and the potential measurements for each specimen were recorded periodically (every 48 hours) by means of a SMART Digital Multimeter that read both the current and the voltage.

The current readings were recorded by removing the jumper and then connecting the positive terminal probe of the ammeter to the specimen (anode), while the negative terminal probe was connected to the bare steel bar (cathode). When stable readings were attained (approximately 5 to 10 minutes), the current readings were recorded.

The potential voltages were recorded by having the refill hole of the electrode (SCE) open, to permit proper electrolyte flow. The positive terminal-probe of the voltmeter was connected to the specimen (anode), while the negative-terminal probe was connected to the SCE. The measurements were recorded when stable reading were attained (approximately 5 to 10 minutes). The positive-terminal probe was connected to the bare electrode (cathode), and the negative- terminal probe was connected to the SCE, to measure the cathode potential. Both steps were implemented in the normal polarity position.

#### 4.3.2 Half-Cell Potentials

The standard test method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete and the principle of the Copper/Copper Sulphate electrode measurement technique is described in ASTM C 876-80. The Half-Cell test is the estimation of the electrical half-cell potential of uncoated reinforcing steel, for determining the corrosion activity of the reinforcing steel.

To measure the half-cell potential, the specimen was removed from the tank. A direct electrical connection to the reinforcing steel was achieved by clamping it to the reference cable, and then the reinforcing steel was connected electrically to the positive terminal of the Digital Half Cell (DHC) voltmeter. The sponge head was moistened with clean, slightly soapy water, and then the solid cap was replaced on the cell. The DHC was placed on each position of the 10 mm x 10 mm grid that was drawn on the specimen. Readings were allowed to stabilize within 10mV and recorded.

# 4.4 Testing for Chlorides

The chloride content of concrete is critical to the life of the reinforcement, because small amounts of chloride can disrupt the passive layer that protects the steel from corrosion. Therefore, to assess the condition of the deteriorating concrete, it is essential to determine the chloride content in the concrete.

Samples were obtained from each specimen at three different locations by drilling three holes in the concrete and collecting the dust produced across the diameter of a specified location, according to the crack width:

- Around the widest crack width
- Around the intermediate crack width
- Around the smallest crack width

The concrete powdered was collected from three different depths (15 mm, 30 mm, 40 mm), to determine how the chloride content changed with depth from the surface. The concrete dust was collected at each location, in small plastic bags and properly scaled and marked.

#### 4.4.1 Chloride Ion Analysis Procedure

The chloride ion content of each powder sample was determined using the Volhard Method of the British Standards 1881 (1988) - Part 124. The following procedure was used. The 2.5 g of the powdered concrete sample was weighed accurately to the nearest mg and dispersed with 25 mL of distilled water, and the sample was then swirled. Five mL of nitric acid are added along within 25 mL of hot distilled water, stirred vigorously and then transferred into a beaker. Next, the solution was heated, allowed to boil for about 4 to 5 minutes, and kept warm for 10 minutes. The sample was filtered, through a fast hardened ashless filter paper, into another beaker. The filter paper, funnel, and the stirring rod were rinsed thoroughly with hot distilled water to ensure complete transfer of the chlorides. Finally, the sample was allowed to cool to room temperature.

The sample was transferred into a stoppered 250 mL conical flash. Standard silver nitrate (AgNO<sub>3</sub>) solution was allowed to flow from one burette into the solution while it was swirled vigorously. The volume of the silver nitrate solution added was recorded to an accuracy of 0.1 mol/L. Then 3 mL of nitrobenzene was added to the mixture and swirled vigorously for about 30 seconds. One mL of iron indicator solution was then added. This was followed by back-titration of the mixture with a potassium thiocyanate solution delivered from a second burette until a reddish-brown colour persisted for one minute. The volume of potassium thiocyanate used was then recorded.

The mass of sodium chloride (NaCl) in milligrams present in the total sample was calculated as:

$$J = \left[V_5 - \left(\frac{V_6 * m}{0.1}\right)\right] \times \left[\frac{0.003545}{M_c}\right] \times \left[\frac{100}{C_1}\right]$$
(4.1)

where

J = Chloride ion content

- $M_c$  = Mass of sample used (in g)
- $V_s$  = Volume of 0.1 M silver nitrate solution added (in mL)
- $V_6$  = Volume of thiocyanate solution used (in mL)
- m = Molarity of the thiocyanate solution (in mol/L)
- $C_1$  = Cement content of the sample used (in %)

## 4.5 Development of the Various Corrosion Levels

The levels of corrosion achieved in the various specimens varied from no corrosion (control specimens) to extensive corrosion. One important aspect of this observation was the organization of experimental data in a longitudinal crack width framework so that the levels of corrosion could be defined for a given specimen under a given set of environmental conditions. While the example cited in this chapter uses the longitudinal cracking as the basis for the level of corrosion desired, or achieved, other methods such as the definition of current readings have been and can also be used to define the level of corrosion. It must be noted that this approach can be combined with other methods of defining the levels of corrosion.

Evaluation of the corrosion attack after exposure of the test piece was undertaken by inspection (it was used for assessing the level of corrosion earlier), measurement of corrosion (pit) depth or changes in the steel ultimate tensile strength. However, the evaluation was normally carried out by determination of the weight loss after the corrosion products had been removed by cleaning the bar with an acid.

In summary, three criteria were used to determine the corrosion levels, examined in this study:

(1) The width and the propagation of the longitudinal concrete crack which is the result of the formation of expansive corrosion products on the steel bar. The onset of cracking in the concrete cover and the increase of the crack width were monitored through visual observations of the specimens and recorded as this formed a reference for selecting subsequent postcracking levels of corrosion. Thus, a longitudinal crack width of 0.15 mm was defined to be the first stage of corrosion, while the last level of corrosion was that associated with a major 9 mm longitudinal crack. The crack width was measured with a crack width comparator at 50 mm intervals along each crack length. An average crack width was then obtained for establishing the level of corrosion as:

Average crack width= 
$$\frac{\sum crack \ width \ at \ each \ interval}{number \ of \ intervals}$$
 (4.2)

(2) Loss of metal relative to the original reinforcing steel bar weight. The weight loss of the steel reinforcing bar was obtained as the difference between the weight of the corroded bar (after the removal of the loose corrosion products) from its weight before corrosion. For this purpose, the specimens were broken open for the retrieval of the reinforcing bar after the completion of the test. The reinforcing bar, for each specimen was cleaned and scrubbed with a stiff metal brush to ensure that the bar was free from any adhering corrosion products. The reinforcing bar was then carefully examined for its general condition as a result of the corrosion effects in terms of pitting, rib degradation, and then weighed for the determination of the weight loss. The percentage weight loss was determined as follows:

(3) The loss of the cross-sectional area relative to the original cross sectional area of the reinforcing bar is obtained from the results of the load-deflection response for each specimen as was described in Chapter 3.

The results obtained for the three criteria are compared for the assurance of the appropriate corrosion level.

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(a) Typical tension specimen geometry



(b): the silicon layer



(c): the 10 mm x 10 mm grid

Figure 4.1: Typical tension specimen



Figure 4.2: Corrosion tank test set-up



Figure 4.3: Electrical circuitry

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# Chapter 5

# **Experimental Program**

This chapter describes the test program to investigate the effect of corrosion on the bond behaviour at the reinforcing steel-concrete interface. Tension specimens, reinforced with a single bar, were subjected to seven different stages of corrosion, and then subjected to axial tensile forces to determine the deterioration of bond between the concrete and the reinforcing steel bar.

## 5.1 Mechanical Properties of Materials

### 5.1.1 Reinforcing Steel

The 20M size reinforcing bars used in the tests were obtained locally, and they conformed to the Canadian Standard Association (CSA) Standard G30.14-M83 (1983) which is equivalent to ASTM Standard A615-72 (1972). To determine the mechanical properties of the reinforcing steel, tension tests were performed on six deformed 20M steel bars, with a nominal diameter of 19.5 mm, and 195 mm in length, with a specified yield strength of 400 MPa. All of the coupons exhibited a yield plateau, with an average experimental yield strength of 432 MPa. Figure 5.1 shows the tensile stress-strain relationship obtained. The strains were measured over a 50 mm gauge length extensioneter. Table 5.1 reports the various mechanical properties of the bars.

## 5.1.2 Concrete

As mentioned earlier, the concrete mixture was supplied by a local ready-mixed concrete

company, with a specified 28-day compressive strength of 25 MPa.

Again, the concrete mix consisted of Type 10 ordinary Portland cement, fine-to-coarse aggregate ratio of 1.0, and a water/cement ratio of 0.45 without any chloride additives. Sieve analysis of both the fine and coarse aggregates were conducted. These tests were conducted according to the CSA Standard A23.2-2A, "Sieve Analysis of Fine and Coarse Aggregate".

The concrete compressive strength,  $f_{er}$  was obtained from tests on three standard cylinders, 150 mm in diameter and 300 mm in height, according to the CSA Standard A23.2 - 9C -Compressive Strength of Cylindrical Concrete Specimens (ACI 318-89). The average compressive strength,  $f_{er}$  was 30.5 MPa. Figure 5.2 shows the concrete compressive stress-strain relationship for these specimens. The modulus of rupture,  $f_{er}$  was obtained by flexural tests on 100 mm by 100 mm by 400 mm long beams which were subjected to third-point loading, according to the CSA Standard A23.2 - 8C- Flexural Strength of Concrete (Using Simple Beam With Third Point Loading). (ASTM C78), over a span of 300 mm. The average modulus of rupture,  $f_{er}$  was 3.9 MPa. Tests were carried out to determine the splitting tensile strength,  $f_{sp}$ , on three standard cylinders 150 mm diameter by 300 mm length according to the CSA Standard A23.2 - 15C- Splitting Tensile Strength of Cylindrical Concrete Specimens (ASTM C496). The average splitting strength,  $f_{sp}$ , was 3.0 MPa. The standard cylindrical specimens were tested on the same day as the tension specimens, at an average age of 100 days. Table 5.2 reports the various mechanical properties of the concrete specimens.

### 5.2 Experimental Procedure

#### 5.2.1 Test Program

This investigation was carried out for seven different stages of corrosion including two specimens constituting the uncorroded stage. In each stage, bond behaviour was studied through an examination of both the splitting cracks and the transverse cracks. The corrosion stages were chosen according to the width of the longitudinal splitting cracks (see Table 6.1).

The tension tests were performed in the Structures Laboratory at McGill University. The Material Test System (MTS) hydraulic servo-controlled machine, with a capacity of 1000 kN in tension with a built-in linear voltage differential transducers (LVDT) for deformation measurements, connected to a PC workstation, was used to test the tension specimens. The data were stored automatically in the PC, analyzed and the relevant information was easily accessible.

The test specimens were installed and centred to subject the specimens to pure tension. Figure 6.15, in Chapter 6, portrays the overall test setup. The test set-up of the MTS machine consisted of a loading frame that transmitted the load through the tension grips at both ends of the reinforcing bar.

### 5.2.2 Testing Procedure and Loading Sequence

The specimens were subjected to an incremental axial loading. The tests were run using displacement control. An automatic data acquisition system determined the axial force (N) and the strains at every deformation stage. The MTS machine is able to register the measurements with different options. The data were acquired from displacement, time, load, and one strain channel. To obtain a complete and precise response, loads, deflections and strains were recorded every 5 seconds during the loading phase controlled by deformation using the computerized data acquisition system. In order to accurately determine the strains, an external MTS extensometer, that read the strains in the steel bar, was mounted along the side of the specimen, by clamping the extensometer to the reinforcing steel bar just on the outside of the concrete as shown in Fig. 6.15. The elongations were measured over a gage length of 1010 mm.

The specimen was subjected to a predetermined deformation using displacement control, however, the load would drop over a short holding period required to examine the specimen thoroughly and to take photographs for the different key load stages during testing and to record the crack development. Cracks were traced over the concrete surfaces as soon as they became visible, and the corresponding load levels, at which that the cracks initiated were marked on the surface beside each crack. A hand microscope (10X) was used to follow the crack propagation, when it was needed. At each cracking stage, the cracks were recorded using a crack width comparator. The maximum loads applied exceeded the yield point of the reinforcing bars.

The data acquired during the test were stored in the PC and was used to provide information about all of the relevant characteristics of the tests, for example, the loaddeformation (or elongation) curves.

Bar Size	Area	f,	ε,	Suli	Es
	(mm²)	(MPa)	(mm/mm)	(MPa)	(MPa)
20M	300	433.0	0.0028	676.4	200000

Table 5.1: Properties of reinforcing steel



Figure 5.1: Stress-strain relationship for steel reinforcment

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 Table 5.2:
 Properties of concrete

Specimen	f.'	f,	f.r
	(MPa)	(MPa)	(MPa)
SS1	30.56	3.9	3.0



Figure 5.2: Concrete compressive stress-strain relationship

# Chapter 6

# **Experimental Results**

This chapter summarizes the results of the overall responses of eight test specimens, tested after the development of the different levels of corrosion within six specimens. The observed behaviour between the uncorroded and the corroded specimens was compared using the load versus elongation response, stress-strain relationships, and the information on crack widths and crack spacings.

### **6.1 Elongation Responses**

The load vs. elongation response commences with the uncracked elastic region when the tensile resistance of the concrete is not exceeded. As mentioned earlier, when the element is cracked, then in general, two stages occur: formation of cracks followed by their stabilization. As the load and deformation increase significantly, and it is increased further, the steel bar yields finally when the specimen is considered to have failed.

The experimental results from the load-elongation responses, for both the corroded and the uncorroded specimens, are summarized in Table 6.1.

#### 6.1.1 Uncorroded Specimens SS1 and SS2

Figures 6.1 and 6.2 show the applied load versus elongation characteristics of the standard control Specimens, SS1 and SS2, respectively. The bare bar response is also shown in each figure; the phenomenon underlying the transfer of the force from the steel bar to the concrete

and vice versa is best understood by comparing the measured member response with that of the bare bar.

As can be seen from the figures, the control specimen elongation is negative prior to the load application due to the shrinkage of the concrete, since the specimen was tested after an average age of 100 days after casting. Ignoring the member shortening effects caused by shrinkage could result in an underestimation of the tension stiffening effect, as confirmed in a recent study by Bischoff (1995), who noted an average free shrinkage strain of  $-0.3 \times 10^{-3}$ .

Both specimens behaved in basically the same manner, as can be noted from their loadelongation responses of both Specimens, SS1 and SS2. The first transverse crack appeared at an applied load of about 12 kN and hence, the stress in the steel at first cracking,  $f_{x,cr}$ , was approximately 40 MPa. After the first cracking load, the load-elongation curve is noted to become parallel to the curve for the bare bar. The Specimens SS1 and SS2 have approximately the same load of 127.3 kN when steel yields at a stress of 423 MPa. The yield load of both of the specimens corresponds to the same bare bar yield strength.

### 6.1.2 Corroded Specimens

The corroded specimens elongation is zero prior to the load application because they were continually immersed in water and hence, there was no shrinkage.

For comparison purposes, the bare bar elongation and the control specimen response, are also reproduced, in each of the corroded specimen response.

#### 6.1.2.1 Specimen CS1

The Specimen CS1 had a longitudinal splitting crack, 0.15 mm wide, prior to testing, due to the corrosion of the steel bar. Figures 6.3 shows the load-elongation response for the corroded Specimen CS1.

As can be seen from the Fig. 6.3, the steel bar in the Specimen CS1 yielded at a load of 125.0 kN, displaying a slightly lower load bearing capacity than the uncorroded Specimen SS1, which had a yielding load of 127.3 kN. The first transverse crack appeared in the Specimen CS1 at an applied load of about 15 kN which is higher than that for the Specimen SS1 due to nonshrinkage, the stress in the steel at first cracking,  $f_{xer}$ , was approximately 50 MPa. Apart from the difference in the concrete contribution and the yielding load, the Specimen CS1 shows behaviour similar to that of the Specimen SS1, which is that after the first cracking load on the steel, the elongation curve became parallel to the load-elongation curve for the bare bar. The load-elongation response of the Specimen CS1 curve near the yield loads shows an initial yield with a very short plateau (Fig. 6.3). At this stage, after a short duration, the load was released. The load was increased and it was noted that the load stabilized at a load higher than the "initial" yielding load, and the specimen demonstrated continued yielding at this load.

### 6.1.2.2 Specimen CS2

The Specimen CS2 represented the second stage of corrosion with a 0.2 mm wide longitudinal crack prior to testing. The overall response observed was similar to that for the Specimen CS1. Figure 6.4 shows the load vs. elongation curves for the Specimens CS2 and SS1. The yielding load for the Specimen CS2 was 124.0 kN, which is slightly lower than the yield load for the Specimen CS1. Also, it shows a smaller concrete contribution. The first cracking load occurs at a load of 18.0 kN, which is a higher than that for the Specimen SS1. The response of Specimen CS2 observed was similar to that for the Specimen CS1 concerning the trend in the yield load (intial yielding load), and the decrease in the concrete contribution. The specimen response at the yield load was similar to that of Specimen CS1. Furthermore, after the first cracking, the load-elongation curve became parallel to the bare bar curve.

### 6.1.2.3 Specimen CS3

Figure 6.5 shows the response of the Specimen CS3 which represents the third stage of corrosion with a 6.0 mm wide of splitting longitudinal crack prior to testing. Comparing the response of the Specimen CS3 with that of the Specimen SS1 (Fig. 6.5), the Specimen CS3

shows a much lower yielding load of 115.0 kN and much less concrete contribution. The first transverse crack appeared at a load of 23 kN which is higher than the first cracking load of the Specimen SS1. Again, after first cracking, the load-elongation curve became parallel to the load-elongation curve for the bare bar; the yield load was the same for Specimens CS3 and CS1 and CS2. The specimen responce at the yield load was similar to that of Specimen CS1.

#### 6.1.2.4 Specimen CS4

Figure 6.6 shows the response of the Specimen CS4 which represented the fourth stage of corrosion. Specimen CS4 had several splitting longitudinal cracks which varied in width from 1.5 mm to 3.0 mm and two transverse cracks that were not around the full circumference of the specimen but rather between the splitting longitudinal cracks, prior to testing. The steel bar in the Specimen CS4 yielded at a load of 113.3 kN which is much lower than that for the Specimen SS1. The first cracking load, which basically opened or widened an existing transverse crack location, was at a load of 10 kN which is lower than the first cracking load for Specimen SS1. The concrete contribution for the Specimen CS4 was much lower than for the other corroded specimens. Observations similar to the other corroded specimens are evident in Fig. 6.6. The load-clongation curve became parallel to the bare bar and the trend of the response at yielding load was similar to that of the other corroded specimens.

#### 6.1.2.5 Specimen CS5

The response of the Specimen CS5 which represented the fifth stage of corrosion is shown in Figure 6.7 along with the response of the Specimen SS1. The Specimen CS5 had several splitting longitudinal cracks and also transverse cracks that varied in width ranging from 1.5 mm to 4.0 mm, prior to testing. The yielding load for the Specimen CS5 was 111.0 kN which is again lower than that of Specimen SS1. The first cracking load which also opened or widened an existing transverse crack, was approximately 5 kN which is lower than the first cracking load for Specimen SS1. The load-elongation curve for the Specimen CS5 hardly shows any concrete contribution. At a load of approximately 30 kN, the load-elongation curves for the Specimen CS5 and the bare bar became almost coincident.

#### 6.1.2.6 Specimen CS6

The Specimen CS6 which represented the sixth and last stage of corrosion and had a 9.0 mm wide longitudinal splitting crack. As shown in Fig. 6.8, the Specimen CS6 had a yielding load of 101.2 kN which is obviously much lower than that of Specimen SS1. No transverse cracks appeared in this specimen until beyond the yield strength of the steel bar at a load of 115 kN when one transverse crack appeared. At a load of approximately 35.0 kN, the load-elongation curve tended to lean to the right side showing a lower response than that of the bare bar. The observations for the Specimen CS6 at the yielding load showed a similar trend compared to the other corroded specimens.

## 6.2 Stress-Strain Relationship

Due to the loss of the cross-sectional area as a consequence of corrosion, the reported stresses were calculated from the equivalant cross-sectional area obtained from the relative yeilding loads of the uncorroded and the corroded bars as was discussed in Chapter 3, thereby representing the actual contribution of the concrete and the tension stiffening in the overall specimen response.

Figures 6.9 through 6.14 show the modified stress-strain relationship for each corrosion level for Specimen CS1 through CS6, respectively. Again, each stress-strain relationship curve obtained for the corroded specimens is coupled with the control specimen stress-strain curve for comparison purposes.

This use of an "equivalent" cross-sectional area led to an apparent increase in the tension stiffening which was observed for each corroded specimen, including CS5 and CS6 where they showed no concrete contribution in the load-elongation curves. This observed difference between the load-elongation curves and the stress-strain relationship curves is due to the fact that in the load-elongation curve for the specimen with a clean uncorroded 20M

bar was compared with that of the specimen with a corroded 20M bar. The response of the latter is obviously smaller than that of the first.

However, as can be seen, from the stress-strain curves, this tension stiffening decreases as the level of corrosion increases. The yielding stresses for each of the specimens are summarized in Table 6.1, while Table 6.2 shows the stresses at first cracking for each specimen.

## 6.3 Cracking Behaviour

The control specimens, SS1 and SS2, exhibited transverse cracks, well before the splitting cracks formed at the latter stages of the tests. By comparison, corroded specimens exhibited only transverse cracks in addition to the widening of some of the existing splitting cracks, as all of them had a splitting crack prior to the test. Figures 6.15 and 6.16 show the cracking behaviour for the uncorroded Specimens SS1 and SS2, respectively. Figures 6.17 through 6.22 show the splitting cracks in each specimen due to corrosion and its width before and after the test.

#### 6.3.1 Crack Spacing

The average crack spacing of the transverse cracks for all of the eight tension specimens was determined. Table 6.2 shows the average crack spacing for the different stages of corrosion and for the control specimens as well as the maximum crack spacing. The results for the number and spacing of the cracks clearly show that as the level of corrosion increases, the crack spacing increases.

#### 6.3.2 Crack widths

Figures 6.23 through 6.27 show the steel stress vs. the maximum crack widths for for the

Specimens CS1, CS2, CS3, CS4 and CS5 respectively, subjected to the different corrosion levels. The maximum crack width curve for the uncorroded specimen was also produced for comparison. The steel-stress vs. crack width curve was not obtained for Specimen CS6 due to the fact that it did not produce any cracking prior to the yielding of the reinforcing steel.

As can be seen from Fig. 6.26 and Fig. 6.27 for the Specimens CS4 and CS5, respectively, the steel stress vs. maximum crack curve starts from zero load because of the fact that the crack existed before the test. The influence of the corrosion level on the crack width is clearly shown in the figure. The crack width increased with an increasing amount of corrosion.

## 6.4 Corrosion Percentage (%)

The observations and measurements of the changes in strength, and of the corrosion pit depths, have been evaluated to determine the effect of the corrosion attack after exposure of the test specimen to the aggressive environment for varied periods of time. The weight loss after cleaning and brushing of the corrosion products was determined. The cross-sectional area of the reinforcing bars was also determined and the results are summarized in Table 6.3.

## 6.5 Chloride Ion Profile

The variation of the chloride ion content across the concrete cover thickness was obtained according to the procedure that was presented in Chapter 4, explaining how the chloride content changes with depth.

The experimental results are presented by plotting the chloride percentage obtained versus the depth from surface of the concrete. Figures 6.28 through 6.33 illustrate the profile for chloride penetration into the hardened concrete for all of the corroded Specimens CS1 through CS6, respectively.

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Specimen	Corrosio	Experimenta	Cross-	Experimenta	Size of
	n	1	sectiona	l	longitudinal
	percent	yielding	1	yielding	crack due
	due to	load	of steel	stress.	to corrosion
	mass		A,	$f_{i}$	(mm)
	loss	(kN)	(mm <sup>2</sup> )	(MPa)	
SS1	0	127	300	424	
SS2	0	127	300	426	
CS1	4	125	295	417	0.15
CS2	5.5	124	292	413	0.2
CS3	11	115	271	383	6.0
CS4	11.5	112	265	373	1.5-3.0
CS5	12	111	262	370	1.5-4.0
CS6	17.5	101	240	367	9.0

Table 6.1: Details of the results for tension specimens.

Specime	No.	Corrosion	Experimentai	Measure	Measured	Measure
n	of T.	percent	stress at	đ	max.	d
	erack	due to	first	average	eraek	max.
	s	loss	cracking.	erack	spacing	eraek
		of mass	$f_i$	spacing		width
			(MPa)	$S_m$	(mm)	
	 			(mm)		(mm)
SS1	10	0	40	83.3	115.0	0,40
SS2	10	0	45	83.3	110.0	0.35
CS1	9	4	50	90.9	103.2	0.35
CS2	8	5.5	50	100.0	105.0	0.4
CS3	3	11	77	167.0	300.0	1.0
CS4	5	11.5	37	200.0	275.0	0.8
CS5	3	12	17	238.0	345.0	0.8
CS6	0	17.5				

Table 6.2: Details of the cracking behaviour

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Specimen	Corrosio	Corrosion	Equival	Average	Size of	Numbers
	n	percent	ent	weight loss	longitudinal	of major
	percent	due to	cross-	per unit	crack due	eracks
	due to	equivalent	sectiona	length	to corrosion	before
	weight	$A_{\infty}$	1			testing
	loss	(%)	of steel	(gm/mm)	(mm)	
	(%)		л,			
			(mm <sup>2</sup> )			
SSI	0.0	0.0	300	0.0		
SS2	0.0	0.0	300	0.0		
CS1	4.0	1.7	295	0.130	0.15	1
CS2	5.5	2.7	292	0.159	0.2	1
CS3	11	10.0	270	0.316	6.0	1
CS4	11.5	11.7	265	0.329	3.0	3
CS5	12	12.7	262	0.359	4.0	4
CS6	17.5	20	240	0.539	9.0	3

Table 6.3: Comparison between the different criteria for corrosion evaluation



Figure 6.1: Tensile force - elongation response (Specimen SS1)



Figure 6.2: Tensile force - elongation response (Specimen SS2)



Figure 6.3: Tensile force - elongation response (Specimen CS1)



Figure 6.4: Tensile force - elongation response (Specimen CS2)



Figure 6.5: Tensile force - elongation response (Specimen CS3)



Figure 6.6: Tensile force - elongation response (Specimen CS4)



Figure 6.7: Tensile force - elongation response (Specimen CS5)



Figure 6.8: Tensile force - elongation response (Specimen CS6)



Figure 6.9: Comparison of calculated stress-strain relationship for CS1



Figure 6.10: Comparison of calculated stress-strain relationship for CS2



Figure 6.11: Comparison of calculated stress-strain relationship for CS3



Figure 6.12: Comparison of calculated stress-strain relationship for CS4



Figure 6.13: Comparison of calculated stress-strain relationship for CS5



Figure 6.14: Comparison of calculated stress-strain relationship for CS6








(a) SS2 before testing

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(a) 0.15mm crack before testing (b) specimen after testing Figure 6.17: Cracking behaviour for Specimen CS1



(a) 0.2mm crack before testing



(b) specimen after testing

Figure 6.18: Cracking behaviour for Specimen CS2





(a) 6mm crack before testing (b) specimen after testing Figure 6.19: Cracking behaviour for Specimen CS3





(a) 6mm crack before testing (b) specimen after testing Figure 6.20: Cracking behaviour for Specimen CS4





(a) 1.5 to 3mm cracking before testing

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(b) specimen after testing

Figure 6.21: Cracking behaviour for Specimen CS5



Figure 6.22: Cracking behaviour for Specimen CS6, 9mm crack before testing



Figure 6.23: Influence of corrosion on maximum crack widths for specimen CS1



Figure 6.24: Influence of corrosion on maximum crack widths for specimen CS2



Figure 6.25: Influence of corrosion on maximum crack widths for specimen CS3



Figure 6.26: Influence of corrosion on maximum crack widths for specimen CS4



Figure 6.27: Influence of corrosion on maximum crack widths for specimen CS5



Figure 6.28: Chloride ion penetration for speimen CS1



Figure 6.29: Chloride ion penetration for spcimen CS2



Figure 6.30: Chloride ion penetration for specimen CS3



Figure 6.31: Chloride ion penetration for specimen CS4





Figure 6.32: Chloride ion penetration for specimen CS5



Figure 6.33: Chloride ion penetration for specimen CS6

# Chapter 7

# Analysis of Test Results

This chapter discusses the results obtained from the accelerated corrosion tests on the tension specimens as described in Chapter 4. The responses observed from the pure tension tests, involving mechanical interaction between a corroded bar and the surrounding concrete, which are accompanied by the formation of splitting cracks are then discussed. The influence of corrosion on the cracking behaviour is presented in terms of both the crack spacing and the crack width.

## 7.1 Accelerated Corrosion Testing

As mentioned earlier, the initial plan was to subject two sister specimens to accelerated corrosion. One specimen was to undergo a direct tension test while the other specimen was to be used for the auxiliary tests such as determination of chloride ion content. mass loss and the strength degradation as a result of corrosion. It was planned to cut the second specimen to retrieve the reinforcement which was to be weighed after cleaning to measure the mass loss and then the bar was to be tested in tension to determine its tensile strength and the stress-strain response. In order to conduct these tests, the sister specimens had to be subjected to the same rate of corrosion and therefore to similar conditions of degradation. It was impossible to obtain identical conditions of accelerated corrosion and degradation, which would have required identical current readings, longitudinal cracking and equal chloride ion contents from the first and second stages of corrosion. This lack of identical behaviour of sister specimens is basically due to the heterogeneity of the concrete in terms of its permeability, which exerts a dominant effect on the corrosion process. The nonuniformity from location to location is conducive to

the formation of random large scale corrosion cells distributed along the steel bar. This kind of heterogeneity could subject the specimen to varying moisture contents. The varying permeability affects the availability of the oxygen at the steel surface, and also varying the distribution of NaCl at the steel surface.

Therefore, despite the controlled effort to control the accelerated corrosion test environment, it was difficult to obtain identical corrosion degradation in the sister specimens. Consequently, the original plan of twin tests for each corrosion condition was discontinued, and individual specimens were tested with all of the auxiliary tests being conducted on the concrete specimen and the reinforcing steel bar obtained from the same specimen.

## 7.1.1 Corrosion Behaviour

The experiment was run as described earlier, and the measurements were recorded periodically. An air pump was provided and placed in the experimental tank to increase the availability of oxygen at the steel surface to sustain the corrosion rate. It was decided that the specimens should be dried on a weekly basis for at least 48 hours, to ensure oxygen diffusion into them. In water-saturated concrete structures, the electrical resistivity is very low and the rate of corrosion of the steel reinforcement is primarily controlled by the diffusion of dissolved oxygen through the concrete cover. Experiments by Hansson and Sorensen (1990), showed that in totally submersed specimens the corrosion rate is so high that the oxygen dissolved in the pore solution of the samples is rapidly depleted and the corrosion reaction is stifled despite the high chloride content of the mortar. Also as expected, the effect of oxygen concentration on the rate of corrosion was quite significant.

As expected, the rate of corrosion of the steel bars in the specimens was very slow at first, until depassivation of the steel occurred when corrosion started and then the rate of corrosion increased significantly. The initial current densities monitored were of the order of  $10^{-4}$  A/m<sup>2</sup> and continued unchanged for a long time, until the chlorides penetrated the cover and initiated corrosion at which time the current increased by over three orders of magnitude

within a few days. The Specimen CS1 displayed the lowest corrosion rate (conditions) which was 4 percent loss of weight due to corrosion, becauese of the smallest exposure time (70 days). This corrosion rate increased with an increase of the exposure time, with the specimen CS6 displaying the most severe corrosion degradation, 17.5 percent loss of weight caused by corrosion, after an exposure period of (120 days). It is interesting to note that specimen CS3 had a wider longitudinal crack and appeared more corroded than the Specimens CS4 and CS5, however, the specimen CS3 (exposure time 90 days) showed slightly lower degradation than the Specimen CS4 (exposure time 99 days) and CS5 (exposure time 110 days) which confirms that the corrosion is influenced by the exposure time until the initiation of corrosion.

### 7.1.2 Chloride-Ion Content

Small amounts of chloride can disrupt the passive oxide layer that is protecting the steel from corrosion. Determining the chloride content is a vital part of any condition survey of the deterioration in the concrete, as chlorides in the concrete are known to be specially effective in eliminating the passivity at the steel bar surface. The results indicate that the loss of metal and the intensity of pitting for the bars increased with the increase of the chloride-ion content in the concrete; these findings are similar to those by Rasheeduzzafar *et al.* (1992). The results for chloride-ion migration into the concrete specimen show how the chloride content changed with the depth from the surface of the concrete towards the embedded steel. The results of the variation of the chloride-ion content with the concrete cover thickness showed that as expected, in the uncracked concrete specimens and parts of the specimens relatively free of cracks, the chloride ion concentration decreased from the exterior concrete surface to the steel bar surface. This phenomenon is basically the migration of chloride ions into the concrete through diffusion.

However, the chloride ion profiles in specimens which had cracked significantly were not consistent with this normal expectation, because the sodium chloride solution penetrated through the main cracks which were quite wide and into the microcracks around the steel bar, which explains the irregularity of the chloride ion profile in the specimens which had cracked

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(see Fig 7.1). The first and second stages of corrosion show the typical profile of diffusion of chlorides, in other words, the chloride concentration decreased from the surface to the interior in the vicinity of the steel surface. With reference to the rest of the corrosion levels, the profile of chloride ions are the opposite of the profile diffusion. This change of the profiles for the chlorides that had penetrated the hardened concrete, that was found, can be explained through the fact of widening the longitudinal cracks and the increase of their numbers as corrosion progressed. Therefore, with such increase of width and number of cracks the chlorides did not only migrate or diffuse into the concrete, that were located on the of the steel-concrete interface.

The chloride ions also migrate through the pores inside the concrete, due to the capillary suction of chloride solution into the air filled pores or by the diffusion in the water-filled pores in uncracked concrete, while in the region of cracks, chlorides penetrate much faster through the cracks until depassivation occurs and corrosion starts. Studies by Funahashi (1990), Rasheeduzzafar et al. (1992) and others showed that the influence of crack width on chloride ion penetration and hence on corrosion process is of minor importance after depassivation. Schiessl (1975) and Beeby (1978) suggest that the rate of corrosion, once it has started, is independent of the cover and the crack width.

## 7.1.3 Half Cell Potential

The electrochemical corrosion monitoring technique for the corrosion potential mapping method, ASTM (C 876-87): Half-Cell Potential of Uncoated Reinforcing Steel in Concrete, detects only the likelihood of corrosion (potential) and does not detect the rate of corrosion and may even be misleading since the corrosion potential depends upon the surrounding environmental conditions.

Other than the slow response time in providing a stable output voltage when contacted to the concrete surface, the Half-Cell potential does not provide reasonable readings. When the specimens has large moisture content, due to the fact that they were immersed in the NaCl solution, readings gave a very negative potential due to the restricted oxygen access, as was explained by Popovics *et al.*(1983). Also, the presence of moisture could have resulted in the conduction of electricity over the complete area of the specimen so that the area of corrosion activity was not identified [John *et al.* (1987)]. Some experimental work by Naish and Carney (1988), suggests that the degree of wetness of concrete has a pronounced effect on the measured corrosion potential and the associated current flowing from the reinforcement.

Consequently, poor success was obtained with this method, and hence, the weekly halfcell testing and measurements of the specimens fluctuated and these could not be attributed to any scientific reason, therefore, the half-cell test was discontinued. The current readings and the visual inspection of the concrete were used as the basis for establishing the corrosion level. Elsener and Bohni (1990) have conducted potential mapping surveys of bridge decks of the Swiss highways using a new eight-wheel electrode measurement device and compared it with the ASTM C 876-80. They found that more than 70% of all the corrosion sites on Cugniertoble Bridge II would not have been detected with the ASTM method, yet, the ASTM method produced better results on other sites. It should be noted that some users have found the corrosion potential test, when used in a grid pattern, to be a very useful indicator of corrosion activity.

# 7.2 Influence of Corrosion on Bond Behaviour

The specimens were designed according to the CSA Standard-A23.3-M84 and the ACI Committee 318-71 recommendations for the development length,  $l_{d}$ , for the bar must extend a distance,  $l_{d}$ , required to transfer the bar force to the surrounding concrete by bond. While the test specimens fulfil the development length requirement, they were left with the surface conditions of the bar, the deteriorated conditions at the interface between the concrete and the steel, and the associated opening of the longitudinal cracks within the corroded specimens.

As a consequence of corrosion, an accumulation of the corrosion products (iron oxides) accumulated along the steel bar. Because of the internal pressure caused in the increased volume of the corrosion products by the specimen confinement due to the increase in the volume of the steel as it is converted to iron oxides and hydroxides, the tensile stresses in the concrete were much greater than the concrete tensile strength and resulted in the longitudinal cracks that were observed in all of the corroded specimens. Figure 7.2 displays an opened specimen after testing showing the heavy layer of the corroded metal that is adhering to the concrete surface. Therefore, any adhesion between the concrete and the reinforcing steel was lost within the corroded specimens before the specimen was tested. Furthermore, this layer of corrosion products also broke down the friction mechanism, except for low levels of corrosion. The change in the level of roughness of the bar surface, as observed by Al-Sulaimani *et al.* (1990), can result in an increase in the bond strength with increasing levels of corrosion up to about 1% corrosion.

### 7.2.1 Bar Profile

Examination of the steel bar embedded in the tension specimen showed 'localized corrosion', as the deterioration was more specific to the various parts or regions of the steel bar, resulting in deep pits on the surface of the steel and its eventual severance, as illustrated in Figure 7.3. This altered the geometric shape and size of the reinforcement and hence it had a significant effect on the transfer of the force from the steel to the surrounding concrete by the mechanical interlocking of the bar ribs affected by the steel corrosion. Therefore, this primary mechanism of bond in deformed bars, that of transfer of forces by mechanical interlocking of the ribs, gets weakened by the effect of corrosion on the ribs.

In summary, two significant phenomena influence the deterioration of the bond strength due to the corrosion of the reinforcing steel, firstly due to the formation of the corrosion products along the reinforcement and secondly due to the longitudinal cracking.

#### 7.2.2 Cracking Behaviour

Figure 7.4 shows the crack patterns for the specimens subjected to different levels of corrosion which are compared with the crack pattern for the uncorroded specimen. The influence of corrosion is well demonstrated in this figure.

In the control Specimens SS1 and SS2, the crack spacings occurred as expected, which is the forming of the additional primary cracks with an increase in the stress level until the crack spacing was approximately twice the cover thickness which was about 83 mm. The calculated initial cracking load (transverse cracks) for the concrete Specimens SS1 and SS2 is 18 kN, but the experimental initial cracking load, adjusted for the concrete shrinkage, was 17 kN for the uncorroded Specimens SS1 and SS2. This is because of the induced tensile stresses (about 5 MPa) caused by the restrained concrete shrinkage affect which affect the amount of cracking that is visible at a given tensile force. Cusick and Kesler (1976) have conducted tensile tests on concrete specimen using two different types of cements, Type I cement and Type K (shrinkage-compensating) cement and compared their cracking behaviour. The Type K specimens exhibited first cracking at a higher load than the Type I specimen, which shows that the tensile stresses, induced in the concrete due to shrinkage, are responsible for a decrease in the cracking loads.

The initial cracking load for the Specimen CS3 was 23 kN. while these values for the Specimen CS4 and CS5 were 10 kN and 5 kN, respectively. In addition, there were no transverse cracks in Specimen CS6. It must be reemphasized that the longitudinal cracking in all of the specimens and the transverse cracking in Specimens CS4 and CS5 had occurred due to the influence of the pressure exerted by the corrosion products (with a large increase of volume) which exceeded the tensile strength of the concrete at the location of these cracks. It should be noted that the observed cracking in the Specimens CS4 and CS5 was more due to the opening and the extension of the cracks caused by the corrosion of the reinforcing steel than due to the exhaustion of the concrete tensile strength. It has also been established that the quality of bond between the concrete and the reinforcing steel deteriorates with the level of

corrosion. The variation of the initial cracking loads in the Specimens SS1, SS2, CS1, CS2, and CS3 can only be attributed to the variation of the concrete strength at the various sections of the specimens. However, this phenomenon needs further examination.

The fact that the corroded specimens had fewer transverse eracks than the control specimens would indicate a reduction of the bond between the concrete and the reinforcing steel. This could be explained by referring to the basic considerations in Chapter 4, that slip will causes a further reduction in the bond stress, which will increase the value of the minimum crack spacing,  $S_{a}$ , as in the case of plain bars, but in the case of deformed bars, this distance  $S_{o}$  is influenced by the internal cracks that are formed as a result of the increase in the loads after the primary crack had formed. A study by Goto (1971) on the cracks formed around the deformed bars in tension, notes that after forming a primary crack, further loading causes loss of adhesion adjacent to the crack, transferring the loads to the ribs of the bar, and then internal cracks form close to the main crack (primary crack), and further loading increases the internal cracking. Therefore, the effect of loss of adhesion and/or the internal cracking would result in reduction of the rate at which the force is transferred from the reinforcement to the surrounding concrete and hence an increase in the distance from the main crack over which the surface stresses are reduced. The reduction of the interlocking phenomenon between the ribs and the concrete keys due to the reduction of cross-sectional area of the ribs is the primary cause of this breakdown of bond. However, the layer of corrosion products also causes loss of adhesion and cohesion between the two materials.

Furthermore, the longitudinal cracking and the widening of the cracks, as corrosion progresses, also contribute to the loss of bond because of the smaller area of the steel surrounded by the concrete (part being lost due to the longitudinal cracking) as well as the loss of cohesion and adhesion between them. In addition, the maximum crack widths in the corroded specimens are much larger than in the uncorroded specimens which also indicates a weakening of the bond, see Fig. 7.5.

Goto (1971), Houde and Mirza (1972), and others observed that for the concrete specimen to crack, the force transferred from the steel bar to the concrete through the mechanical interlock between the bar ribs and the concrete must be enough to cause the concrete to crack. Again, once one or more cracks have developed in the specimen, further cracking would occur only if enough force was transferred between the steel and the concrete over half of the distance between the two cracks (or a crack and a free end) to cause the concrete to crack. As mentioned earlier, the corrosion of the bar ribs has a significant influence on the mechanical interlocking between the steel ribs and the concrete leading to a deterioration of the bond between the corroded ribs and surface of the steel bar, and the concrete. Thus, the force is not transferred as efficiently from the steel bar into the concrete, leading to a larger spacing between the cracks in a specimen with a corroded bar than in a specimen with an uncorroded bar. A hypothetical extreme would be an extremely corroded bar in which the ribs are completely corroded and the adhesion and cohesion between the reinforcing steel and the concrete has been completely destroyed. The crack spacing would be very large in such a specimen because of the significant loss of bond strength, for example, the specimen CS6, with a weight loss 17.5 percent due to corrosion, which did not exhibit any transverse crack until beyond the yield strength of the reinforcing steel.

In order to form a new crack between two existing cracks, the tensile force which causes cracking of the concrete should be introduced into the concrete by this bond. The significant degradation in the profile of the ribs indicates that the bond integrity of the bar is lost and that it takes a longer length of the steel bar to redistribute the forces and transfer them to the concrete where it would develop a new transverse crack.

### 7.2.3 Load-Elongation Response

In general, local bond failures are evidenced with increasing steel stresses, when full transverse cracks form. The bond stress value at the crack drops to zero. This can be noted on the load-elongation curve, where there is a slight dropping of the load. The experimental load-elongation curves show the contribution of the concrete in the specimens basically because this corrosion

is normally scattered along the length of the steel bar, leaving some areas or some ribs not completely destroyed, therefore, transfer of force from the steel bar to the concrete occurs in these regions, contributing to the overall transfer of force from the steel to the concrete, and vice versa.

The observations from the load-clongation responses mainly showed a decrease in the load carrying capacity for the corroded specimens, indicating a 2 to 20 percent loss of the cross-sectional area of the reinforcement, from the first level to the last level of corrosion, respectively. Figure 7.6 shows the relationship between the ultimate load versus corrosion percentage. The decreasing effect of the ultimate load capacity with increasing levels of corrosion is obvious again. The data show that the ultimate load decreases gradually up to a weight loss of 5.5 percent due to corrosion, i.e. for the 4.0 percent weight loss due to corrosion, the ultimate load is reduced by 1.7 percent, while the ultimate load is reduced to the extent of 20 percent for about 17.5 percent weight loss due to corrosion. Since the bond stress is well below the ultimate load in all cases of corrosion, this reduction is attributable to the loss of bond. The loss of ultimate load is most likely due to the reduction compares well with the "equivalent" cross-sectional area of the reinforcing steel bar (see Table 7.1).

Moreover, the curves are shifted to the right closer to the bare bar response, signifying much less concrete contribution, i.e. a decrease in the tension stiffening. In fact, the Specimen with the fifth and sixth levels of corrosion, with weight losses of 12 and 17.5 percent, respectively, hardly showed any concrete contribution, which was expected after experiencing the loss in the steel bar mass; the bar ductility was also reduced significantly. However, after modifying the stress-strain relationship, the corroded specimens indicated a better concrete contribution. The contribution of the concrete was less than that in the control specimens, decreasing gradually with the increasing level of corrosion. This could be explained by the heavy layer of the corrosion products adhering to the concrete and the steel, keeping the two materials in contact. Also, the fact, that the reinforcing bar showed localized corrosion, implies that some areas of the bars are not as badly corroded as the others, and hence some ribs along

the steel reinforcing bar are in fact interlocking with the surrounding concrete. In addition, the increasing number of the longitudinal splitting cracks and their increasing widths combined with the advancing corrosion also contribute to the loss of adhesion and cohesion between the two materials.

Specimen	Corrosion percent due to weight loss	Ultimate load capacity (kN)	Percentage loss of ultimate capacity
SS1	0.0	183	0.0
CS1	4.0	180	1.6
CS2	3.5	177	3.0
CS3	11	164	10.4
CS4	11.5	162	11.5
CS5	12	160	12.6
CS6	17.5	146	20

Table 7.1: Percentage loss of ultimate load.

## 7.2.4 Relative Bond Performance of Corroded Bars

If the bond stress, u, is assumed to be uniformly distributed over the length of the bar, l, between the cracks, i.e. the length over which the steel force is transferred, then

$$(\pi d_b 1) u - (\frac{\pi d_b^2}{4}) f_v$$

where  $d_b$  is the bar diameter, and  $f_i$  is the steel yield strength, giving

$$u = \frac{\vec{a}_b f_v}{4l}$$
 (7.1)

If  $d_b$  and  $f_v$  are constant, then

$$u = \frac{k}{l}$$
 (7.2)

where k is a constant, i.e. the average uniform bond stress is inversely proportional to the distance between the cracks.

If the bond stress in the uncorroded specimen is considered to be 100%, the relative percentage for the corroded specimens can be calculated using Equation 7.2. These values were calculated for all of the specimens and are summarized in Table 7.2 and plotted in Fig. 7.7. It can be noted from this qualitative plot that the bond between the concrete and the reinforcing steel deteriorates increasingly as the level of corrosion increases.

Specimen	Corrosion percent due to weight loss	Average crack spacing (mm)	Nominal bond stress percentage
SS1	0.0	83.3	100
CSI	4.0	90.90	91
CS2	5.5	100	83
CS3	11	167	50
CS4	11.5	200	42
CS5	12	238	35
CS6	17.5	1000	8

Table 7.2: Nominal bond stress percentage.

As can be seen, with 4 percent weight loss due to corrosion there is a 9 percent loss of bond strength, while a 17.5 percent weight loss due to corrosion, results in a 92 percent loss of bond strength which is expected. A study by Andrade *et al.* (1990), suggested that a reduction of 10 to 25 percent in the bar section in the critical zones of the structure will result in the depletion of its service life, while a reduction up to 5 percent, even with cracking and spalling, will indicate an early stage of deterioration with the remaining service life not as significantly influenced.

This most significant deterioration of bond in the postcracking corrosion stage, i.e. 17.5 percent weight loss resulted in 92 percent loss of bond strength. This can be explained on the basis of several interactive factors. The significant degradation of the steel reinforcing bar ribs

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due to the severe localized corrosion, the heavy layer of the corrosion products forming along the steel reinforcing bar and adhering to the concrete acting as a barrier between the steel reinforcing bar and the concrete which results in loss of adhesion and cohesion, as well as the reduction in the cross-sectional area. Therefore, it is postulated that the mechanical interlocking between the ribs of the reinforcing bar and the concrete deteriorate significantly with the increase of the corrosion level. Furthermore, the widening of the longitudinal cracks with the advancing of corrosion have also contributed to the loss of adhesion and cohesion between the reinforcing bar and the surrounding concrete.



Distance from surface, mm

Figure 7.1: Chloride ion penetration for each specimen CS1, CS2, CS3, CS4, CS5 and CS6



Figure 7.2: The corrosion products along the reinforcing steel bar

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Fig. 7.3: Severe localized corrosion



Fig. 7.4: Transverse tensile cracks and longitudinal splitting cracks for different levels of corrosion





Figure 7.5: Influence of corrosion on maximum crack widths for CS1,CS2, CS3, CS4 and CS5



Figure 7.7: Relative bond stress versus level of corrosion.

# Conclusions

This chapter summarizes the findings of the experimental studies on the influence of corrosion on the bond characteristics between the reinforcing steel and the concrete, and cracking of the reinforced concrete element due to corrosion.

# 8.1 Summary and Conclusions

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The primary objective of this research program was to simulate the prevalent conditions under severe local corrosion of the steel reinforcement to study the effect of the various levels of corrosion on the response of a standard tension specimen with a single reinforcing steel bar and the effectiveness of bond between the reinforcing steel and the concrete. The corrosion rate is evaluated for the different levels of corrosion along with the determination of the chloride ion penetration. It is anticipated that the findings of this research program and its application will result in an improved understanding of the corrosion problem, its seriousness, and its influence on the bond between the steel and concrete behaviour.

A study of the influence of corrosion and cracking on the bond behaviour of reinforced concrete, aimed at understanding of how are the bond stresses transferred from the corroded steel to the surrounding concrete (steel-concrete interface), was conducted. Tension tests were carried out for a preliminary study of the influence of the steel reinforcement corrosion on the bond behaviour. The bond strength was studied through both transverse and splitting cracks; a relative bond effectiveness of the corroded bars was determined from the crack spacing.

Different stages of the steel reinforcement corrosion were established to study their effect on the relative bond behaviour, ranging from no corrosion at all to "complete" corrosion at the steel-concrete interface. An electrochemical method was used to accelerate the corrosion within the specimens. Direct current was applied for increasing periods of time to the reinforcing bar embedded in the tension specimens, immersed in a concentrated sodium chloride solution (5% NaCl by weight of water). The reinforcing bar served as the anode, while a bare steel bar was located in the water to serve as the cathode. The chloride content was obtained for each tension specimen by chemical analysis using the Volhard Method [British Standard (1988), Part 124].

The bond strength decreases rapidly with an increase in the corrosion level, especially in the case of any severe localized corrosion. The bond behaviour is influenced by the deterioration of the reinforcing bar ribs, and by the reduced adhesion at the reinforcing bar surface due to the widening of the splitting cracks resulting from corrosion. These phenomena were examined in this research program and the following conclusions are drawn:

1) In the tension tests simulating severe localized corrosion, the bond strength decreases as corrosion increases until it becomes negligible, i.e. 4 percent loss of weight due to corrosion resulted in a 9 percent loss of bond, while a 17.5 percent weight loss resulted in 92 percent loss of bond between the steel and the surrounding concrete.

2) Corrosion, especially with severe localized corrosion, causes a significant reduction of the interlocking phenomenon between the ribs and the concrete keys due to the deterioration of the reinforcing bar ribs. This reduction of the interlocking between the reinforcing bar and the concrete, retards the primary mechanism of the bond in deformed bars, which is the transfer of forces by mechanical interlocking of the ribs, and hence, the bond strength decreases significantly.

3) The decrease in the tension stiffening with an increase in the corrosion level signifies the initiation of bond breakdown, which is very much influenced by the surface conditions of the bar and the level of its adhesion and cohesion to the surrounding concrete. 4) The number of transverse cracks in a specimen decreases as the corrosion level increases, i.e. the 4 percent weight loss due to corrosion exhibited 10 transverse cracks and resulted in a 9 percent of loss of bond, while a 17.5 percent weight loss did not exhibit any transverse cracks and resulted in a 92 percent loss of the bond between the reinforcing bar and the concrete, signifies a reduction of bond between the reinforcing steel bar and the concrete. The width of these transverse cracks increases as the corrosion level increases, and it signifies a reduction of bond between the reinforcing steel bar and the signifies a reduction of bond between the reinforcing steel bar and it signifies a reduction of bond between the reinforcing steel bar and the concrete.

In summary, three significant phenomena influence the deterioration of the bond strength due to the corrosion of the reinforcing steel:

a) With increasing corrosion levels, especially in the case of severe pitting corrosion (localized corrosion), the bond behaviour is strongly influenced by the severely deteriorated reinforcing bar ribs.

b) The effect of the formation of the corrosion products along the steel reinforcing bar surface. Furthermore, this layer of corrosion products also breaks down the friction mechanism, except for low levels of corrosion.

c) The reduced adhesion and cohesion between the reinforcing bar and the concrete due to the widening of the longitudinal splitting cracks.

# 8.2 Suggestions for Future Research

It is suggested that a similar study of the influence of corrosion on the bond behaviour be undertaken using the standardized tests covered by this investigation to cover the following experimental parametric studies:

1. Effect of the concrete strength and type. (Normal and high strength concretes, fibre-

reinforced concrete, selected admixtures to accelerate and inhibit corrosion).

2. Different types and sizes of reinforcing steel bars. (Normal, epoxy-coated steel bars, other treatments, with bar sizes ranging from 15M to 30M bars).

3. Concrete cover thicknesses, (ranging from 20mm to 70mm),

- 4. Reinforcement made from advanced composite materials.
- 5. Effectiveness of selected surface sealants.
- 6. Effect of different repair techniques.
- 7. Effect of cathodic protection applied initially.

It is suggested that a study of the "force transfer" be undertaken for steel bars of different diameters using the technique that Houde, (1972) used on uncorroded bars which is using specially prepared steel bars, with internal strain gauges installed in milled grooves, embedded in standard tension specimens with selected bar sizes. A study of the influence of cover depth ratio would be quite useful to examine the progressive deterioration of bond with increasing levels of corrosion. The internal strain readings will be used to calculate the variation of bond stress along the bar embedment length. The experimental data will be useful to determine the effect of the various parameters on corrosion of the reinforcing steel bars. Also, comparing the experimental data from the laboratory tests with the field data from selected bridge decks and parking garages would be very useful to establish a correspondence between the stage of corrosion observed in the laboratory and those observed in the field. Based on this correlation, an empirical relationship can be developed to assist the engineer with the establishment of the service life of the system being considered. This will enable development of appropriate practice-oriented guidelines for eliminating corrosion and enhancing the service life of the system.

Progressive deterioration of bond between the concrete and the steel bars under cyclic loading has already been demonstrated in the cases where the steel bars had not corroded. Similar dynamic studies should be carried out to study the influence of corrosion under dynamic and repeated loads.

# References

Abrams, D. A. (1951), "The Readers Write Section," Civil Engineering, Vol. 21, No. 6, pp. 51-52.

Abrishami, H.H., Cook, W.D., and Mitchell, D. (1994), "Studies on Bond and Cracking of Structural Concrete," Structural Engineering Series Report No. 94-5, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada, 183 pp.

ACI Committee 408 (1966), "Bond Stress - The State of the Art," Journal of the American Concrete Institute, Vol. 63, No. 11, pp. 1161-1188.

ACI Committee 318 (1971), "Building Code Requirements for Reinforced Concrete," American Concrete Institute, Detroit.

ACI Committee 222R (1989), "Corrosion of Metals in Concrete," Report of the ACI Committee 222, ACI Manual of Concrete Practice 1994, Part I, Materials and General Properties of Concrete, pp. 222R-1 to 222R-30.

ACI Committee 318 (1983), "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, 111 pp.

ACI Committee 408 (1991), "Bond Under Cyclic Loading - State of the Art," ACI Materials Journal, Vol. 88, No. 6, pp. 669-673.

ACI Committee 224.2R (1994), "Cracking of Concrete in Direct tension," Report of the ACI Committee 224, ACI Manual of Concrete Practice 1994, Part 3, Use of Concrete in Buildings - design Specifications and Related Topics, pp. 224.2R-1 to 224.2R-12.

Al-Sulaimani, G. J., Kaleemullah, M., Basunbul A., and Rasheeduzzafar (1990)," Influence of Corrosion and Cracking on Bond Behavior and Strength of Reinforced Concrete Member," ACI Journal, pp. 220-230.

Andrade, C., Alonso, M.C., and Gonzalez, J.A. (1990), "An Initial Effort to Use The Corrosion Rate Measurements for Estimating Rebar Durability," Corrosion Rates of Steel in Concrete, Special Technical Publication 1065, American Society For Testing and Materials, Philadelphia, Pennsylvania, pp. 29-37.

ASTM C78-84 (1992), "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third Point Loading)," Section 4 - Construction, Volume 04.02 Concrete and Aggregates, pp. 33-35.

ASTM C496-90 (1992), "Standard Test Method for Splitting Tensile Strength of Cylindrical Specimens," Section 4 - Construction, Volume 04.02 Concrete and Aggregates, pp. 269-



272.

ASTM C876-91 (1992), "Test Methods for Potentials of Uncoated Reinforcing Steel in Concrete," 1992 Annual Book of ASTM Standards, Section 4 - Construction, Volume 04.02 Concrete and Aggregates, pp. 437-442.

ASTM A615-90 (1992), "Specifications for Reformed and Plain Billet-Steel Bars for Concrete Reinforcement," 1992 Book of ASTM Standards, Section 1 - Iron and Steel Products, Volume 01.04 Steel - Structural, Reinforcing Pressure Vessel, Railway, pp. 389-392.

Bažant, Z.P. (1992), "Fracture Mechanics of Concrete Structures," Elsevier Applied Science, Proceedings of the First International Conference on Fracture Mechanics of Concrete Structures, Breckenridge, Colorado.

Beeby, A.W. (1978), "Corrosion of Reinforcing Steel in Concrete and its relation to Cracking," The Structural Engineer, Vol. 56A, No. 3, pp. 77-81.

Beeby, A.W. (1979)," The prediction of Crack Widths in Hardened Concrete," The Structural Engineer, Vol. 57A, No. 1, pp. 9-17.

Berkeley, K.G.C., and Pathmanaban, S. (1990), "Cathodic Protection of Reinforcement Steel in Concrete," Butterworth & Co., London, U.K. pp. 1-52.

Biczók, I. (1972), "Concrete Corrosion Concrete Protection," Eighth Edition, Akadémiai Kiadó, Budapest, Hungary, pp. 1-37.

Bischoff, P.H. (1995), "Influence of Shrinkage on Tension Stillening of Concrete", 1995 Annual Conference of the Canadian Society for Civil Engineering, Ottawa, Ontario, pp. 433-442.

Borgard, B., Warren, C., Somayaji, S., and Heidersbach, R., (1990) "Mechanisms of Corrosion of Steel in Concrete," Corrosion Rates of Steel in Concrete, ASTM STP 1065, N. S. Berke, V. Chaker, and D. Whiting, Eds., American Society for Testing and Materials, Philadelphia, pp 174-188.;

British Standard 1881 (1988), Part 124, "Methods for Analysis of hardened Concrete," Testing Concrete, British Standards Institution, London, U.K., pp. 1-35.

Broms, B., and Raab, A. (1961), "The Fundamental Concepts of the Cracking Phenomenon in Reinforced Concrete Beams," Report No. 310, School of Civil Engineering, Cornell University, Ithaca, pp. 1095-1108.

Brown, R.D. (1980), "Mechanism of Corrosion of Steel in Concrete in Relation to Design,

Inspection and Repair of Offshore and Coastal Structures," Performance of Concrete in Marine Environments, SP-65, ACI, Detroit, pp. 169-204, Editor: (V.M. Malhotra).

Byfors, K., Hansson, C.M., and Tritthart, J., (1986), Cement and Concrete Research, Vol.16, pp. 760-770.

Cabrera and Ghoddousi (1992), "The effect of Reinforcement Corrosion on the Strength of the Steel/Concrete Bond" International Conference - Bond in Concrete - from Research to Practise Proceedings, pp. 11-24.

CEB Task Group VI (1981), "Bond Action Behaviour of Reinforcement - State of the Art Report," Comité Euro-International du Béton, 153 pp.

CEB-MC90 (1990), "CEB-FIP Model Code 1990," Thomas Telford Services Ltd., London, England,

Collins, M.P., and Mitchell, D. (1991), "Prestressed Concrete Structures," Prentice-Hall Inc. Englewood Cliffs, N.J.

Cornet, I., Ishikawa, T., and Bresler, B. (1968), Materials Protection, Vol. 7, No. 3, pp. 44-47.

CSA Standard A23.2-2-13C (1994), "Splitting Tensile Strength of Cylindrical Concrete Specimens," CSA A23.2-94 Methods of Tests for Concrete, Canadian Standard Association, Rexdale, Ontario, pp. 333-338.

CSA Standard A23.3-M84 (1984), "Design of Concrete Structures for Buildings," Canadian Standard Association, Rexdale, Ontario, 281 pp.

CSA Standard A23.2-8C (1994), "Flexural Strength of Concrete (Using Simple Beam with Third Point Loading)," CSA A23.2-94 Methods of Tests for Concrete, Canadian Standard Association, Rexdale, Ontario, pp. 314-316.

CSA Standard A23.2-2A (1994), "Sieve Analysis of Fine and Coarse Aggregates," CSA A23.2-94 Methods of Tests for Concrete, Canadian Standard Association, Rexdale, Ontario, pp. 158-163.

CSA Standard G30.14-M83 (1983), "Deformed Steel Wire for Concrete Reinforcement," Rexdale, Ontario.

Cusick, R.W., and Kesler, C.E. (1976), "Interim Report-Phase 3: Behaviour of Shrinkage-Compensating Concretes Suitable for Use in Bridge Decks," T. & A.M. report No. 409, Department of theoretical and Applied Mechanics, University of Illinois, Urbana.



Elsener, B., and Böhni, H. (1990), "Potential Mapping and Corrosion of Steel in Concrete," Corrosion Rates of Steel in Concrete, Special Technical Publication 1065, American Society For Testing and Materials, Philadelphia, Pennsylvania, pp. 143-156.

Erlin, B., and Verbeck, G.J. (1975), "Corrosion of Metals in Concrete-Needed Research", SP-49, Corrosion of Metals in Concrete, ACI, Detroit, pp. 39-46,

Farah, F. (1993)"Accelerated Corrosion Testing of Reinforcing Steel Embedded in Concrete," M.Eng. Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada, pp. 217.

Gjorv, O. E (1975), " Control of Steel Corrosion in Concrete Sea Structures", SP-49, Corrosion of Metals in Concrete, ACI, Detroit, pp. 1-9.

Gjorv, O. Vennesland, O., and El-Busaidy, A. (1976), "Diffusion of Dossolved Oxygen Through Concrete," Corrosion/76, Paper No. 17, National Association of Corrosion Engineers, Houston.

Feliu, S., Gonzalez, J.A. and Andrade, C., (1995), "Effect of Current Distribution on Corrosion Rate Measurements in Reinforced Concrete," CORROSION, Vol. 51, No. 1, pp. 79-86.

Goto, Y. (1971), "Cracks Formed in Concrete Around Deformed Tension Bars," ACI Journal, Proc. Vol. 68, No. 4, pp 244-251.

Goto, Y., and Otsuka, K. (1979), "Experimental Studies on Cracks Formed in Concrete Around Tension Bars," Reprinted from: The Technology Report of the Tohoku University, Vol. 44 (1979) No. 1, June, pp. 49-83.

Griffin, D. and Henry, R. (1963), Proceedings, American Society for Testing and Materials, Vol. 63, 1963, pp. 1046-1079.

Houde, J., and Mirza, M., (1972), A study of Bond Stress -Slip Relations in Reinforced Concrete," Structural Concrete Series No. 72-8, McGill University.

John, D.G., Eden, D.A., Dawson, J.L., and Langford, P.E. (1987), Proceedings, CORROSION/87, Paper No. 136, National Association of Corrosion Engineers, Houston.

Lutz, L.A., and Gergely, P. (1967)," Mechanics of Bond and Slip of Deformed Bars in Concrete," ACI Journal, pp. 711-721.

Lutz, L.A., and Gergely, P., and Einter, G. (1966)," The Mechanics of Bond and Slip of Deformed Reinforcing Bars in Concrete," Structural Engineering Report No. 324, Cornell University.


Mathey, P., and Watstein, P. (1959), "Width of Cracks in Concrete at the Surface of Reinforcing Steel Evaluated by Means of Tensile Bond Specimens," ACI Journal, Proceedings Vol. 56, No.1, pp. 47-56.

Mehta, P.K., and Monteiro, P.J.M. (1992), "Concrete Structure - Properties and Materials," Prentice-Hall Inc. Englewood Cliffs, N.J.

Mirza, S.M., and Houde, J. (1979), "Study of Bond Stress-Slip Relationships in Reinforced Concrete," ACI Journal, Vol. 76, No. 1, pp. 19-46.

Nielsen, A. (1985), "Durability," Beton Bogen (The Concrete Book), Edited by Aa. D. Herholdt, Chr. F.P. Justesen, P. Nepper-Christensen, and A. Nilesen, Aalborg, Portland, pp. 200-43.

Ouyang, C., and Shah, S.P. (1994), "Fracture Energy Approach for Predicting Cracking of Reinforced Concrete Tensile Members." ACI Structural Journal, Vol. 91, No. 1, pp. 69-78.

Page, C., Al Khalaf, M.N., and Ritchie, A.G.B. (1978), "Steel/Mortar Interfaces: Mechanical Characteristics and Electrocapillarity," Cement and Concrete Research, Vol. 8, pp. 481-490.

Page, C.L., Page, N.R., and El Tarras, A. (1981), "Diffusion of Chloride Ions in Hardened Cement Paste," Cem. Concr. Res., 11, 395.

Palumbo, N. (1991), "Accelerated Corrosion Testing of Steel Reinforcement in Concrete," M.Eng. Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada, pp. 217.

Park, R., and Paulay, T. (1975), "Reinforced Concrete Structures", John Wiley & Sons, Inc. N.Y.

Popovics, S., Simenov, Y., Boghinov, G., and Barovsky, N. (1983), "Durability of Reinforced Concrete in Sea Water," Corrosion of Reinforcement in Concrete Construction, A. Crane, Ed., Ellis-Hor-Wood Ltd., Chichester, U.K. 19 pp.

Pourbaix, M. (1976), Atlas of Electrochemical Equilibrium in Aqueous Solutions, Pergamon, London.

Rasheeduzzafar, Ehtesham, H.S., and Al-Saadoun S.S. (1992), "Effect of Tricalcium Aluminate Content of Cement on Chloride Binding and Corrosion of Reinforcing Steel in Concrete," ACI Materials Journal, pp. 3-13.

Rehm, G. (1957), "The Fundamental law of Bond," Proceedings, Symposium on Bond and Crack Formation in Reinforced Concrete, Stockholm, RILEM, Paris, Pub. Tekniska Hogskolans Rotaprinttrychkeri, Stockholm,

Rosenberg, A., Grace, W.R. and Co.; Hansson, C., and Andrade, C. (1989), "Mechanism of Corrosion of Steel in Concrete," Materials Science of Concrete, Vol. 1, pp. 285-313.

Sakamoto, N., and Iwasaki, N. (1995), "Influence of Sodium Chloride on the Concrete/ Steel and Galvanised Steel Bond," Bond in Concrete, Applied Science Publishers, London, 1982, pp. 239-249.

Shalon, R., and Raphael, M. (1959), "Influence of Sea water on Corrosion of Reinforcement," ACI Journal, pp. 1251-1268.

Stratfull, R.F., Jurkovich, W.J., and Spellman, D.L (1975), "Corrosion Testing of Bridge Decks," Transportation Research Record No. 539, Transportation Research Board, pp. 50-59.

Tepfers, R. (1973), "A Theory of Bond Applied to Overlapped Tensile Reinforcement Splices for Deformed bars," Division of Concrete Structures, Chalmers University of Technology, Goteborg, Sweden, Publication 73:2, 328 pp.

Treece, R.A., and Jirsa, J.O. (1989), "Bond Strength of Epoxy-Coated Reinforcing Bars," ACI Materials Journal, Vol. 86, No. 2, pp. 167-174.

Verbeck, G.J. (1975), "Mechanism of Corrosion of Steel in Concrete," SP-49, Corrosion of Metals in Concrete, ACI, Detroit, pp. 21-38.