Bond and Durability of Fusion-Bonded Epoxy-Coated Steel Bars

by

Wael Mohamed Ali Elleithy

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

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In

CIVIL ENGINEERING

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This thesis, written by **Wael Mohamed Ali Elleithy** under the direction of his thesis Advisor and approved by his Thesis Committee, has been presented to and accepted by the Dean of the College of Graduate Studies, in partial fulfillment of the requirements for the degree of MASTER OF SCIENCE.

Thesis Committee

Dr. Al-Farabi Mohammad Sharif
Thesis Adviser

Dr. Omar S. Baghabra Al-Amoudi
Co-Advisor

Dr. Abul Kalam Azad
Member

Mohd. Maslehuddin
Dr. Mohammad Maslehuddin
Member

Dr. Al-Farabi Mohammad Sharif
Department Chairman

Dr. Ala Husain Al-Rabeh
Dean, College of Graduate Studies

16-6-95
Date

This thesis
is dedicated
to my parents, brother
and to those
who do good deeds
for the sake of
ALLAH

ACKNOWLEDGMENT

Praise and thanks be to Almighty Allah for his limitless help and guidance; and peace and prayers be upon his Prophet.

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THESIS ABSTRACT

FULL NAME OF STUDENT	:	Wael Mohamed Ali Elleithy
TITLE OF STUDY	:	Bond and Durability of Fusion-Bonded Epoxy-Coated Steel Bars
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Fusion-bonded epoxy-coated steel reinforcement is used to enhance the corrosion resistance of concrete structures. However, the effectiveness of epoxy-coating may be compromised due to inadvertent presence of holidays and surface damage. Additionally, the bond strength of epoxy-coated rebars is expected to be less effective than that of uncoated bars and may be further impaired due to prolonged exposure to thermal variations. This research was designed to evaluate the bond strength between FBEC bars and concrete under heat cool cycling exposure. The influence of holidays and coating damage on corrosion resistance of FBEC bars was also investigated.

Results of this research indicated a reduction in the critical bond strength between steel and concrete due to the use of epoxy-coating. The potential measurements demonstrated the need for an appropriate interpretation of results when this technique is used. The corrosion current density measurements indicated no corrosion of FBEC bars.

MASTER OF SCIENCE DEGREE

KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS
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June 1995

خلاصة الرسالة

اسم الطالب : وائل محمد علي الليثي
عنوان الدراسة : شدة ترابط وتحمل أسياخ الحديد المطلية بمادة الإيبوكسي
التخصص : الهندسة المدنية
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تستخدم أسياخ التسليح المطلية بالإيبوكسي لزيادة مقاومة صدأ الحديد في المنشآت الخرسانية، ولكن قد تتأثر فاعلية هذه المواد نظرا لوجود تلف سطحي أو فراغات في الطلاء الإيبوكسي، بالإضافة الى ذلك فان شدة الترابط بين الحديد والخرسانة قد تتأثر بوجود الطلاء. صمم هذا البحث لدراسة شدة ترابط الأسياخ المطلية بالإيبوكسي مع الخرسانة. أجريت دراسة معملية تم فيها تعريض عينات خرسانية لدورات سخونة-برودة متكررة. وقد تم تصميم الجزء الخاص بدراسة مقاومة الصدأ لدراسة تأثير وجود فراغات في الطلاء أو تلف سطحي للطلاء الإيبوكسي في وجود ثلاث نسب مختلفة من الكلوريدات.

دلت نتائج هذا البحث على وجود خفض لإجهاد الترابط الحرج بين الحديد والخرسانة لوجود مادة الطلاء. قياسات الجهد الكهربائي للصدأ أوضحت أن هناك حاجة لصحة توقيع النتائج عند استخدام الأسياخ الإيبوكسية. أثبتت قياسات كثافة تيارات الصدأ أن أسياخ الحديد المطلية بالإيبوكسي كانت في حالة جيدة.

درجة الماجستير في العلوم

جامعة الملك فهد للبترول والمعادن
الظهران – المملكة العربية السعودية
يونيو ١٩٩٥م

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Concrete is foremost among all the building materials used in the construction industry around the world. The facility with which, concrete can be deposited and made to fill forms or molds of almost any practical shape while it is plastic, the economic superiority, its versatility and adaptability are some reasons that make concrete very pronounced over other construction materials [1]. With all the advantages, there are some limitations associated with concrete. Examples include, the low ductility, its low tensile strength compared to its compressive strength, and the considerable irreversible shrinkage which concrete undergoes due to moisture loss. The most disturbing problem, however, seems to be its low durability performance in aggressive environments. Premature deterioration of concrete structures is very common in bridge decks, parking garages and where deicing salts are used [2]. In coastal regions, this problem is of particular importance as the structures are exposed to salt spray or sea water. In the tropical countries, the problem of concrete deterioration due to reinforcement corrosion is accelerated due to high levels of temperature and humidity [3].

Along the Arabian Gulf coasts, the shortage of suitable construction materials, skilled man power and inadequate specifications are some of the principal causes for the deterioration of concrete within a short span of time [4]. Furthermore, the deterioration is accentuated by the aggressive environmental conditions [5]. The large fluctuations in the daily and seasonal temperature and humidity conditions significantly affect concrete durability. The temperature can vary by as much as 30 ^{o}C during a typical summer day, and the relative humidity ranges from 40 to 100% within 24 hours. The variation in the temperature may initiate ever present cycles of expansion-contraction and hydration-dehydration of the hardened cement paste in concrete which causes damage due to thermal and mechanical stresses. This damage is reflected by microcraking leading to enhanced permeability of concrete [6]. At the early stages of concrete mixing, the climatic factors can influence the properties of hardened concrete. Hot weather conditions and insufficient curing may lead to a 30 to 40% reduction in concrete strength [7]. The geomorphic conditions which are characterized by contaminated and absorptive aggregates, soil and ground water contaminated with chlorides and sulfate salts further accentuate the deterioration processes.

1.2 CORROSION OF REINFORCING STEEL

Corrosion of reinforcing steel is the major cause of concrete deterioration world-wide. It is well known that well-made and salt-free concrete provides both physical and chemical protection to reinforcing steel. The chemical protection is provided by the formation of a thin gamma ferric oxide film that forms on the steel when embedded in fresh concrete. This film is stable in the highly alkaline environment (pH in the range of 11 to 13) [8]. The high alkalinity of the pore solution is attributed to calcium hydroxide and other alkali hydroxides; the later being formed from the small amounts of sodium and potassium oxides present in the cement as impurities [9].

According to Page [10], the lime rich layer, which is observed at the steel-concrete interface, provides further protection to the steel. This was confirmed by Leek and Poole [11] who reported that the interfacial layer consists of an aggregate-free zone of portlandite [$Ca(OH)_2$] of variable thickness (5 to 15 μm). This layer is thought to screen most of the surface of steel from direct access of aggressive ions and to act as an alkaline buffer to pH reduction resulting from the hydrolysis of corrosion products [11]. According to Sagoe-Crenstil and Glasser [12], both $Ca(OH)_2$ and calcium silicate hydrate (C-S-H) gel form a buffering pair, and a high pH is as readily maintained by both C-S-H and $Ca(OH)_2$. The physical protection to steel is provided by the dense and impermeable structure of the concrete cover, which retards the diffusion of the aggressive species, like chlorides, carbon dioxide, oxygen and moisture, to the steel-concrete interface. Depassivation of steel may occur by the reduction of the pore solution pH, due to carbonation, or by the ingress of chloride ions to the steel-concrete interface.

When the atmospheric carbon dioxide (CO_2) makes a possible ingress through the concrete matrix and its penetrating front advances so deeply that it intercepts the steel reinforcement, it reacts with the calcium hydroxide and other compounds in cement forming calcium carbonate. The consequence is the reduction in the pH of the pore solution. Both CO_2 and moisture are necessary for the process of carbonation, as gaseous CO_2 does not react directly with hydrated cement [13]. Carbonation is, therefore, minimum at both low and high humidity. The process of carbonation is, however, known to be slow with good quality concrete. Moreover, the products of carbonation (i.e. $CaCO_3$) have more volume than the reactants (i.e. $Ca(OH)_2$) thereby blocking some of the pores in concrete and impeding further ingress of CO_2 and moisture. The process of carbonation slows down reaching a negligible rate in normal exposures.

The chloride ion is the principal cause of corrosion of reinforcing steel in concrete. Its presence in concrete is uniquely effective in promoting rebar corrosion. A number of mechanisms by which chlorides break down the passive layer have been proposed, e.g., the chemical dissolution of the film [14], the build up of the metal holes at the film/substrate interface [15], and due to high chloride concentrations at the iron oxide/pore solution interface which leads to local acidification and pitting [16]. Some studies have shown that chloride ions initiate corrosion by breaking the bond between the passive film and the metal [11]. Further, chlorides also reduce the electrical resistivity of concrete, leading to sustenance of reinforcement corrosion.

Chloride ions can enter the concrete from a variety of sources, both during and after construction, the principal sources of chlorides are as follows [9]:

<u>During construction:</u>

1. Use of sea or brackish water in mix.
2. Use of contaminated aggregates.
3. Deliberate addition of calcium chloride as an accelerator.

<u>After construction:</u>

1. Sea water ingress.
2. Saline ground water permeation.
3. Use of de-icing salts.

Corrosion of reinforcing steel leading to deterioration of, and damage to, reinforced concrete can take different forms. The two common forms of reinforcement corrosion in concrete are the general and localized (pitting) corrosion. The general corrosion occurs uniformly over the steel surface, typical of carbonation, while localized or pitting corrosion prevails when the corrosion is concentrated on a relatively small proportion of the total steel area and most frequently occurs when the reinforced concrete is exposed to chloride-bearing environments.

Sufficient moisture and oxygen are required to promote corrosion [8]. However, once corrosion is initiated, it is the electrical conductance of concrete that controls the rate of corrosion. Whatever is the cause of corrosion, the volume of the rust product is in the range of 2 to 14 times that of the parent iron from which it is formed [11]. The corrosion products exert tensile stresses approximately 10 times the tensile strength of normal concrete. At an advanced stage of the corrosion process, the excessive pressure causes the concrete cover to crack and may eventually spall off, which leads ultimately to a reduction in the cross sectional area of a structural member thereby endangering the safety of that member.

1.3 ECONOMIC SIGNIFICANCE 0F REINFORCEMENT CORROSION

Corrosion of reinforcing steel has gained economic significance in the whole world, since considerable resources have to be diverted towards the repair and rehabilitation of deteriorated structures [17]. In the United States of America, 162,222 bridges are detrimentally affected by chloride-induced corrosion, requiring a multi-billion dollar repair and reconstruction program [18]. Similarly, a cost of 600 million pound Sterling is estimated for the repair of road bridges in the United

Kingdom [19]. The cost of repair and rehabilitation of reinforced concrete structures in the Arabian Gulf is not very well documented but, undoubtedly, considerable resources have to be allocated towards restoring the useful service-life of concrete structures serving in this environment.

1.4 CORROSION CONTROL OPTIONS

Many protective measures have been proposed in order to mitigate reinforcement corrosion, particularly at the stage of construction. Some of these have been shown to be effective while others have failed. The most successful systems are listed below [20]:

i) Overlays and patches of very low water-cement ratio (0.32) using conventional low-slump concrete, latex-modified concrete, concrete containing silica fume, and high-range water reducing admixtures.
ii) Waterproof membranes
iii) Surface protective-barrier systems produced from selected silanes, siloxanes, epoxies, polyurethanes, and methacrylates [21].
iv) Cathodic protection [9].
v) Polymer impregnation.
vi) Use of corrosion inhibitors [22].
vii) Use of fusion-bonded epoxy-coated reinforcing steel.

The durability performance of reinforced concrete structures can be enhanced by the adoption of one or several of the above options. The use of fusion-bonded epoxy-coated rebars (FBECR), however, seems to be one of the most favored forms of protection. While several studies have been devoted to study the effect of epoxy-coatings on bond strength and corrosion resistance, there is not enough information in the literature on the effect of degree of damage or holidays on the performance of epoxy-coated reinforcing bars in chloride bearing environments and the effect of heat-cool cycling on the bond strength between epoxy-coated bars and concrete. Such a study is useful for the Arabian Gulf environment, since the concrete in this region is contaminated with chloride salts and the concrete structures are often exposed to thermal variations.

CHAPTER 2

LITERATURE OVERVIEW

2.1 HISTORICAL REVIEW OF THE USE OF FBEC BARS IN CONCRETE

Coating of reinforcing steel with a fusion-bonded epoxy powder was originated in the USA in the early 1960's to mitigate the wide spread and severe deterioration of bridge decks due to reinforcement [2,20]. The first recorded research [2,18,24,25,26] evaluating the effectiveness of epoxy-coated reinforcing bars was initiated in 1970 by the Federal Highway Administration (FHWA). In that investigation, 47 different coatings were investigated, whereby the only coatings to demonstrate adequate toughness and impermeability were the epoxy powder coatings developed originally to coat steel pipes. Based on the results of that study, the Federal Highway Administration set up a program to encourage state highway departments to use epoxy-coated rebars in the bridges.

The first major use of epoxy coated bars was in a Pennsylvania bridge deck in 1973 [2,25,27]. By the end of 1975, about 40 bridges were constructed using these bars [18]. However, fusion-bonded epoxy-coated rebars (FBECR) did not become commercially available in the market until 1976 [25]. Since then, the usage of these bars in the construction industry is continually increasing. FBECR is now being specified for new and replacement bridge decks by most state highway departments in USA [25,27,28]. Further, it has been extensively used in Canada, the Middle East, and elsewhere [18].

In recent years, the use of FBECR has spread to many other types of concrete structures such as waste water treatment plants, parking garages, marine structures, and subways. A more recent development is the epoxy-coated strands for prestressing applications. In 1981, the American Society for Testing and Materials (ASTM) published a standard for FBECR (ASTM A775-81). The standard [29] specifies that the epoxy coating be applied by the electrostatic spray method and that, before application, the reinforcement be cleaned by a massive blast to near white metal. It gives the coating curing time, and specifies the film thickness limits of 130 to 300 µm. Further, this standard lists in detail the test requirements for film thickness, film integrity, adhesion to reinforcement, and chemical and electrical resistance. Bond strength tests of coated bar to concrete, creep tests, coating abrasion, impact and hardness are also specified. The ASTM A775-81 was followed by the Japanese Specification JSCE EP10 in 1986 and the British Standard BS 7295 in 1990. There are currently other national standard specifications and guidelines in the final stages of development [30].

2.2 BOND BETWEEN CONCRETE AND FBECR

An important and necessary consideration in the use of fusion-bonded epoxy-coating is the evaluation of its impact on bond behavior between the rebar and the surrounding concrete. In order to study the relative bond strength of coated and uncoated bars, a series of 28 No. 6 (19 mm) bars were embedded in large concrete prisms and subjected to pullout tests by Mathey and Clifton [24]. Twenty three bars had varying coating thicknesses and five bars were uncoated. Two different bar deformation patterns were used (a barrel or diamond-shaped pattern). Each bar was evaluated based on the tensile force required to produce a 0.01 in. (0.25 mm) slip at the loaded end or a 0.002 in. (0.05 mm) slip at the free end, whichever is lesser. They concluded that bars with a coating thickness from 25 to 280 µm developed acceptable bond strength. However, they recommended that bars with an epoxy coating thickness greater than 250 µm should not be used.

Treece and Jirsa [31] conducted a comparative study of the strength of lapped splices of coated and black bars using beam specimens. Their study formed the basis for the present ACI standards (ACI 318-89) [32]. The bond strength of epoxy-coated bars was compared to that of uncoated ones. The variables studied included bar size, concrete strength, casting position and coating thickness. They concluded that epoxy-coating significantly reduced the bond strength. The bond strength of epoxy-coated bars was approximately 85 and 65% of that of the uncoated bars for pullout and splitting failures, respectively. They also concluded that the reduction in bond strength was independent of the bar size and concrete strength. The width and spacing of cracks were significantly increased in concrete specimens with FBEC bars. Moreover, they found that the reduction in bond strength was insensitive to variations in the coating thickness when the average coating thickness was in the range of 130 to 360 µm. They suggested that the basic development length should be multiplied by a suitable factor when epoxy-coated bars are used.

In an investigation by Lin et. al. [33], pullout tests were conducted on reinforced concrete slabs cast with coated and uncoated reinforcement. A minimum cover of 51 mm was provided to the 19 mm diameter test bars. Tests were performed at temperatures of 230, 300, 500 and 700 ^{0}F (110, 150, 260 and 370 ^{0}C). All the bars were able to attain their yield strength, although the coated bars displayed greater free-end slip than the uncoated bars at temperatures of 230 ^{0}F (110 ^{0}C) and above.

In a study conducted at King Fahd University of Petroleum and Minerals by Al-Sulaimani et. al. [34], pullout test specimens were used to evaluate the effect of epoxy coating thickness and the level of rebar corrosion on the bond behavior. Their results indicated that epoxy coating reduced bond strength and increased the slip in comparison to uncoated bars. Corrosion of reinforcing steel had a similar effect. They reported that, beyond a certain cracking level, corroded bars tend to have a lower bond strength and a higher slip than bars protected by epoxy coating within ASTM requirements, whilst bars with corrosion below that level had about the same bond strength and lower slip than coated bars.

Ziraba et. al. [35] presented a mathematical model to predict the local shear stress-slip law, derived from extensive pullout tests of epoxy-coated bars. The model requires the bar diameter and coating thickness as an input. The formulation was based on equilibrium and the derived bond stress-slip law results in a one dimensional model if the strain in the surrounding concrete was neglected as being small compared to the strain in the steel. The element stiffness relations for each zone of the stress-slip law were incorporated in a non-linear finite element program. Static condensation was used for the elements that straddle two characteristic slip zones, thus avoiding mesh regeneration. Convergence was not significantly affected. Close agreement existed between experimental and numerical results.

Choi et. al. [36] conducted a large scale study to determine the effects of coating thickness, deformation pattern and bar size on the reduction in bond strength between reinforcing steel and concrete caused by the epoxy-coating. The tests included beam-end and splice specimens containing No. 5, 6, 8 and 11 (16, 19, 25 and 35 mm) bars with an average coating thickness ranging from 80 to 430 μm. Three deformation patterns were evaluated. Their findings indicated that the extent of reduction in bond strength was less than that used to select the modification factors in the 1989 ACI Building Code (ACI 318-89) [32]. The coating thickness was found to have a little effect on the amount of bond strength reduction for No. 6 (19 mm) bars and larger. Thicker coatings caused a greater reduction in bond strength than thinner coatings for No. 5 (16 mm) bars. They also concluded that the reduction in bond strength increases with bar size. Bars with relatively larger rib-bearing areas with respect to the bar cross-section were less affected by the coatings than bars with smaller bearing areas. Their results suggested that a lower penalty can be employed for coated bars with a 2 bar diameter cover than that recommended by Treece and Jirsa [31], as mentioned previously.

Swamy and Koyama [37] reported different reduction magnitudes in bond strength when bars of three different rib patterns were used. The patterns were perpendicular, diagonal and double diagonal in shape. In pullout specimens, reductions of 4.9, 13.7 and 31.4 % in bond strength were measured at a free end slip of 0.05 mm, respectively. The authors concluded that the bond strength of epoxy-coated bars in concrete depended on the geometry and surface configuration of the lugs. Bars with lugs perpendicular to the bar axis had nearly the same bond strength as uncoated bars.

Cleary and Ramirez [38] carried out a study on the bond of epoxy-coated reinforcement in slab-type members. The slabs dimensions were 13 ft (4 m) long, 2 ft (0.6 m) wide, and 8 in (20.3 cm) deep. The tests were conducted on four series of specimens. The reinforcement consisted of three No. 6 (20 mm) bars spliced at mid span. The cover to bar diameter ratio was 2.67 and the splice lengths varied from 16 in (40.6 cm) to 10 in. (25.4 cm). They reported a reduction of bond strength of 5 to 18 % for different detailing and concrete compressive strengths.

Cusens and Yu [26] conducted single and double pullout tests in order to evaluate the bond strength and slip of epoxy-coated reinforcing bars in concrete. Slip measurements were made while a tensile force was applied to the reinforcing bars embedded in concrete in extended pullout tests. In the double pullout tests, they applied 20 cycles of load at levels of steel stress between zero and 0.5 times of the characteristic steel strength. They measured strains by strain gauges glued inside the bars. Both the epoxy-coated and uncoated bars were used in order to obtain comparative results. They found that the strain gradient along the bar was lesser for the coated reinforcement. The epoxy coating was found to increase the slip in bond thereby reducing the bond performance of coated bars.

Hester et. al. [39] conducted a total of 65 beam and splice specimen tests. From these tests they concluded that the epoxy coating significantly reduced the splice strength of deformed reinforcing bars in concrete. The extent of the reduction was less than that used to select the development length modification factors in the 1989 ACI Building Code (ACI 318-89) [32]. They also concluded that the transverse reinforcement improved the strength of the splices containing both coated and uncoated bars.

Cusens and Yu [40] studied bond and flexural behavior of concrete beams with FBEC reinforcing bars under static, repeated and sustained loads. They confirmed that the bond strength of epoxy-coated bars was reduced due to the effect of epoxy coating, and the relative coating was unlikely to affect the overall behavior of reinforced concrete beams in service. They concluded that neither cyclic loading up to 70% of the yield strength nor sustained load at the same level for 45 days affected significantly the bond behavior of beams at the ultimate load.

Cleary and Ramirez [41] subjected 23 beams with splices placed in a constant moment region to repeated loading and then tested them to failure in order to compare the ultimate load behavior of beams with coated and uncoated reinforcement. The repeated loading consisted of one million or five million cycles in the service-load range. They found that deflections, crack widths and reinforcement stresses were larger in beams with epoxy-coated reinforcement during the service loading while the differences in crack widths, deflections and bar stresses were reduced with repeated loading.

Hasan and Ramirez [42] conducted laboratory and field investigations to study the effect of static and repeated loading on the bond strength in concrete bridge decks and slabs reinforced with epoxy-coated bars. Twenty-four beams with No. 7 and No. 11 (22 and 35 mm) bars were tested. After the initial loading, the beams were subjected to cyclic loading. The laboratory findings indicated that the average concrete crack width was larger in specimens with epoxy-coated bars than that in companion specimens with uncoated bars. In the field specimens, the authors observed no signs of concrete delamination in one of the five bridge decks studied.

Moreover, the rebar sections extracted from the deck showed no signs of damage to the coating.

Hadje-Ghaffari et. al. [43] conducted an extensive testing program involving 637 beam-end and 85 full-scale splice specimens. They presented the results obtained from 376 beam-end specimens used to evaluate the effect of cover, casting position, slump and consolidation on the reduction in bond strength between reinforcing bars and concrete caused by the epoxy coating. They concluded that the epoxy coating significantly reduced the bond strength. The authors recommended that the ACI development length modification factor can be reduced from 1.5 to 1.35, while the factor of 1.2 is realistic. They also concluded that the relative bond strength of epoxy-coated reinforcement increased as the cover increases. Their findings indicated that, in low slump concrete, the ratio of the bond strength of bottom cast bars to that of top cast bars is about the same for coated and uncoated bars. As slump increases, the ratio increases significantly for uncoated bars and decreases slightly for coated bars.

In order to investigate the effect of fusion-bonded epoxy-coating on the bond strength, two sets of tests were recently conducted by Cairns and Abdullah [44]. Results of that study showed that the coating of bars reduced the friction between the steel and concrete by 50% when the stress normal to the interface is low. As the normal stress increases, the difference in behavior reduces. Their results indicated that the inclination of the bearing face of the ribs affects the bond behavior and the difference in behavior of coated bars is less with steeper ribs. They suggested that the difference in development length requirements can be reduced if restrictions are placed on rib geometry of bars selected for coating.

The above studies indicated that the epoxy-coating reduces the bond strength between the reinforcing bars and surrounding concrete. Further, testing is needed to address the influence of bar size, concrete cover, coating thickness, deformation patterns and the effect of heat-cool cycling on the bond of epoxy-coated reinforcing steel. Part of this research aimed at assessing the effect of coating thickness and bar diameter on the bond between steel and concrete. The effect of heat-cool cycling simulating the environmental conditions of the Arabian Gulf on the bond strength was also evaluated.

2.3 CORROSION PERFORMANCE OF FBECR

The basic purpose for coating the steel bars is to improve the durability of reinforced concrete, by inhibiting reinforcement corrosion. Clifton [45] presented the results of a testing program which was sponsored by the Federal Highway Administration (FHWA) to screen 47 organic coatings for reinforcing steel in concrete. That study indicated that the only coating to demonstrate adequate toughness and impermeability was the epoxy-powder coating.

In 1980, the FHWA initiated an outdoor study at its exposure site to determine the effect of coupling coated and uncoated reinforcing bars in concrete [46]. That study was also set to determine whether significant reductions in the corrosion rate could be achieved by coating all bars with epoxy rather than coating only that portion which would be exposed to salty environment. The results of that study confirmed that the epoxy-coated rebar slabs had much less corrosion than the uncoated steel slab. Corrosion of the black steel slab had induced widespread cracking, while none was found in the epoxy-coated rebar slabs. Though the slabs with epoxy-coated rebars performed 11.5 to 41 times better than the slabs with black steel bars, there was light rusting under the epoxy-coating at certain locations. However, the best solution was achieved when all the bars were epoxy-coated or the coated bars were electrically isolated from other metal in the structure.

In order to evaluate the effectiveness of epoxy-coating to reduce reinforcement corrosion, Swamy and Koyama [37] subjected square prisms, containing a centrally placed bar with an embedment length of 760 mm, to two exposure regimes. The environments tested were a natural marine exposure in a tidal zone and an accelerated cyclic sea water immersion in the laboratory. For comparison purposes, prisms containing uncoated plain and galvanized bars were also tested. They included additional tests on prisms containing artificial damage to the coating by tearing part of the coating for about half of the embedded length of the bars. The authors highlighted that even when the coating was accidentally damaged, with adequate cover and good quality concrete, corrosion damage in the specimens made with epoxy-coated bars was negligible, far less than that of comparable uncoated or galvanized bars.

Satake et. al. [47] conducted a long-term study on the corrosion-resistance of epoxy-coated, plain (uncoated) and galvanized bars. The tests were conducted on centrally reinforced concrete prisms with variable cover and pre-formed cracks. Three different thicknesses of epoxy-coating varying from 100 to 300 μm were used. The specimens were subjected to accelerated laboratory and marine exposure tests in a tidal zone up to two years. They concluded that galvanized bars performed better than the uncoated ones up to 2 years of accelerated tests. Galvanization, however, did not provide complete protection against pitting. Extensive and deep rusting in plain bar specimens and localized rusting in the galvanized bars were indicated in concrete specimens exposed to marine environments. The test results indicated that epoxy-coating can provide an effective long-term protection to steel against corrosion.

An investigation by Yeomans [48] to compare the corrosion performance of black, hot dip galvanized and fusion-bonded epoxy-coated steel reinforcements confirmed that epoxy-coating effectively eliminated corrosion, provided the coating was not damaged. Where coated reinforcement was left with cut ends unrepaired, the epoxy coated bars showed early corrosion of the exposed steel with corrosion progressing along the bar under the coating. Even where cut ends were repaired, the epoxy-coated bars showed many sites of breakdown of the repair and corrosion of the underlying steel.

Hededahl et. al. [49] employed a worst case experimental design to investigate the effectiveness of epoxy-coated reinforcement in concrete highway structures exposed to deicing salts. They exposed two cracked concrete barrier walls with coated reinforcement and small cover to frequent applications of de-icing salts. The results were compared with a similar barrier wall with uncoated reinforcement. It was confirmed that epoxy-coating was effective in protecting the reinforcing steel in chloride contaminated-concrete during the nine years of exposure.

Scanell and Clear [28] conducted a 6.5 years outdoor exposure comparative study of coated and uncoated reinforcing steel bars. Twelve by 24 by 6 inch (30x60x15 cm) slabs were evaluated under situations of epoxy-coated both mats and epoxy-coated top mat only. They included uncoated bars in both mats as a comparative study variable. Their results indicated that the epoxy-coated bars were many times more resistant to corrosion-induced damage than uncoated bars when embedded in a salt contaminated concrete. The overall best performance was achieved when both mats were epoxy-coated.

Sohanghapurwala and Clear [50] studied the corrosion characteristics of straight and bent epoxy-coated reinforcing steel bars. A total of 40 small concrete slabs were tested. They reported that both straight and bent epoxy-coated bars provided a significantly better resistance to chloride-induced corrosion than uncoated bars. The effect of various coating parameters on the ability of the epoxy-coating to provide corrosion protection was not discerned. They also reported that visible coating damage was the only distinct difference between the bent and straight epoxy-coated bars.

Sharafi et. al. [51] investigated the effect of fusion-bonded epoxy-coated rebars, two water to cement ratio (0.44 and 0.6) concrete and two rebar covers on rebar corrosion. They assessed the effectiveness of FBECR through laboratory accelerated testing and exposure to tidal zone, above ground and below ground conditions. They reported that the performance of concrete specimens reinforced with FBECR, both in the accelerated laboratory environment and at the severe exposure conditions, was encouraging and no symptoms of corrosion were noted.

Treadaway and Davies [52] subjected three types of reinforcement, in two concrete mixes at three levels of chloride contamination, to accelerated testing on a natural exposure site for a 5-year period. The types of reinforcement investigated were

FBEC, plain and galvanized steel, while the levels of chloride contamination were zero, 1.6 and 5.4% by weight of cement. They concluded that epoxy coating provided considerable protection to the steel for the length of the trial when exposed in concrete with high concentration of added chloride leading to significant practical advantage. They also observed that corrosion was spreading from points of defect in the coatings when the steel was embedded in concrete containing high levels of chlorides. They recommended a further study for the long-term implications of under-film corrosion.

A recent study was conducted at King Fahd University of Petroleum and Minerals. [53] to evaluate the corrosion-resistance and bond strength of steel bars coated with high performance vinyl acrylic primer and epoxy based-coatings. They compared the performance of these bars with plain steel bars. The results of this study indicated that the performance of vinyl acrylic coating, in terms of corrosion of steel, was not better than that of fusion-bonded epoxy coating. However, the bond between concrete and all coated bars was less than that for uncoated bars.

In an investigation conducted by Erdogdu and Bremner [54], a total of 48 concrete slabs were used in order to conduct field and laboratory testing on epoxy-coated bars in concrete. For the laboratory testing, a set-up simulating a marine environment was used, whilst for the field work, a severe exposure site was chosen. A total of 16 slabs, measuring 55x200x300 mm, containing uncoated bars were employed for comparison. Coated bars with different levels of surface damage (0, 1 and 2%) were used in the remaining slabs. Corrosion activity was monitored continuously, over a two-year period. They indicated that the corrosion rate of epoxy-coated rebars was negligible, regardless of the degree of damage to the coating. Similarly, the epoxy-coated rebars removed from the slabs at the end of one and two years of exposure showed no propensity to cause cracking and spalling due to corrosion products and no visible signs of corrosion were found on the surface of concrete.

In a study by McKenzie [55], the effect of defects in the commercially-produced epoxy-coated reinforcement was assessed. The conditions investigated included uncoated ends, repaired ends and bent bars. The experiment involved both salt-bonded specimens and specimens with salt inducted to the concrete mix. The author [55] concluded that the extent of concrete cracking and the severity of reinforcement corrosion were reduced for epoxy-coated reinforcement in chloride-contaminated test specimens in comparison with those containing uncoated reinforcement over a two year period. Corrosion did spread under the coating from defects in the epoxy. Reinforcement corrosion observed beneath the epoxy-coating was light surface rusting without loss in bar section, or peeling or blistering of the coating, whilst there was noticeable loss in bar section due to corrosion of uncoated bar.

Despite the reported beneficial effects of using FBEC bars, concerns have recently been raised regarding their effectiveness in preventing rebar corrosion and extending the useful service-life of structures [56]. In Florida, USA, some newly built bridges using epoxy-coated reinforcement exhibited signs of steel corrosion within 10 years of exposure [57]. Though the problem in Florida seems to be an isolated case, it puts forward several questions regarding the usefulness of FBEC bars. The Canadian Strategic Highway Research Program (C-SHRP) initiated a research project in 1990 [58] to determine the effectiveness and long-term (50 years or more) performance of fusion-bonded epoxy-coatings in preventing the corrosion of reinforcement in highway structures exposed to environments representative of the Canadian conditions. The preliminary field and laboratory test results suggested that fusion-bonded epoxy-coatings will not be effective in providing long-term corrosion protection to reinforcement in salt-contaminated concrete. Furthermore, Clear [58] indicated that the increase in life of epoxy-coated rebar structures over those constructed with uncoated rebars will be in the range of only 3 to 6 years; rather than the above 40 years previously estimated.

Some concerns have also been voiced at the ability of FBEC to inhibit the diffusion of chloride ions to the steel surface in highly concentrated chloride-bearing media. Rasheeduzaffar et. al. [59] conducted a 7-year exposure site program for corrosion resistance performance in chloride-bearing concrete using bare, galvanized, epoxy-coated and stainless-clad reinforcing steel bars. The bars were cast in prismatic specimens of 0.45 water-to-cement ratio, good-quality concrete. The specimens were contaminated with 0.6, 1.2 and 4.8% chloride ions by weight of cement. They observed that mild steel bars had suffered from severe corrosion in all specimens, while in concrete made with galvanized reinforcing steel, there was a delay in the onset of cracking, a reduction in metal loss and an amelioration in the incidence and severity of concrete failure. The epoxy-coated bars performed exceedingly well as corrosion-resistant steel in specimens contaminated with 0.6 and 1.2% of chlorides. No corrosion and concrete cracking were observed in these specimens. However, in the specimens contaminated with 4.8% of chlorides, a significant corrosion of the substrate under the coating, causing a systematic breakdown of the coating and cracking of concrete, was noted. The stainless-clad reinforcing bars exhibited the best performance. Their results indicated that the epoxy-coatings have a finite tolerance limit for chlorides.

To summarize, most of the literature reviewed provide a strong indication that fusion-bonded epoxy-coating is an effective corrosion control option. However, other views do indicate that in marine and highly concentrated chloride environments, the application of epoxy-coating has some limitations in long-term service. Further, there is not enough information in the literature on the effect of degree of coating damage or number of holidays on the performance and effectiveness of epoxy-coated reinforcing bars. Therefore, part of this research aimed at investigating the influence of these parameters on reinforcement corrosion.

CHAPTER 3

OBJECTIVES AND RESEARCH PROGRAM

3.1 OBJECTIVES

The review of the literature, presented in Chapter 2, indicates that there is no unanimous consensus on the quantum of reduction in bond strength of fusion-bonded epoxy-coating rebars (FBECR) and concrete. Furthermore, there is no sufficient data on the effect of heat-cool cycling on the bond strength. Moreover, the effect of degree of coating damage and number of holidays on reinforcement corrosion at varying levels of chloride contamination are not well documented.

This research program, is therefore, aimed at assessing the bond strength and durability performance of FBECR. In particular, the bond strength between FBECR and concrete under heat-cool cycling exposure, and the influence of holidays and coating damage on reinforcement corrosion were investigated.

The specific objectives of this research program were:

1. to evaluate the effect of bar size and coating thickness on the bond strength of fusion-bonded epoxy-coated bars,

2. to investigate the effect of heat-cool cycling on the bond strength of fusion-bonded epoxy coated bars; and

3. to evaluate the effect of number of holidays and the degree of surface damage of the FBE coating on reinforcement corrosion in chloride-bearing concrete.

3.2 RESEARCH PROGRAM

In order to achieve the above objectives, the experimental work was divided into two parts as follows:

1. Pull-out tests were conducted to assess the bond strength of fusion-bonded epoxy-coated bars. In this part of the study, the effect of coating thickness, bar diameter and heat-cool cycling on the bond strength were investigated. Two coating thicknesses along with two bar diameters were utilized in the bond studies. Coating thicknesses of 150 and 300 μm and bar diameters of 12 and 20 mm were used. Concentric pullout tests were conducted after 0, 30, 60 and 120 heat-cool cycles. The details of the experimental work pertinent to this part of the study are depicted in Fig. 3.1.

2. The effect of holidays and coating damage on corrosion of fusion-bonded epoxy-coated steel bars were investigated using salt-contaminated concrete specimens. In this part of the study, 16 mm diameter bars with a coating thickness of 150 μm were used. In order to study the effect of holidays, rebars with varying number of holidays (0, 1, 2, 3 per linear foot) were utilized. The

effect of coating damage on the durability of FBEC bars was evaluated by inducing three levels of damage (0.5, 1 and 1.5 %). The concrete specimens in this series were contaminated with 0.4, 1 and 2% chlorides by weight of cement. Additionally, a limited test was also conducted to examine the difference, if any, in corrosion resistance of FBEC bars with localized and distributed damage. This was done by casting three more specimens using a surface damage of 1.5% and contaminated with 1% chloride ions. Figure 3.2 illustrates the experimental work related to this part of the study.

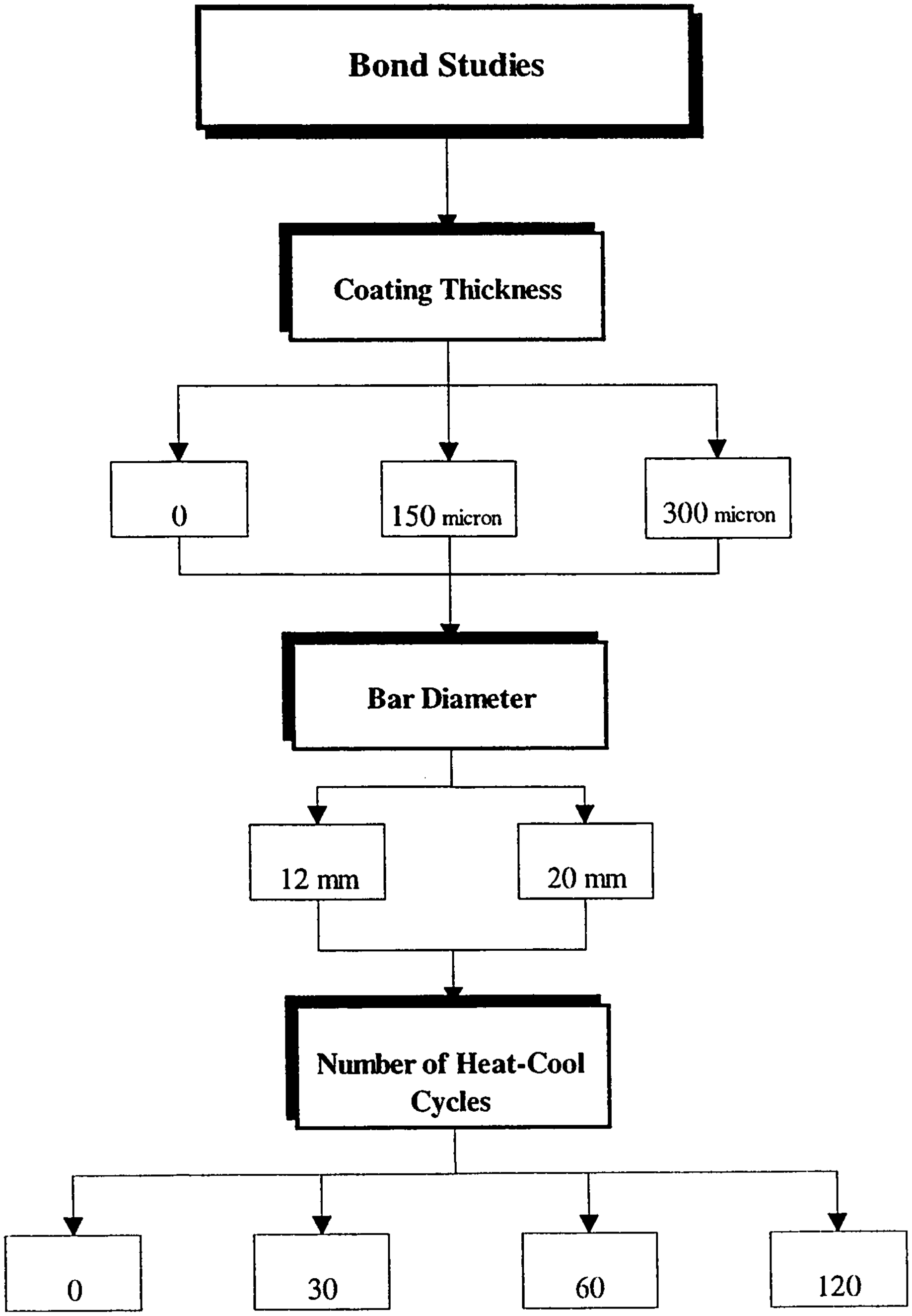

Fig. 3.1: Schematic Diagram Showing the Experimental Plan for Evaluating the Bond Strength of FBEC Steel Bars

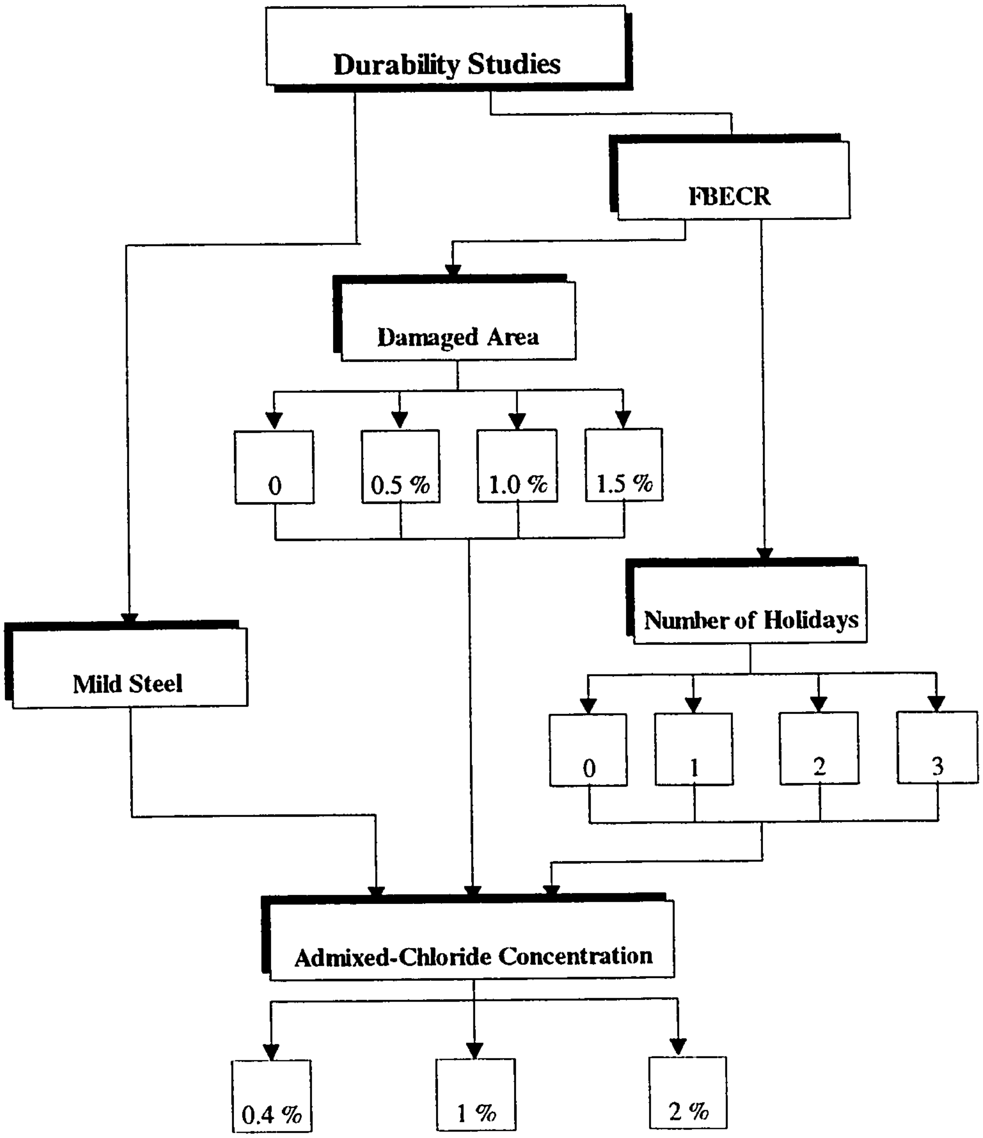

Fig. 3.2: Schematic Diagram Showing the Experimental Work for Evaluating the Corrosion Performance of FBEC Steel Bars

CHAPTER 4

METHODOLOGY OF RESEARCH

This Chapter outlines the materials and the experimental test methods utilized to achieve the objectives of this investigation. Whenever applicable, standard ASTM or BS test methods were adopted.

4.1 MATERIALS

4.1.1 Steel Bars

The deformed mild steel bars produced by the Saudi Iron and Steel Company (Hadeed) were used in all the tests. The bars were coated by a specialist local coating company. The coating was applied by electrostatic spraying as required by ASTM A 775/A 775M-93 [60]. Plate 4.1 shows some fusion-bonded epoxy-coated deformed bars used in this study. Table 4.1 shows the details of the bars used in each category.

The bar sizes (12, 16 and 20 mm) were selected as they represent the practical range of bars used in flexural members, while the 150 and 300 μm represent the upper and lower limits recommended by the ASTM A 775/A 775M-93 [60]. Though there is not enough information in literature about the degree of surface damage and number of holidays, a maximum of 1% surface damage and 2 holidays per linear foot (6 per linear meter) were specified [60]. Therefore, surface damages of 0.5, 1.0 and 1.5% and 1, 2 and 3 holidays per linear foot were selected to assess the influence of these parameters on reinforcement corrosion.

The prescribed levels of surface damage was achieved by intentionally damaging the coating till the steel surface was visible. This was done by using a scriber. Plates 4.2 shows typical fusion-bonded epoxy-coated steel bars with 0.5, 1.0 and 1.5% surface damage. The desired levels of holidays were created uniformly along the steel bars. Plate 4.3 illustrates the fusion-bonded epoxy-coated steel bars with 1, 2 and 3 holidays per linear foot.

4.1.2 Aggregates

Crushed limestone from Abu-Hadriyah with a maximum size of 12.5 mm was used as coarse aggregate. They were first sieved into different sizes and then washed with potable water to remove dust and salt contamination. It was then dried for 24 hours and, thereafter, stored. The coarse aggregate had a bulk specific gravity of 2.43 and an average absorption of 2.57%. Dune sand with a specific gravity of 2.64 and average absorption of 0.57% was used as fine aggregate.

The coarse aggregate was proportioned to conform to ASTM C 33 grading limits of size number 7. Table 4.2 shows the grading of the coarse aggregates used in this study, while Table 4.3 shows the fine aggregate grading.

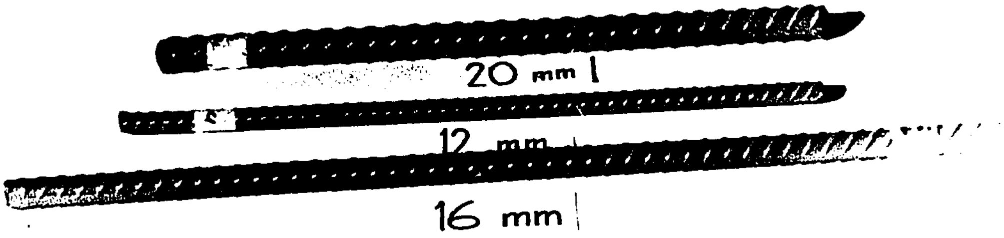

Plate 4.1: Some of the FBEC Steel Bars Used in the Study

Table 4.1: Details of Steel Bars Used in This Research Program

Diameter (mm)	Coating Thickness (µm)	Length (cm)	Number of Bars	Performed Test
12	0	40	12	Bond Strength
12	150	40	12	Bond Strength
12	300	40	12	Bond Strength
20	0	40	12	Bond Strength
20	150	40	12	Bond Strength
20	300	40	12	Bond Strength
16	0	55	9	Corrosion Resistance
16	150	55	63	Corrosion Resistance

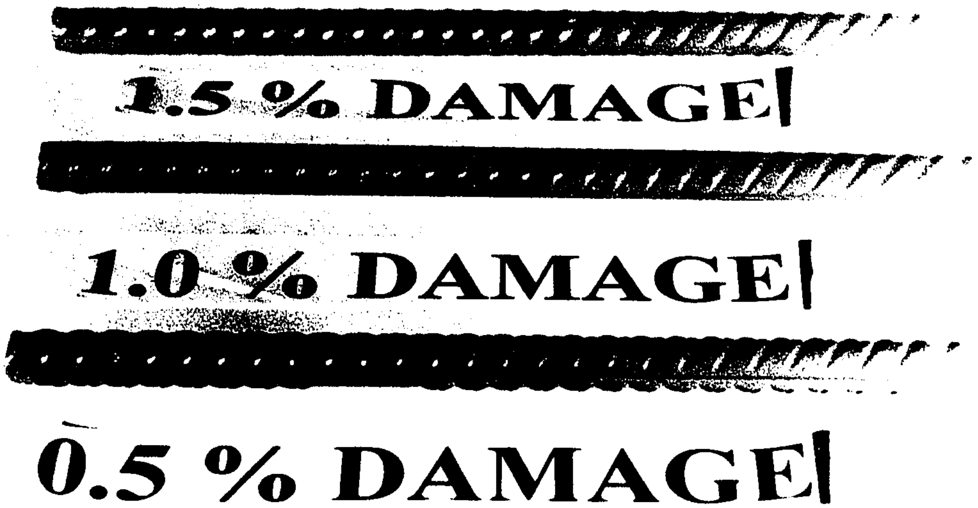

Plate 4.2: Typical FBEC Bars with 0.5, 1.0 and 1.5% Surface Damage

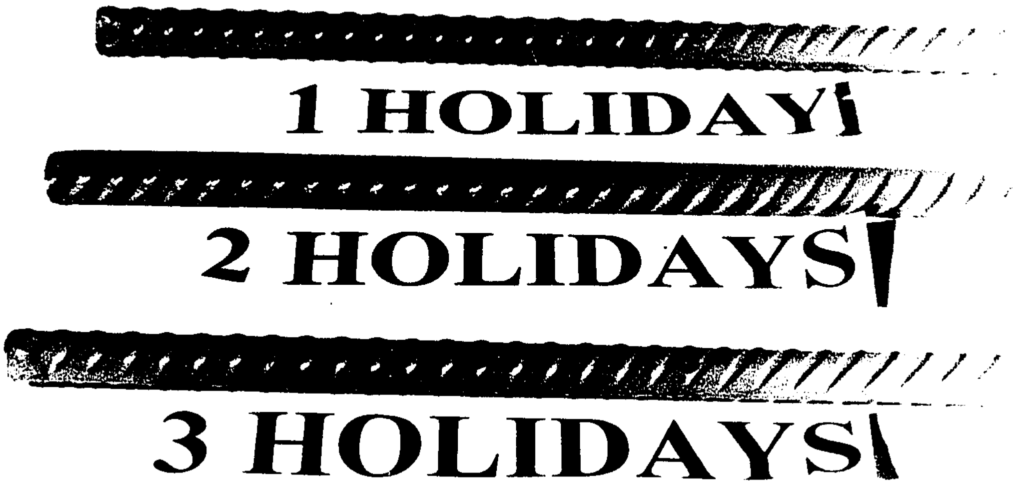

Plate 4.3: Typical FBEC Bars with 1, 2 and 3 Holidays

Table 4.2: Grain-Size Analysis of the Coarse Aggregate

Sieve Opening (mm)	% Retained	Commutative % Retained	% Passing
19.0	0	0	100
12.5	10	10	90
9.5	50	60	40
4.75	30	90	10
2.375	10	100	0

Table 4.3: Grain-Size Analysis of the Fine Aggregate

Sieve Opening	% Passing
1.2 mm	100.0
0.6 mm	96.2
0.3 mm	61.4
150 μm	21.9
75 μm	1.0

4.1.3 Cement

ASTM C 150 Type I cement was used throughout this investigation. The chemical composition of the cement used is shown in Table 4.4.

4.2 PREPARATION OF CONCRETE SPECIMENS

Reinforced concrete specimens with reference (black) and fusion-bonded epoxy-coated bars were used. Concrete cube specimens, 6x6x6 in. (152x152x152 mm), were used to determine the bond strength through concentric pullout tests. The durability of FBEC bars in chloride-contaminated concrete was evaluated using 4x2.5x12 in. (102x64x305 mm) prismatic concrete specimens. The concrete mixes were designed in accordance with the absolute volume method. The following mix design parameters were adopted:

Cement content	370 kg/m^3
Coarse aggregate / fine aggregate	1.6
Effective W/C ratio	0.45

4.2.1 Preparation of Concrete Specimens for Evaluating Bond Strength

The specimens used in the bond studies were cast by fixing the 12 or 20 mm diameter steel bars at the center of the 152x152x152 mm cube molds. A total of 72 Pull-out specimens was cast. Due to the limited number of molds available and the limited capacity of the drum mixer used, the specimens were cast in three different batches.

Table 4.4: Chemical Composition of Cement

Constituent	% by Weight
Compound Composition:	
Silicon Dioxide (SiO_2)	20.5
Aluminum Oxide (Al_2O_3)	5.6
Ferric Oxide (Fe_2O_3)	3.8
Calcium Oxide (CaO)	65.4
Magnesium Oxide (MgO)	2.1
Sulfur Trioxide (SO_3)	2.1
Loss on Ignition	0.7
Potassium Oxide (K_2O)	0.3
Sodium Oxide (Na_2O)	0.2
Phase Composition:	
Tricalcium Silicate (C_3S)	56.7
Dicalcium Silicate (C_2S)	16.1
Tricalcium Aluminate (C_3A)	8.5
Tetracalcium Aluminoferrite (C_4AF)	11.6

Each batch of concrete was used to prepare 24 specimens representing a certain coating thickness for both the 12 and 20 mm bars. Plates 4.4 and 4.5 show the steel molds with 12 and 20 mm steel bars, respectively.

The concrete was cast in two layers and consolidated on a vibrating table. After casting, the samples were cured in the laboratory air for 24 hours and then water-cured for fourteen days. Plate 4.6 shows the curing of the pull-out specimens. Six companion 3x6 in. (76x152 mm) cylindrical concrete specimens were cast from each batch and cured in a similar manner. These cylindrical specimens were used to determine the compressive strength of concrete. The embedment length for each group of samples was varied according to the diameter of the reinforcing steel. This was done to avoid premature failure of concrete because of the tensile stresses generated due to longer embedment lengths, and to avoid yielding of the steel bars. The embedment length for the 12 mm diameter bars was kept equal to six times the diameter, whilst for the 20 mm diameter bars, an embedment length of two times the bar diameter was used [62]. The required embedment length was controlled using polyvinyl chloride (PVC) sleeves to cover the unembedded length as shown in Plate 4.7. The gap between the reinforcing steel and sleeves was sealed with a silicon sealant. The details of a typical pull-out specimen are shown in Fig 4.1.

4.2.2 Concrete Specimens Used to Evaluate Reinforcement Corrosion

Prismatic concrete specimens (102x64x305 mm) with a 16 mm diameter reinforcing Bars were used to evaluate reinforcement corrosion. Plate 4.8 shows a typical prismatic steel mold used to cast these specimens.

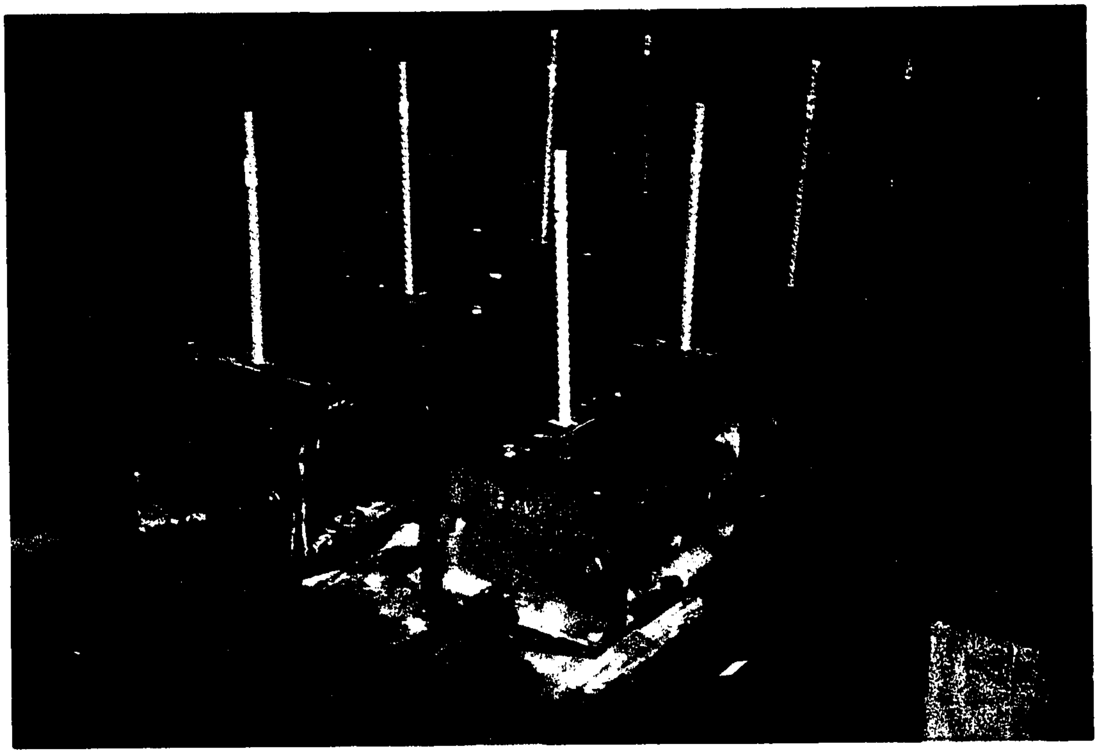

Plate 4.4: Typical Steel Molds with 12 mm Diameter Steel Bars

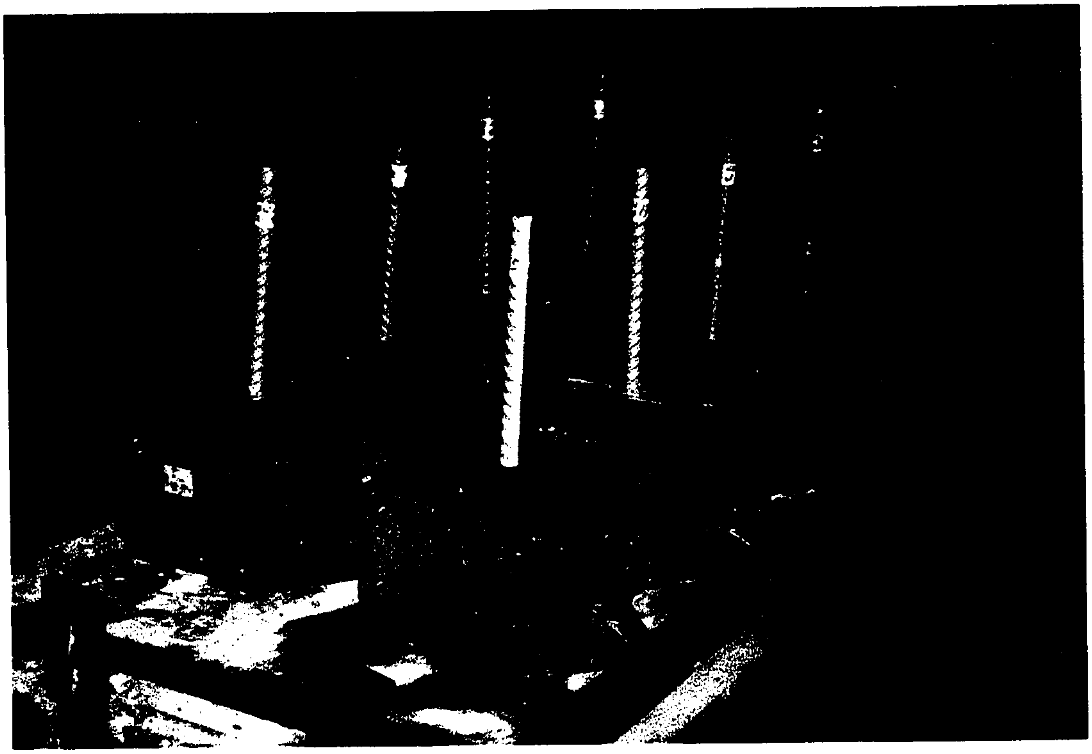

Plate 4.5: Typical Steel Molds with 20 mm Diameter Steel Bars

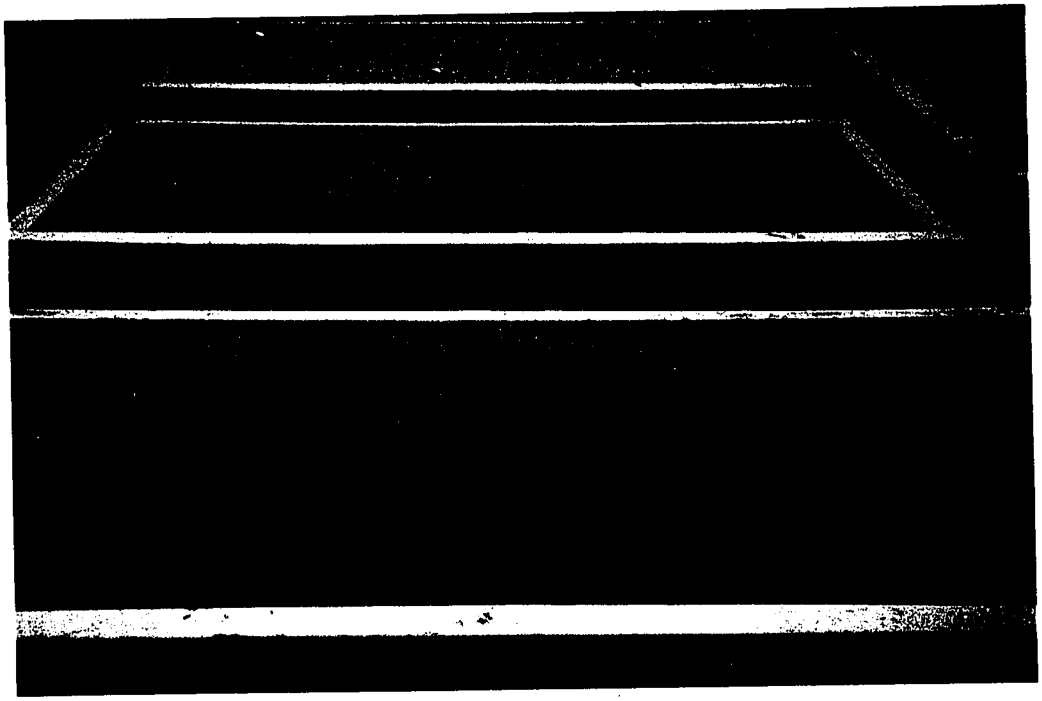

Plate 4.6: Curing of Pull-Out Specimens

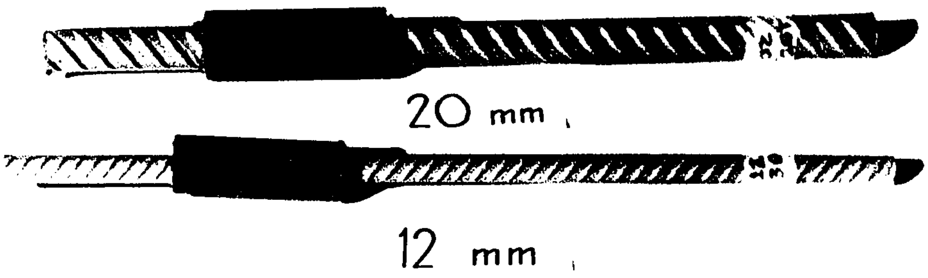

Plate 4.7: Typical Steel Bars with the Required Embedment Lengths

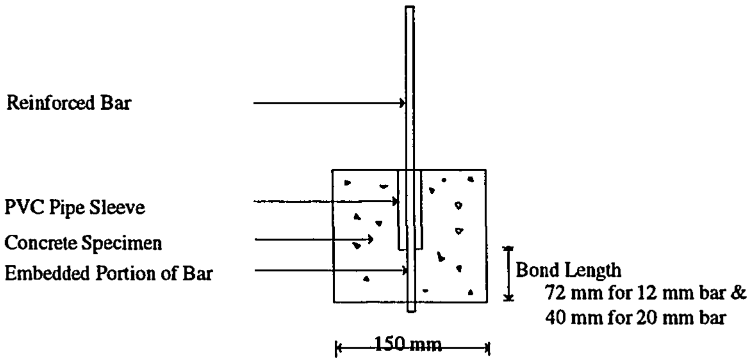

Fig 4.1: Details of a Typical Pull-out Specimen

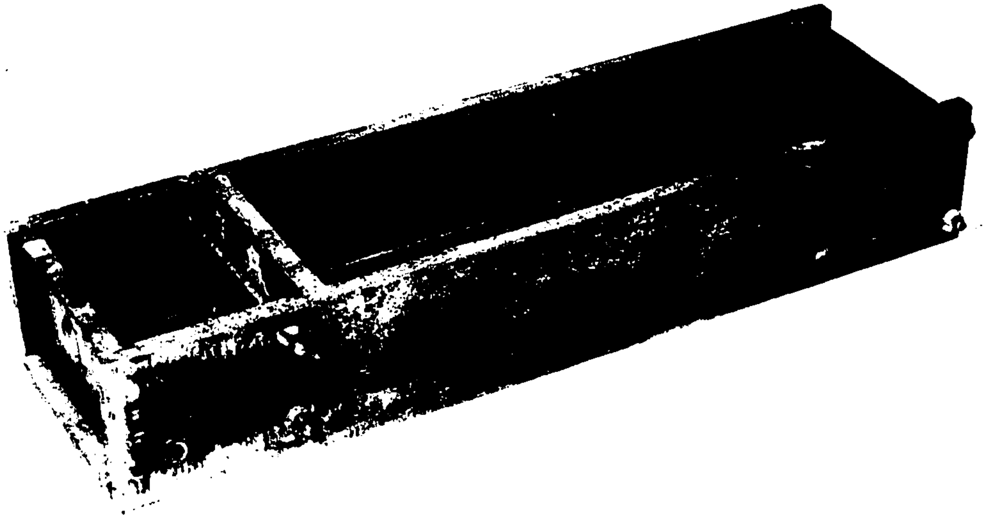

Plate 4.8: Typical Prismatic Steel Mold Used for Casting Reinforcement Corrosion Specimens

The steel bars had a concrete cover of 25 mm in the 305 mm direction. The ends of the steel bars were isolated from concrete by wrapping insulation electrical tapes. Figure 4.2 and Plate 4.9 shows typical specimens used in this part of the study.

The concrete was cast in a similar way to the pull-out specimens, except for the addition of admixed sodium chloride. Curing of the durability specimens was done by covering the specimens with wet cloth for 14 days. The concrete specimens contaminated with 0.4, 1 and 2% chlorides, were then partially submerged in solutions with 1 and 2 and 4% chloride levels as shown in Plate 4.10.

4.3 HEAT-COOL TREATMENT

In the bond studies, four groups of cube specimens, each consisting of 18 specimens, were exposed to 0, 30, 60 and 120 heat-cool cycles. The 18 specimens in each group consisted of 3 specimens representing coating thickness of 0, 150 and 300 µm and the two bar diameters used in this study (12 and 20 mm).

The heat-cool cycles were designed to represent the daily and seasonal variations in the Arabian Gulf region. The specimens were placed in an electric oven (Plate 4.11) and the temperature was gradually increased till it reached a maximum temperature of 70 ^{0}C (158 ^{0}F). It took about 4 hours for the oven to attain this temperature. The oven was maintained at 70 ^{0}C (158 ^{0}F) for two hours, and then the temperature was gradually decreased to room temperature in four hours. The oven was maintained at room temperature (23 ± 3 ^{0}C) for two hours, before another heat-cool cycle was initiated.

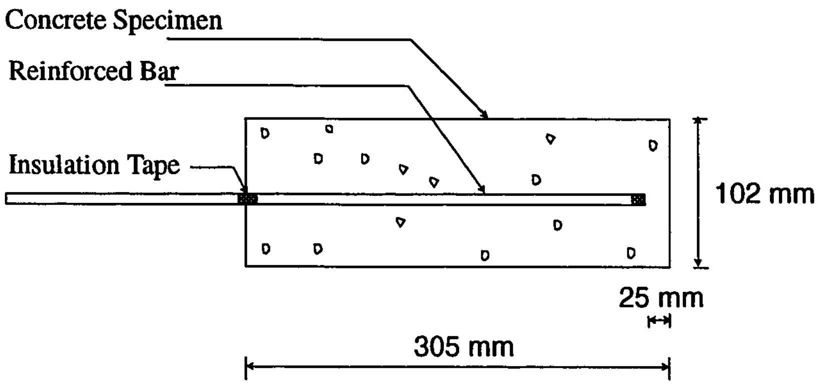

Fig. 4.2: Schematic Diagram of Concrete Specimen Used to evaluate Reinforcement Corrosion

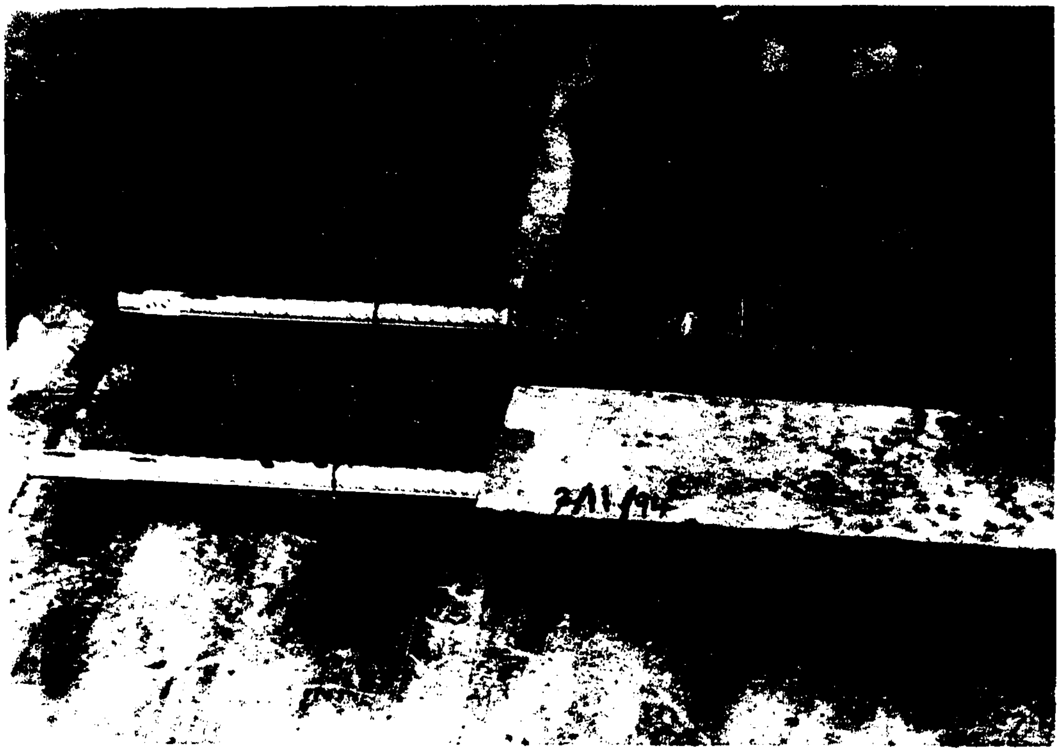

Plate 4.9: Typical Specimens Used to Evaluate Reinforcement Corrosion

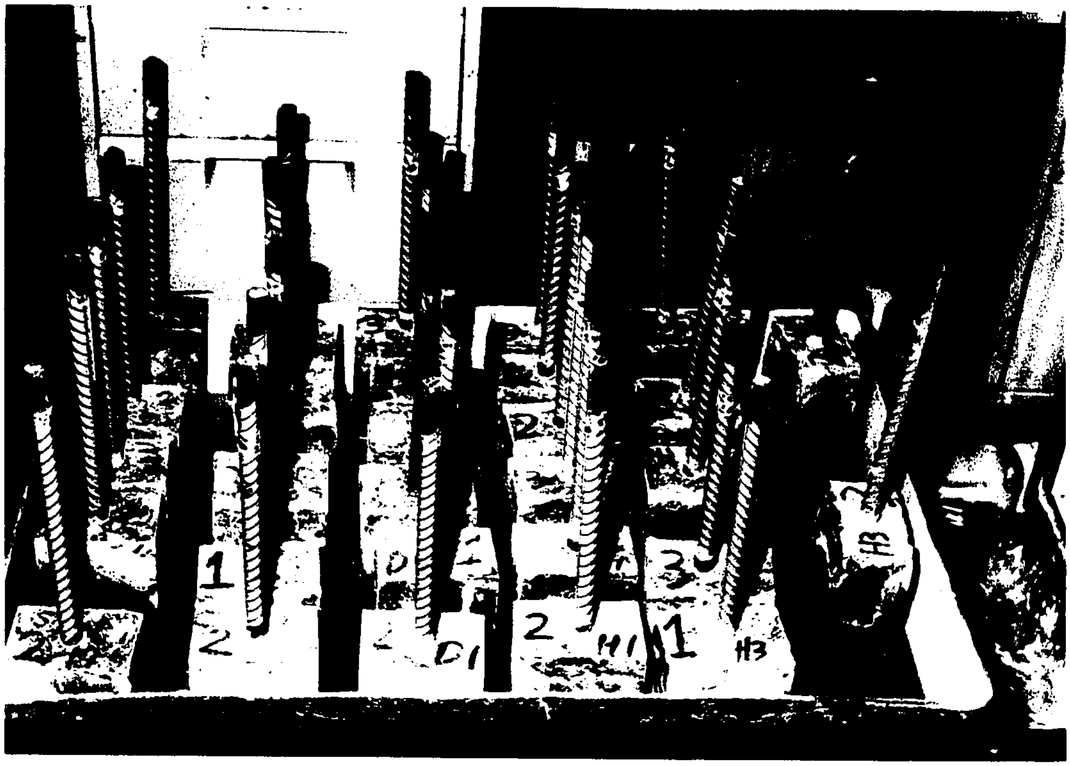

Plate 4.10: Reinforced Concrete Specimens Placed in the Chloride Solution

Plate 4.11: The Oven Used to Keep the Bond Specimens

4.4 TESTING

4.4.1 Testing for Bond Strength

For the bond strength, a total of 72 specimens was exposed to varying heat-cool cycles before being tested in bond. The concentric pullout test used in this experimental program was similar to the one outlined in ASTM C 234 [61], though this type of test does not represent the actual structural condition. In a reinforced concrete beam or a slab, the concrete surrounding the tensile reinforcement is in tension, whereas in the concentric pullout tests, the concrete is in compression. The eccentric pullout test is thought to be a more realistic representation of what happens in a beam. The modified cantilever beam tests solve some of the force field discrepancies of the concentric and eccentric pull-out tests [62]. The concentric pullout specimens used in this investigation suffer the disadvantage inherent in any pullout specimen. However, in view of its apparent simplicity, it has been widely used for comparative studies of bond strength behavior. Accordingly, the concentric pullout tests were used to develop a comparative assessment regarding the role of epoxy-coating thickness and the bar diameter on the bond strength.

A 250 kN capacity Instron testing machine was used for the pullout testing. Two linear variable displacement transducers (LVDT) were used to measure the loaded-end slip (LES) of the bar. Another transducer was used to measure the free-end slip (FES). A schematic diagram for the pullout test set-up is shown in Fig 4.3 and Plates 4.12 and 4.13.

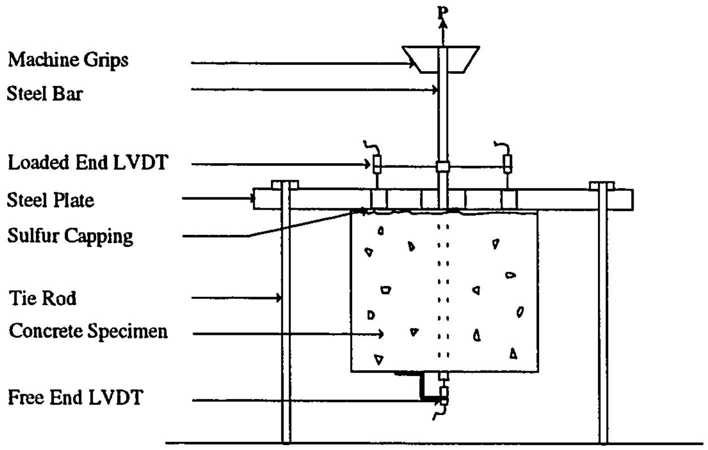

Fig. 4.3: Schematic Representation of the Pull-out Test Set-up

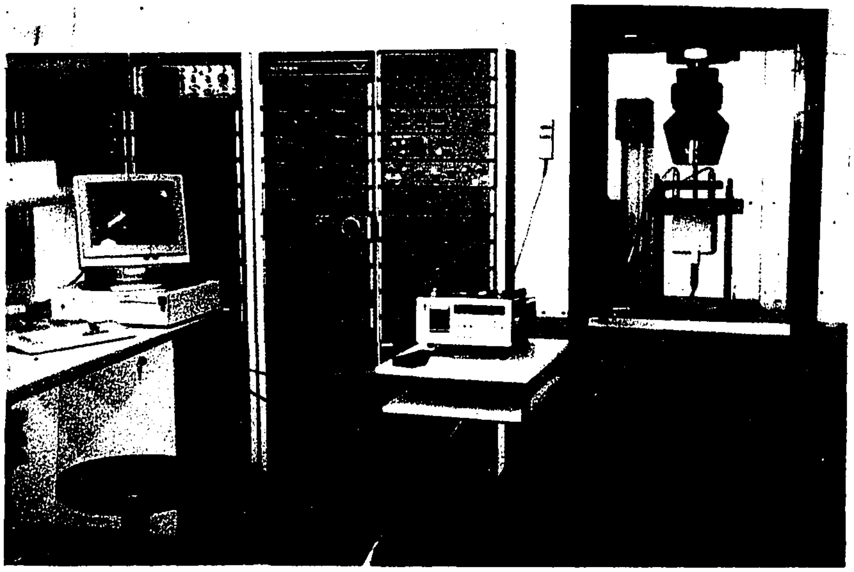

Plate 4.12: Pull-out Test Set-up

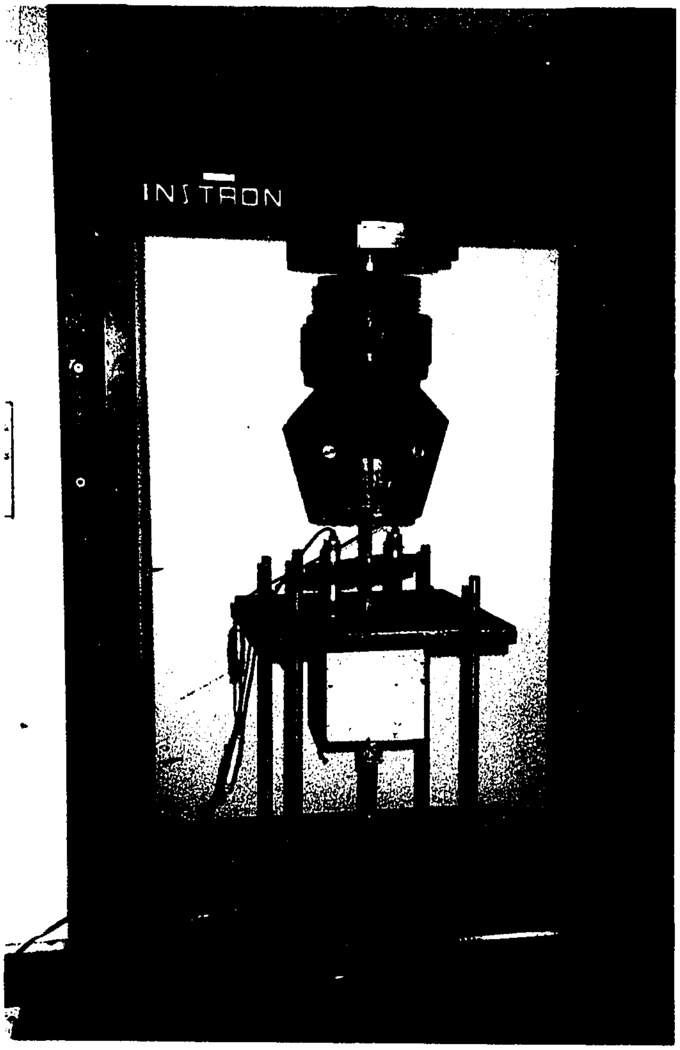

Plate 4.13: Typical Pull-out Specimen Fixed in the Testing Machine

For each of the samples tested in bond, both loaded-end slip and free-end slip were recorded and plotted against the bond stress. The maximum bond stress, and the corresponding loaded-end and free-end slips were obtained from those plots. The critical bond stresses corresponding to 0.002 in. (0.051 mm) free-end slip and 0.01 in. (0.254 mm) loaded-end slip were calculated. The ASTM C234 [61] recommends the use of the loaded-end slip criteria for 20 mm diameter bars. However, several researchers [44] have evaluated the critical bond strength using the 0.051 mm free-end slip especially, for the 12 mm diameter bars.

The bond stresses were calculated from external loads on the bar and the total surface area of the embedded portion of the bar, thereby representing an average value of stress along the bonded length of the bar [61]. This can be expressed by the following equation:

$$f_b = P/\Sigma_0 L \tag{4.1}$$

where

f_b = Average bond stress at each load level (MPa)
P = Applied load (N)
Σ_0 = Nominal perimeter of bars (mm)
L = Embedment length of bars (mm)

The LES was corrected for bar extension. This can be expressed by the following equations:

$$LES = \frac{LES(1) + LES(2)}{2} - \Delta L \qquad (4.2)$$

where:

$LES(1)$ and $LES(2)$ are the two slips measured at the loaded end by the two LVDTs (mm)

$\Delta L = \frac{PLo}{AE}$ = Correction for bar extension (mm)

P = Applied load (N)

Lo = Length of the bar between the concrete surface and the clamp holding the transducers (mm)

E = Modules of elasticity of steel (N/mm^2)

A = Cross sectional area of the steel bar (mm^2)

The percentage loss of bond between FBEC bars and concrete compared to the specimens with mild steel was evaluated as follows:

$$\% \text{ loss in bond stress} = \frac{f_{bcc} - f_{bcu}}{f_{bcu}} \times 100 \qquad (4.3)$$

where:

f_{bcc} = Critical bond stress between FBEC bars and concrete which corresponds to either 0.051 mm FES or 0.254 mm LES

f_{bcu} = Critical bond stress between reference (mild) steel bars and concrete which corresponds to either 0.051 mm FES or 0.254 mm LES

4.4.2 Reinforcement Corrosion

The prismatic concrete specimens (102x64x305 mm) reinforced with 16 mm diameter steel bars embedded at the center of the samples were used for corrosion studies. The concrete specimens had an effective cover of 25 mm at the bottom end. Three concrete specimens for each parameter were cast. A total of 72 specimens were prepared. The influence of the pinholes and surface damage of FBE coating on reinforcement corrosion was evaluated. Reinforcement corrosion was monitored by measuring the corrosion potentials and corrosion current density at periodic intervals.

The corrosion potentials were measured at two week intervals using a saturated calomel reference electrode (SCE) and a high impedance voltmeter. These measurements provide qualitative information on the state of passivity of the reinforcing steel [65]. The results obtained were categorized according to ASTM C 876 [63]. The potentials more negative than -275 mV with respect to the SCE indicate a probability of corrosion greater than 95%. Values less negative than -125 mV indicate a probability of corrosion below 5%, while those failing between -275 and -125 mV are not easily interpreted, and the probability of corrosion is uncertain.

The corrosion current density was measured by using the linear polarization resistance method (LPRM). This is a traditional DC technique for measuring the corrosion rate of steel. In this technique, the polarization resistance (Rp) is determined by conducting a linear polarization scan in the range of ±10 mV of the corrosion potential.

The corrosion current density is then calculated using the following Stern-Geary formula (64):

$$I_{corr} = B/Rp \qquad (4.4)$$

Where:

I_{corr} = corrosion current density, $\mu A/cm^2$

Rp = polarization resistance, $kOhms\text{-}cm^2$

$B = (\beta a * \beta c)/(2.3(\beta a + \beta c))$

βa and βc are the anodic and cathodic Tafel constants, respectively.

For steel in aqueous media, values of βa and βc of 100 mV are normally used. However, in the absence of sufficient data on βa and βc for steel in concrete, a value of B equal to 52 mV for steel in passive conditions and equal to 26 mV for steel in active conditions are normally used [65]. Andrade et al. [66] have indicated a good correlation between the weight loss determined using gravimetric and electrochemical methods when these values were used. Lambert et. al. [67] indicated a good correlation between the corrosion current density determined by the linear polarization resistance method and the gravimetric weight loss using these values.

To determine the resistance to linear polarization, the reinforcing steel bar was connected to the working electrode, and a stainless steel frame was used as a counter electrode. A saturated calomel electrode was used as a reference electrode. A potentiostat/galvanostat was used to polarize the steel (Plate 4.14).

The system was able to polarize the rebar (working electrode) in concrete by impressing a voltage through the counter electrode. The impressed voltage causes current to flow between the working and counter electrode. After the connections were made, the open-circuit potential (Ecorr) of the reinforcing steel was determined. Scanning was then initiated by changing the potential of the steel in concrete in a range of ±10 mV about Ecorr. The scan rate was 0.1 mV/s. During scanning the difference in potential and the corresponding current was recorded each 30 seconds The Ohmic drop between the working electrode and the reference electrode was compensated using a positive feed back technique.

The data at the end of the test was processed and the polarization resistance (Rp) was evaluated. After that, the corrosion current density (I_{corr}) was calculated using the Stern-Geary relationship and values of B equal to 52 mV for steel in passive conditions and equal to 26 mV for steel in active conditions [65].

In literature, the steel is considered to be in a passive state of corrosion if the corrosion current density is lower than 0.1 $\mu A/cm^2$ [68]. Erodogdu and Bremner [54] considered a long-term maintenance-free performance for steel if the magnitude of the corrosion current density is lower than 0.01 $\mu A/cm^2$ [54].

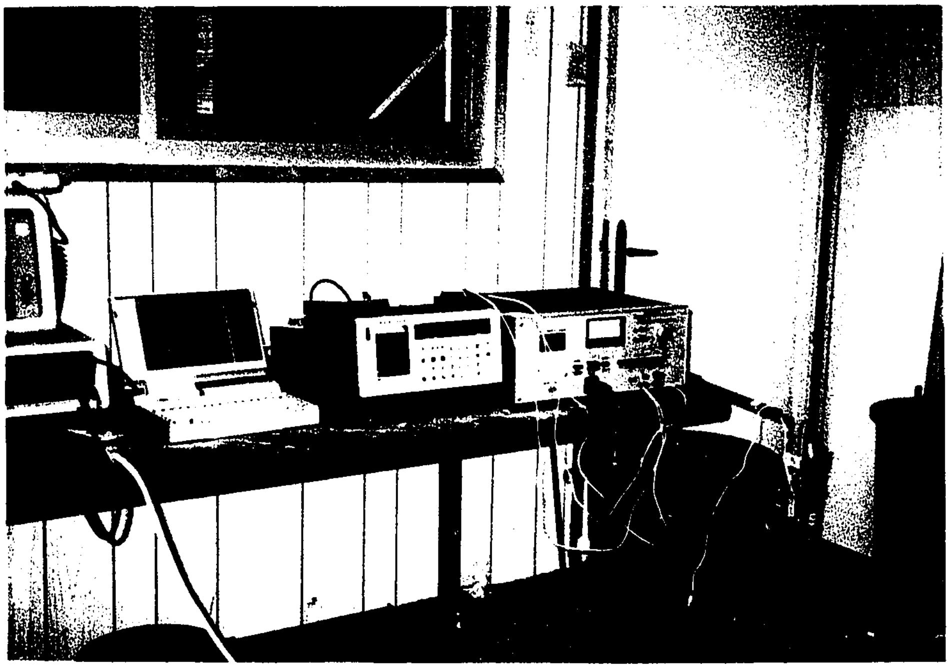

Plate 4.14: Layout of Corrosion Rate Measuring Instrument

CHAPTER 5

RESULTS PRESENTATION AND DISCUSSION

This Chapter is devoted to the presentation of results obtained in this study. The results are analyzed and discussed to examine the salient objectives of this work.

The test results pertinent to the bond strength are presented first, followed by a discussion of the results of corrosion potentials and corrosion current density tests.

5.1 BOND STRENGTH

This study was aimed at evaluating the effect of bar size, coating thickness and heat-cool cycling on the bond strength of fusion-bonded epoxy-coated steel bars in concrete. A total of 72 pull-out specimens were exposed to varying heat-cool cycles before being tested in bond (Section 4.4.1). Four groups of cube specimens, each consisting of 18 specimens, were exposed to 0, 30, 60 and 120 heat-cool cycles (Section 4.3). Each group contained specimens with reference (black) and fusion-bonded epoxy coated steel bars. The diameters of the steel bars used were 12 and 20 mm, while the coating thicknesses were 150 and 300 μm. For each of the samples tested in bond, the loaded-end and free-end slips were plotted against the bond stress as shown in Fig. 5.1.

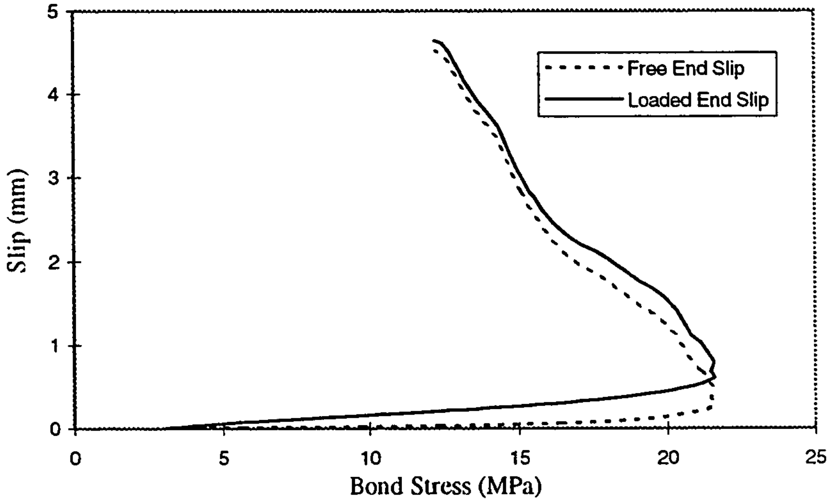

Fig. 5.1: Typical Bond Stress-Slip Relationship
(Bar Size=20 mm, Coating Thickness=150 μm)

The maximum bond strength, and the corresponding loaded-end and free-end slips were obtained from these plots. The critical bond stresses which corresponded to a free-end slip of 0.002 in. (0.051 mm) and a loaded-end slip of 0.01 in. (0.254 mm) were also evaluated.

The bond strength and slip data for the specimens exposed to normal temperature i.e., no heat-cool treatment are shown in Table 4.1. These data indicated that the ultimate bond strength values for the 12 mm diameter bars were in the range of 21.25 to 21.42 MPa. The ultimate bond strength in specimens with 20 mm diameter steel bars were in the range of 19.46 to 20.88 MPa. Further, the ultimate bond strength in the specimens with the coated bars (150 and 300 µm) was almost the same as that in the uncoated bars. However, higher slips were recorded in the specimens with coated bars, particularly those with a coating thickness of 300 µm.

It is also shown in Table 4.1 that the critical bond strength values for both 12 and 20 mm diameter bars decreased with the coating thickness. A higher reduction in critical bond strength was noticed in specimens with bars of 300 µm coating thickness.

5.1.1 Effect of Heat-Cool Cycling on Bond Strength

Three groups of cube specimens, each consisting of 18 specimens, were exposed to 30, 60 and 120 heat-cool cycles before being tested in bond. Results of pull-out tests for specimens in these groups, are presented in Tables 5.2 to 5.4. The FES, LES, ultimate and critical bond strengths represent average values of measurements conducted on three specimens, representing similar coating thickness and bar diameter. These data also indicated that the FBEC bars (150 and 300 μm) had almost the same ultimate bond strength as the uncoated ones. Further, higher slips were recorded in the specimens with coated bars. It was also noticed that the smaller number of heat-cool cycles (30 cycles), had virtually negligible adverse effect on ultimate bond strength. However as the number of cycles increased, reduction in bond strength was noted in all cases, as shown in Figures 5.2 and 5.3.

Although the critical bond strength values were evaluated using both FES and LES criteria, as shown in Tables 5.1 through 5.4, critical bond strength was taken as the bond stress corresponding to 0.254 mm loaded-end slip for specimens with 20 mm diameter steel bars [61], and 0.051 mm FES for specimens with 12 mm steel bars [44]. A comparison of data in Tables 5.1 through 5.4 indicated a slight increase in the critical bond strength after 30 heat-cool cycles. The critical bond strength values, however, decreased as the specimens were exposed to higher numbers of cycles, as shown in Figs. 5.4 and 5.5 for 20 and 12 mm diameter steel bars, respectively. This behavior may be attributed to the following factors:

i) Increase in the compressive strength at early period, due to elevated temperature.

ii) Deterioration of compressive strength due to temperature variations at later ages, per se after 60 thermal cycles.

Table 5.1: Ultimate Bond Strength and the Corresponding FES and LES in the Specimens Exposed to the Laboratory Temperature (No Heat-Cool Cycles)

Bar Diameter (mm)	Coating Thickness (μm)	Ultimate Bond Stress (MPa)	Max. FES (mm)	Max. LES (mm)	Critical Bond Stress at 0.051 mm FES (MPa)	Critical Bond Stress at 0.254 mm LES (MPa)
12	0	21.30	0.673	1.216	11.097	6.106
12	150	21.25	0.899	1.581	7.940	4.814
12	300	21.42	0.960	1.650	4.645	4.580
20	0	19.74	0.489	0.791	13.350	11.806
20	150	19.46	0.513	0.853	8.425	9.667
20	300	20.88	0.781	0.952	7.150	8.486

Table 5.2: Ultimate Bond Strength and the Corresponding FES and LES in Specimens Exposed to 30 Heat-Cool Cycles

Bar Diameter (mm)	Coating Thickness (μm)	Ultimate Bond Stress (MPa)	Max. FES (mm)	Max. LES (mm)	Critical Bond Stress at 0.051 mm FES (MPa)	Critical Bond Stress at 0.254 mm LES (MPa)
12	0	21.453	0.472	1.782	13.490	7.169
12	150	22.071	0.420	1.890	9.964	4.540
12	300	22.400	0.859	2.040	7.884	4.860
20	0	20.220	0.324	0.579	14.406	13.481
20	150	19.232	0.746	0.807	8.626	11.563
20	300	20.902	0.866	1.083	6.843	10.414

Table 5.3: Ultimate Bond Strength and the Corresponding FES and LES in Specimens Exposed to 60 Heat-Cool Cycles

Bar Diameter (mm)	Coating Thickness (μm)	Ultimate Bond Stress (MPa)	Max. FES (mm)	Max. LES (mm)	Critical Bond Stress at 0.051mm FES (MPa)	Critical Bond Stress at 0.254 mm LES (MPa)
12	0	20.753	0.612	1.591	12.162	5.097
12	150	21.081	0.721	1.504	8.821	4.960
12	300	21.876	0.771	2.185	8.034	3.250
20	0	19.493	0.655	0.855	13.231	12.90
20	150	19.131	0.591	1.102	8.660	11.539
20	300	19.718	0.896	1.145	6.997	10.925

Table 5.4: Ultimate Bond Strength and the Corresponding FES and LES in Specimens Exposed to 90 Heat-Cool Cycles

Bar Diameter (mm)	Coating Thickness (µm)	Ultimate Bond Stress (MPa)	Max. FES (mm)	Max. LES (mm)	Critical Bond Stress at 0.051 mm FES (MPa)	Critical Bond Stress at 0.254 mm LES (MPa)
12	0	20.172	0.418	1.705	12.350	5.387
12	150	20.776	0.463	2.232	9.647	5.206
12	300	20.975	0.957	2.275	8.332	5.162
20	0	18.349	0.718	0.794	9.022	12.283
20	150	18.678	0.780	0.996	7.786	11.401
20	300	17.553	0.942	1.140	7.112	10.697

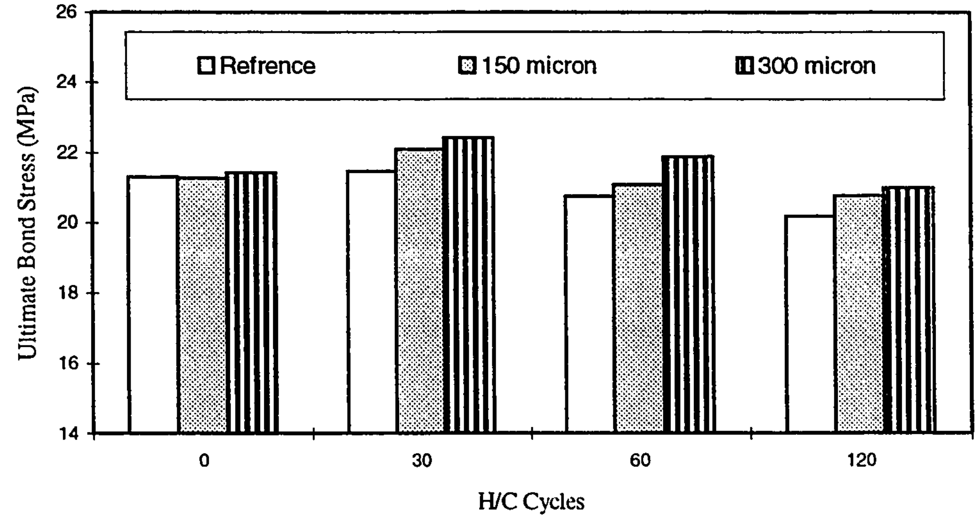

Fig. 5.2: Effect of H/C Cycling on Ultimate Bond Strength in concrete specimens with 12 mm Diameter Steel Bars

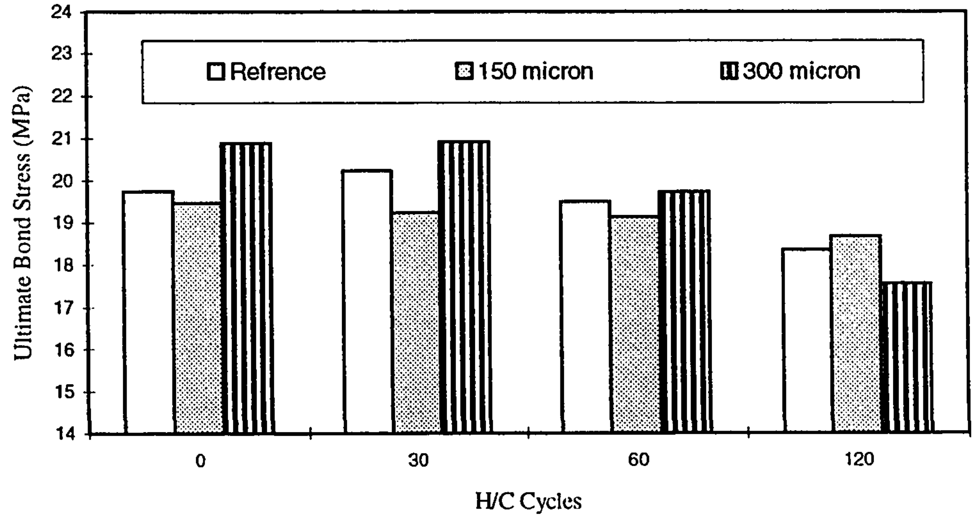

Fig. 5.3: Effect of H/C Cycling on Ultimate Bond Strength in Concrete Specimens with 20 mm Diameter Steel Bars

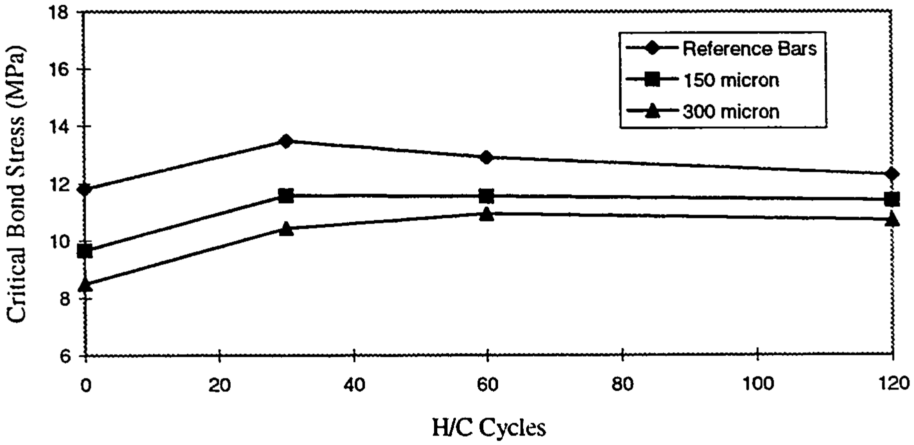

Fig. 5.4: Critical Bond Strength in Concrete Specimens with 20 mm Diameter Steel Bars

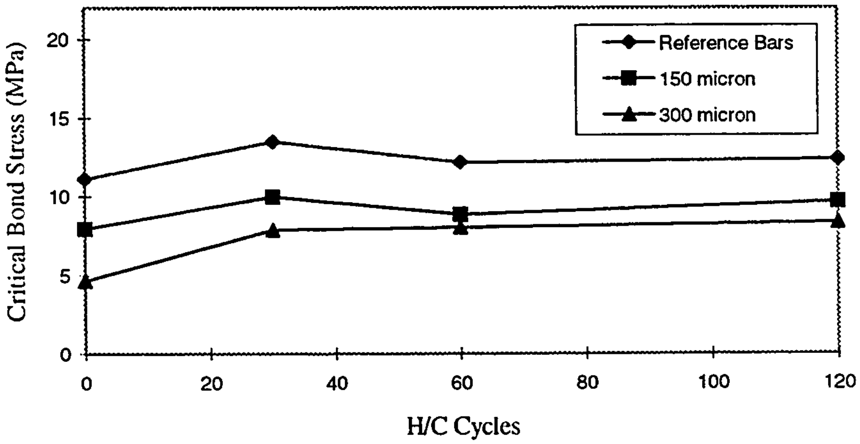

Fig. 5.5: Critical Bond Strength in Concret Specimens with 12 mm Diameter Steel Bars

The percentage reduction in the critical bond strength due to FBEC and heat-cool cycling is presented in Tables 5.5 and 5.6 for 20 and 12 mm diameter steel bars, respectively. These data indicated a reduction in the loss of critical bond strength, due to the use of epoxy-coating, as the number of heat-cool cycles was increased. This behavior was noticed for both 12 and 20 mm diameter bars and coating thickness of 150 and 300 μm. The percentage loss in the critical bond strength is further illustrated in Fig. 5.6 and 5.7. This behavior is mostly due to the fact that fusion-bonded epoxies are thermosetting polymers and they cannot be turned to a molten state by application of heat [2], thus insulating the steel bar. This can reduce the effect of heat-cool cycles in initiating micro-cracks due to thermal expansion-contraction of steel bars.

5.1.2 Effect of Coating Thickness on Bond Strength

It is evident in Figs. 5.4 and 5.5 that an increase in the coating thickness resulted in a decrease in the critical bond strength in concrete specimens with 12 and 20 mm diameter bars. The data in Tables 5.1 through 5.4 also indicate that the FES and LES for the 300 μm coating, recorded at the ultimate bond stress, were higher than those for the 150 μm coating. Further, the percentage reduction in the critical bond strength increased with coating thickness (Fig. 5.6 and 5.7), indicating the inverse effect of thicker coatings on the bond strength.

The reduction in the critical bond strength and increased slip associated with epoxy-coating of rebars may be attributed to the reduction of the chemical adhesion and

Table 5.5: Reduction in Critical Bond Strength for the 20 mm Diameter Bars due to FBEC

Coating Thickness (μm)	Reduction in Critical Bond Strength, %			
	0 H/C Cycles	30 H/C Cycles	60 H/C Cycles	120 H/C Cycles
150	18.15	14.23	10.55	7.18
300	28.12	22.75	15.31	12.91

Table 5.6: Reduction in Critical Bond Strength for the 12 mm Diameter Bars due to FBEC

Coating Thickness (μm)	Reduction in Critical Bond Strength, %			
	0 H/C Cycles	30 H/C Cycles	60 H/C Cycles	120 H/C Cycles
150	28.45	19.70	27.47	21.89
300	58.14	41.56	33.94	32.53

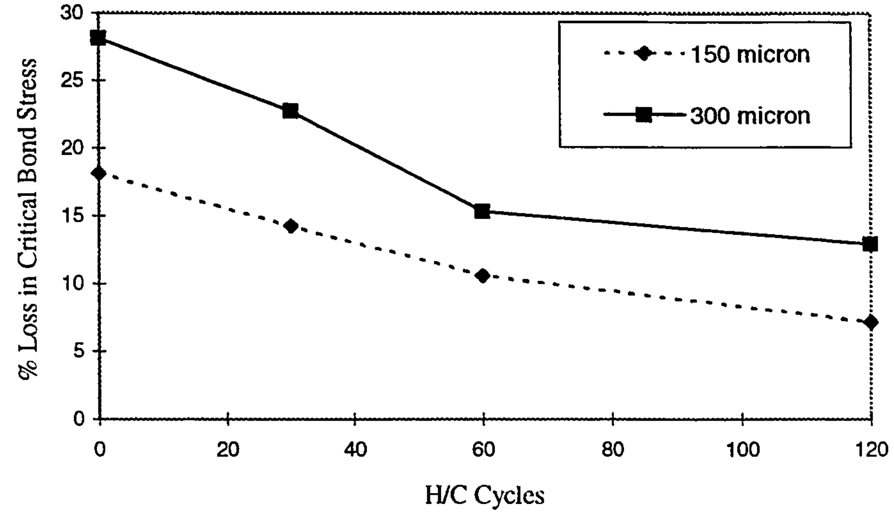

Fig. 5.6: Effect of H/C Cycling on the Reduction in the Critical Bond Strength in Concrete Specimens with 20 mm Steel Bars

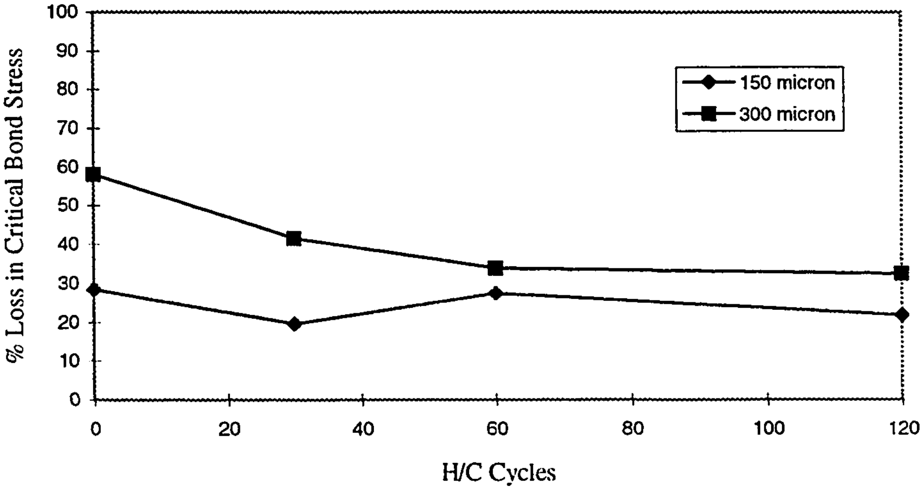

Fig. 5.7: Effect of H/C Cycling on the Reduction in the Critical Bond Strength in Concrete Specimens with 12 mm Steel Bars

frictional components of bond caused by the smoother surface of the coated bar [34]. Also, the higher slip observed in bars with thicker coatings may be attributed to a reduction in the lug height and hence the interlocking capacity of the bar is reduced.

5.1.3 Effect of Bar Diameter on Bond Strength

To further illustrate the effect of bar diameter on the percentage loss in critical bond strength, Tables 5.5 and 5.6, indicate that the percentage loss is higher for the specimens with 12 mm diameter bars than those with 20 mm diameter for the same coating thickness. This behavior is further illustrated in Figs. 5.8 and 5.9.

The reduction in the lug height is one of the reasons to cause the reduction in bond strength in the coated bars. This may be more predominant for smaller bar diameters.

In general, the data in Tables 5.1 to 5.6 indicated a reduction in critical bond strength due to the use of epoxy-coating. For the 20 mm diameter bars the reduction in the critical bond strength in specimens not exposed to thermal variations were 18.15 and 28.12% for 150 and 300 µm coating thickness, respectively. In the 12 mm diameter bars, the reduction was 28.45 and 58.14% for coating thickness of 150 and 300 µm, respectively. Up to 120 cycles, the heat-cool treatment had a significant influence on the reduction in critical bond strength for both 12 and 20 mm diameter bars.

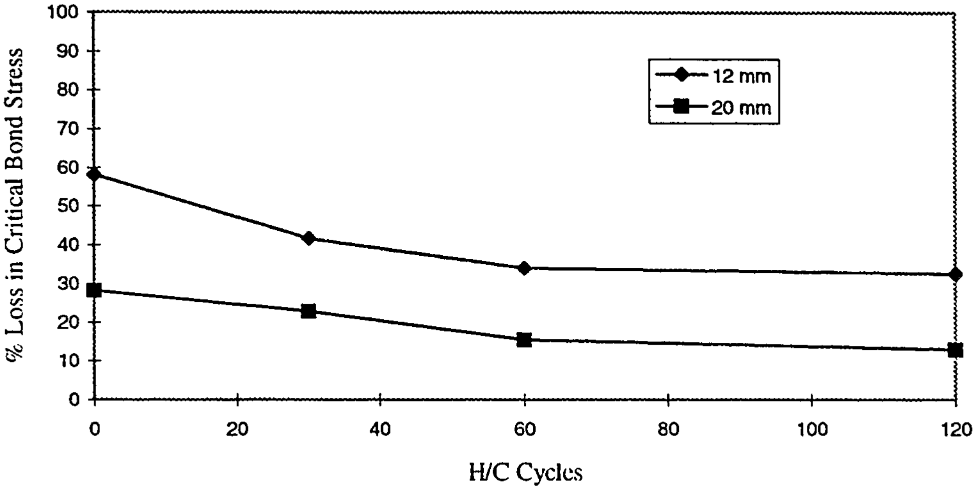

Fig. 5.8: Effect of H/C Cycles on the Reduction in the Critical Bond Stress in Concrete Specimens with 12 and 20 mm Bars and 300 μm Coating Thickness

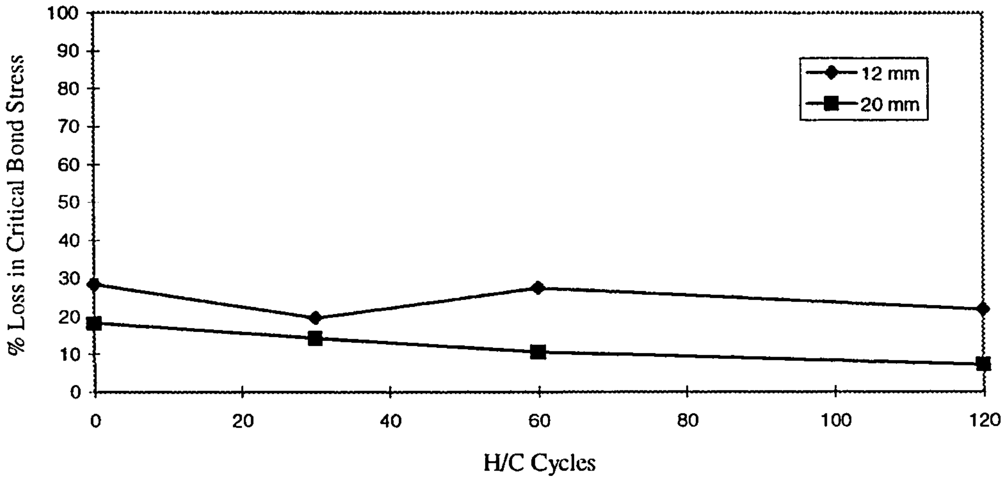

Fig. 5.9: Effect of H/C Cycles on the Reduction in the Critical Bond Stress in Concrete Specimens with 12 and 20 mm Bars and 150 μm Coating Thickness

5.2 CORROSION POTENTIALS

The corrosion of reinforcing steel was monitored by measuring the corrosion potentials and corrosion current density at regular intervals. The time-corrosion potential curves for the reference (black) steel reinforced specimens are shown in Fig. 5.10 for specimens contaminated with 0.4, 1 and 2% chlorides. The corrosion potentials decreased with the exposure period for specimens contaminated with 0.4% chlorides. The time to initiation of reinforcement-corrosion, based on ASTM C 876 criterion of -350 mV CSE or -270 mV SCE [63], was indicated after 115 days. In specimens contaminated with 1 and 2% chloride ions, Fig. 5.10 indicated active corrosion of reinforcement from the beginning of exposure.

The time-corrosion potential curves for FBEC bars are plotted in Figures 5.11 through 5.16. In general, these curves indicated high initial potentials in specimens with FBEC steel bars. Most of the potentials, particularly in specimens with 1 and 2% chlorides, did not change with time. No significant drop in the readings was noted in all the specimens except those with 3 holidays, 0.5% surface damage and the defect-less bars in specimens contaminated with 0.4% chlorides. Additionally, all these curves indicated active corrosion, based on ASTM C 876 criterion of -270 mV SCE. The high corrosion potentials recorded in the FBEC bars may be attributed to the high resistance of the coating. Several researchers [70] have indicated the unsuitability of ASTM C 876 criteria for evaluating reinforcement corrosion. Further, it may not be out of place to mention that, while the corrosion potentials provide a qualitative indication of reinforcement corrosion, the quantitative information can only be obtained by measuring the corrosion current density.

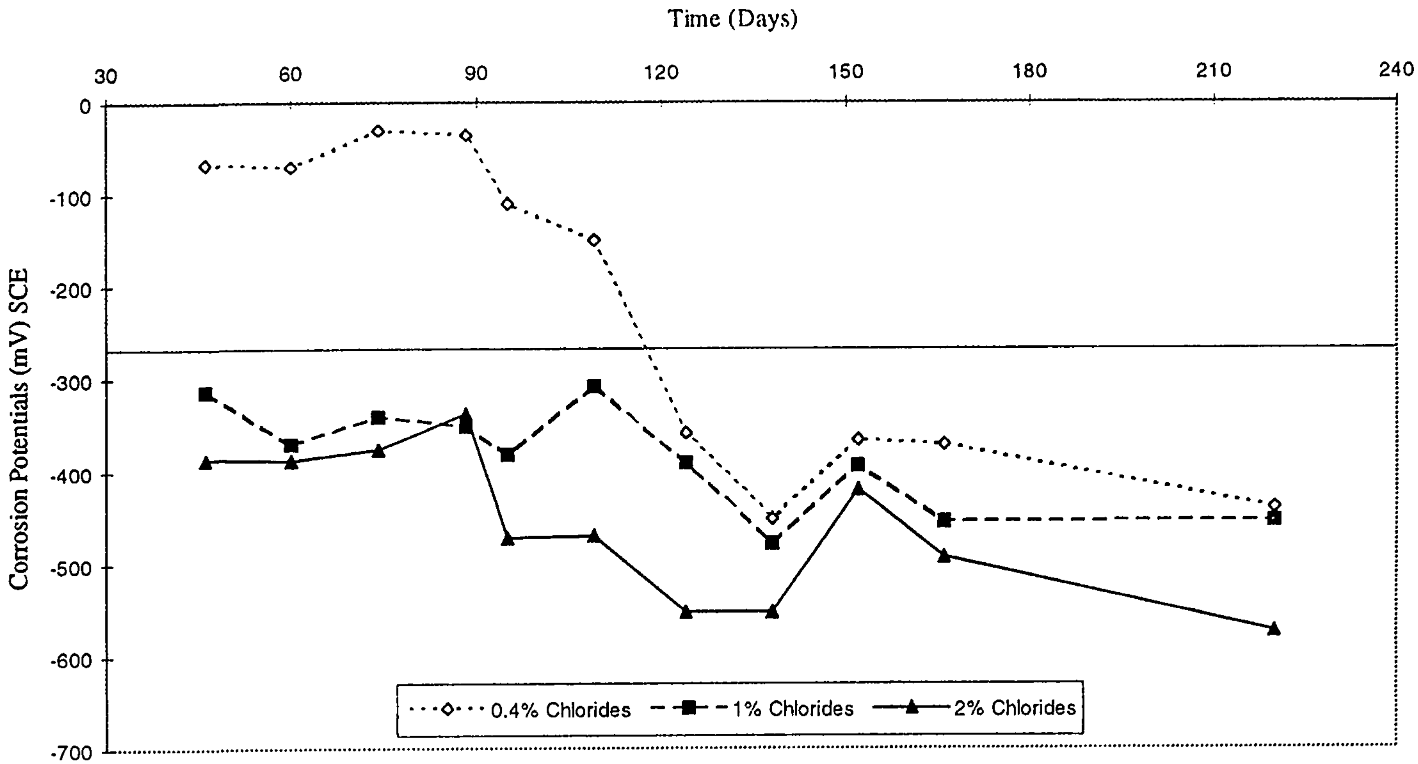

Fig. 5.10: Corrosion Potentials on Uncoated Steel in Concrete Specimens Contaminated with 0.4, 1 and 2% Chloride Ions

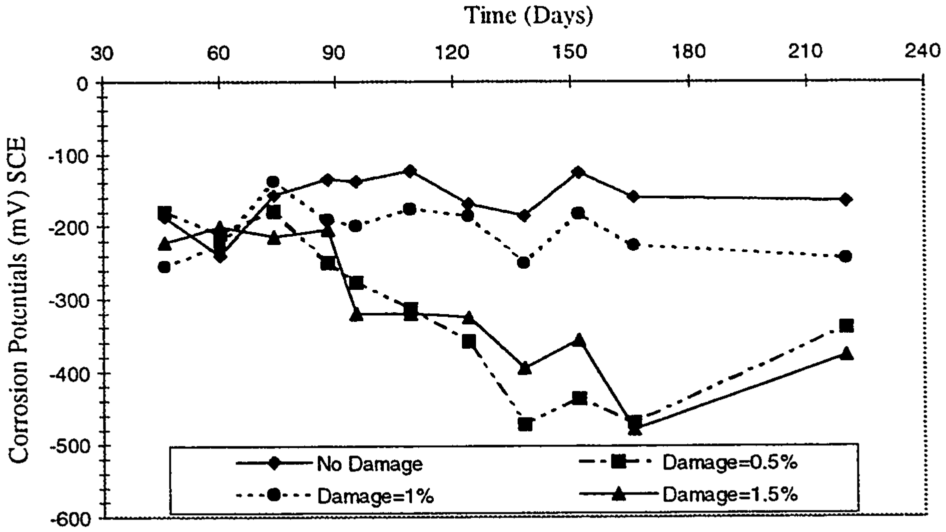

Fig. 5.11: Corrosion Potentials on Coated Steel with and without Surface Damage (Concrete Specimens Contaminated with0.4% Chloride Ions)

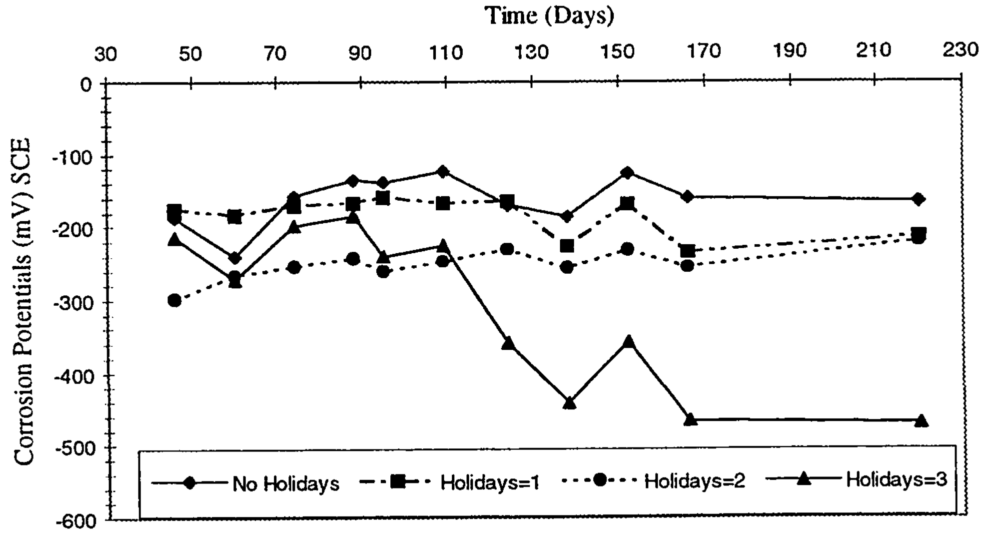

Fig. 5.12: Corrosion Potentials on Coated Steel with and without Holidays (Concrete Specimens Contaminated with 0.4% Chloride Ions)

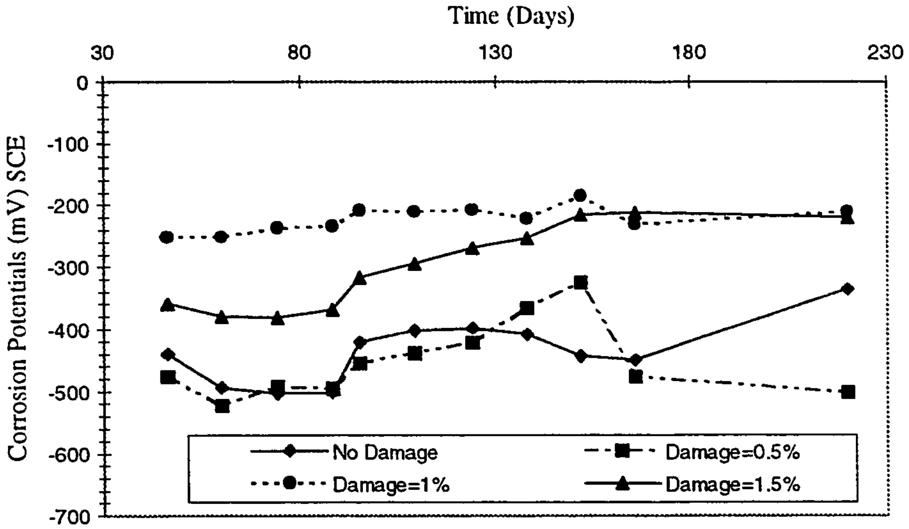

Fig. 5.13: Corrosion Potentials on Coated Steel with and without Surface Damage (Concrete Specimens Contaminated with 1% Chloride Ions)

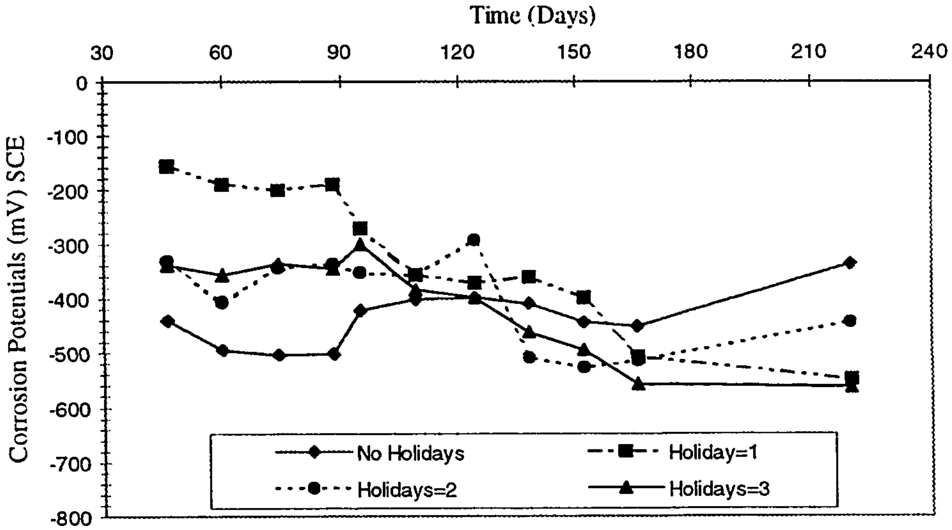

Fig. 5.14: Corrosion Potentials on Coated Steel with and without Holidays (Concrete Specimens Contaminated with 1% Chloride Ions)

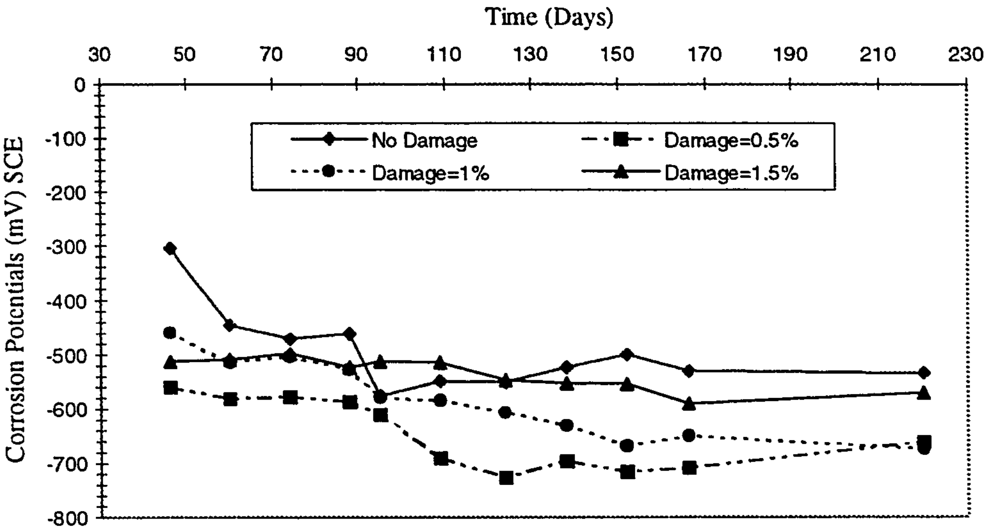

Fig. 5.15: Corrosion Potentials on Coated Steel with and without Surface Damage (Concrete Specimens Contaminated with2% Chloride Ions)

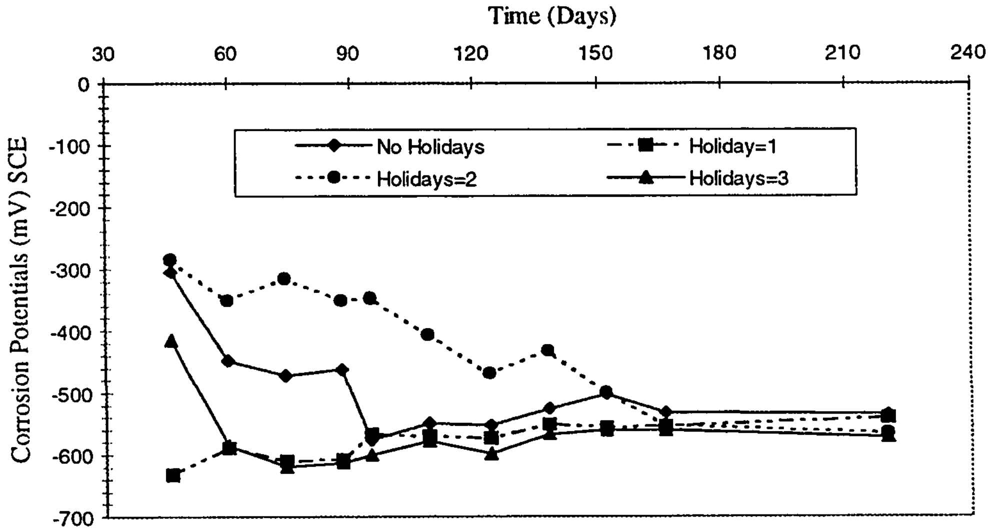

Fig. 5.16: Corrosion Potentials on Coated Steel with and without Holidays (Concrete Specimens Contaminated with 2% Chloride Ions)

5.3 CORROSION CURRENT DENSITY

The corrosion current density was evaluated periodically by measuring resistance to polarization (Rp). A typical linear polarization resistance plot is shown in Fig. 5.17. The corrosion current density (Icorr) was calculated using Equation 4.4.

5.3.1 Effect of Coating on Reinforcement Corrosion

The corrosion current density on uncoated and fusion-bonded epoxy-coated bars are shown in Table 5.7 and Fig. 5.18 through 5.20. These values are the average of measurements conducted on three specimens, representing similar bars and contamination levels. It was noted that for all the chloride levels, the fusion-bonded epoxy-coated steel bars were in passive state, the Icorr was less than the threshold value of 0.1 $\mu A/cm^2$ [68]. These results indicate the invalidity of the potential measurements.

The data on Icorr, Figure 5.18 and 5.20, indicated an active corrosion on uncoated bars contaminated with 1 and 2% chloride ions after about 55 days of exposure. In the specimens contaminated with 0.4% chloride ions, corrosion initiation, based on an Icorr of more than 0.2 $\mu A/cm^2$, was indicated after about 120 days. This time is very near to the value of 115 days indicated by the corrosion potential measurements. However, the British Standard BS8110 allows a maximum chloride content of 0.4%, while the American Building Codes (ACI 318-85) limit the water soluble chlorides to 0.15% by weight of cement.

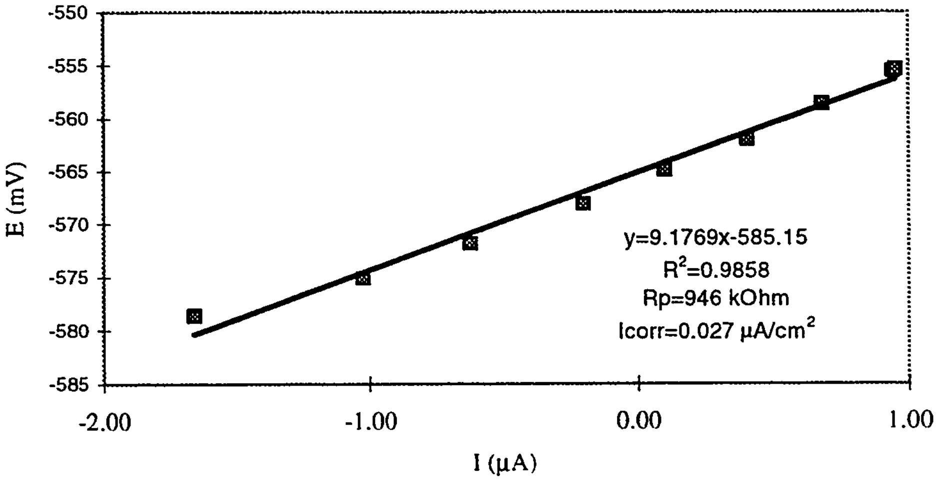

Fig. 5.17: Typical Linear Polarization Resistance Plot (2% Chlorides, 1% Surface Damage, 105 Days of Exposure)

Table 5.7: Corrosion Current Density on Steel in Specimens with Uncoated and Coated Steel Bars

Time	Corrosion Current Density ($\mu A/cm^2$)					
	0.4% Chlorides		1% Chlorides		2% Chlorides	
(Days)	Uncoated	Coated	Uncoated	Coated	Uncoated	Coated
55	0.029	6.6 E-04	0.193	3.4 E-03	0.186	5.0 E-04
80	0.047	8.0 E-04	0.290	3.0 E-03	0.241	2.5 E-03
105	0.076	8.3 E-04	0.254	2.7 E-03	0.353	2.4 E-03
130	0.345	8.7 E-04	0.691	3.2 E-03	0.881	3.4 E-03
155	0.335	10.0 E-04	0.803	1.4 E-03	0.690	4.0 E-03
180	0.783	10.0 E-04	1.110	3.8 E-03	1.262	4.8 E-03
230	0.703	11.0 E-04	1.112	2.1 E-03	1.300	3.9 E-03

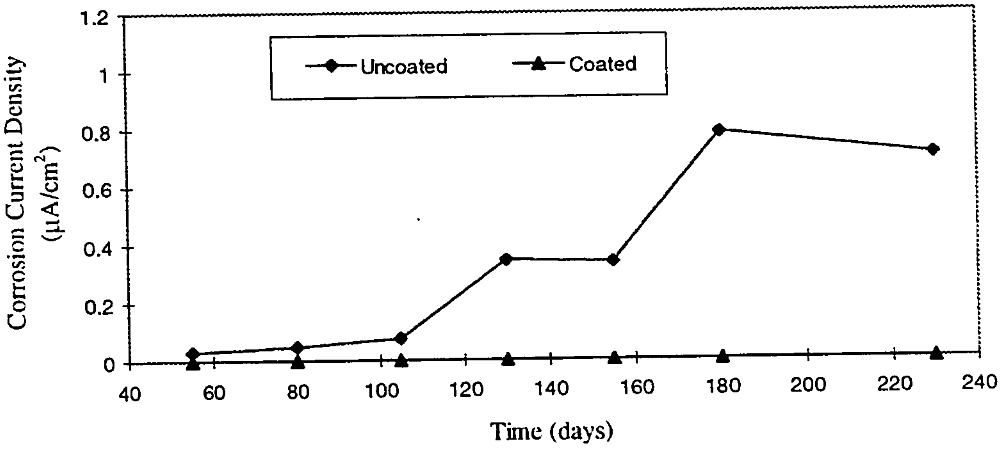

Fig 5.18: Variation of Corrosion Current Density on Steel with Time in Concrete Contaminated with 0.4% Chlorides by Weight of Cement

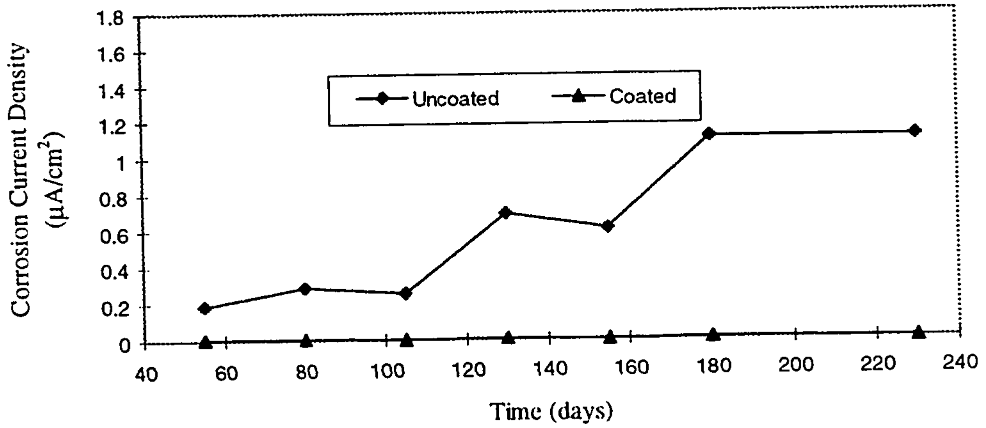

Fig 5.19: Variation of Corrosion Current Density on Steel with Time in Concrete Contaminated with 1% Chlorides by Weight of Cement

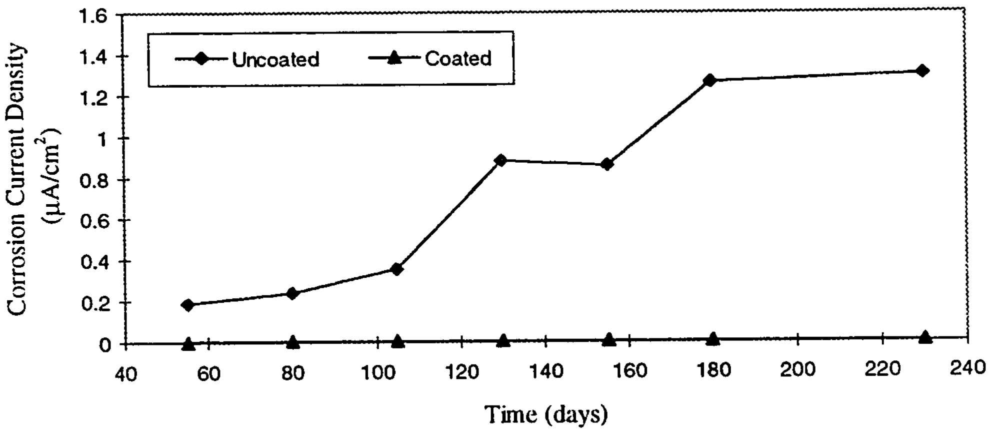

Fig 5.20: Variation of Corrosion Current Density on Steel with Time in Concrete Contaminated with 2% Chlorides by Weight of Cement

5.3.2 Effect of Surface Damage on Reinforcement Corrosion

The corrosion current density on FBEC bars with different levels of surface damage (0, 0.5, 1 and 1.5%) are presented in Tables 5.8 through 5.10 and Figs. 5.21 through 5.23 for the three levels of chlorides 0.4, 1 and 2%, respectively.

These data indicated that for all chloride contamination levels, the corrosion current density increased with the surface damage. However, all FBEC steel specimens are in passive state, with insignificant corrosion rates. Other observations are further identified based on the magnitude of corrosion current density higher than 0.01 μ A/cm^2 (0.01 mA/ft^2), which is considered as the maximum value for long-term maintenance-free performance [54]. These observations are as follows:

i. For 0.4 and 1% chloride contamination, the Icorr values were well below the 0.01 μA/cm^2 limit for all the three levels of damage (0.5, 1 and 1.5%). However, in specimens contaminated with 1% chloride, the Icorr values tended to approach a value of 0.01 μA/cm^2 after 230 days of exposure in specimens with 1 and 1.5% surface damage.

ii. For 2% chloride level, the Icorr values on steel with 0.5, 1 and 1.5% surface damage were higher than the 0.01 μA/cm^2 limit.

The above observations indicated that the chloride level plays an important role in determining the maximum allowable surface damage. ASTM A 775 [60] specifies a maximum of 1% surface damage without any information about the

Table 5.8: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 0.4% Chlorides

Time (Days)	Corrosion Current Density $\mu A/cm^2$			
	No Damage	0.5% Surface Damage	1% Surface Damage	1.5% Surface Damage
55	7.0 E-04	1.0 E-03	1.4 E-03	2.5 E-03
80	8.0 E-04	2.0 E-03	2.3 E-03	4. E-030
105	8.3 E-04	1.9 E-03	2.4 E-03	4.8 E-03
130	8.7 E-04	1.2 E-03	2.4 E-03	4.7 E-03
155	10.0 E-04	1.3 E-03	2.4 E-03	5.2 E-03
180	10.0 E-04	2.3 E-03	3.1 E-03	6.5 E-03
230	11.0 E-04	2.4 E-03	2.6 E-03	6.4 E-03

Table 5.9: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 1% Chlorides

Time (Days)	Corrosion Current Density ($\mu A/cm^2$)			
	No Damage	0.5% Surface Damage	1% Surface Damage	1.5% Surface Damage
55	3.4 E-03	1.5 E-03	4.5 E-03	7.0 E-03
80	3.0 E-03	1.7 E-03	6.3 E-03	9.7 E-03
105	2.7 E-03	3.3 E-03	4.4 E-03	7.8 E-03
130	3.2 E-03	5.7 E-03	4.4 E-03	8.4 E-03
155	1.4 E-03	2.1 E-03	3.7 E-03	6.1 E-03
180	3.8 E-03	5.2 E-03	9.0 E-03	9.7 E-03
230	2.1 E-03	5.8 E-03	7.7 E-03	8.8 E-03

Table 5.10: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 2% Chlorides

Time (Days)	Corrosion Current Density (μA/cm^2)			
	No Damage	0.5% Surface Damage	1% Surface Damage	1.5% Surface Damage
55	5.0 E-04	9.4 E-03	1.6 E-02	1.9 E-02
80	2.5 E-03	7.9 E-03	1.4 E-02	2.2 E-02
105	2.4 E-03	1.7 E-02	2.3 E-02	3.9 E-02
130	3.4 E-03	1.0 E-02	2.2 E-02	3.5 E-02
155	4.0 E-03	1.4 E-02	2.6 E-02	3.1 E-02
180	4.8 E-03	1.6 E-02	3.2 E-02	4.3 E-02
230	3.9 E-03	2.4 E-02	3.3 E-02	4.5 E-02

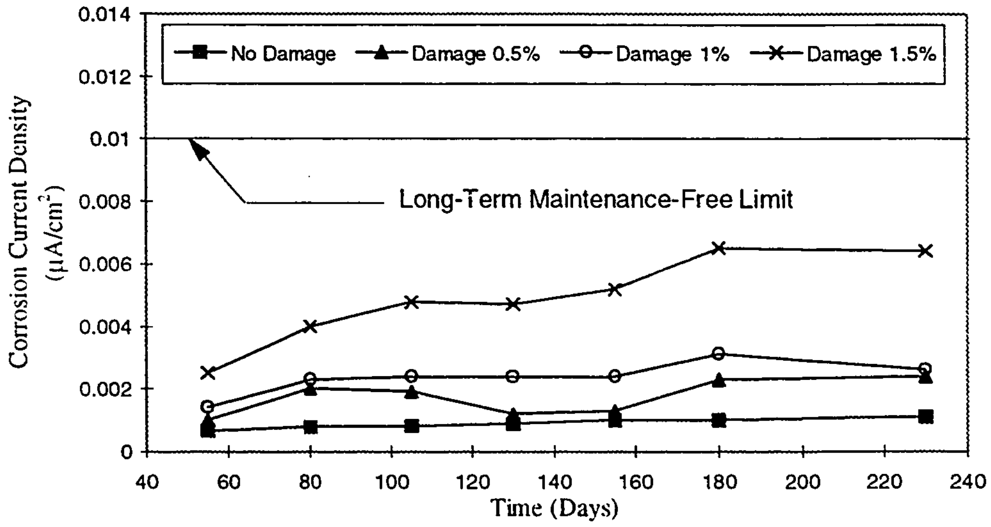

Fig 5.21: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 0.4% Chlorides

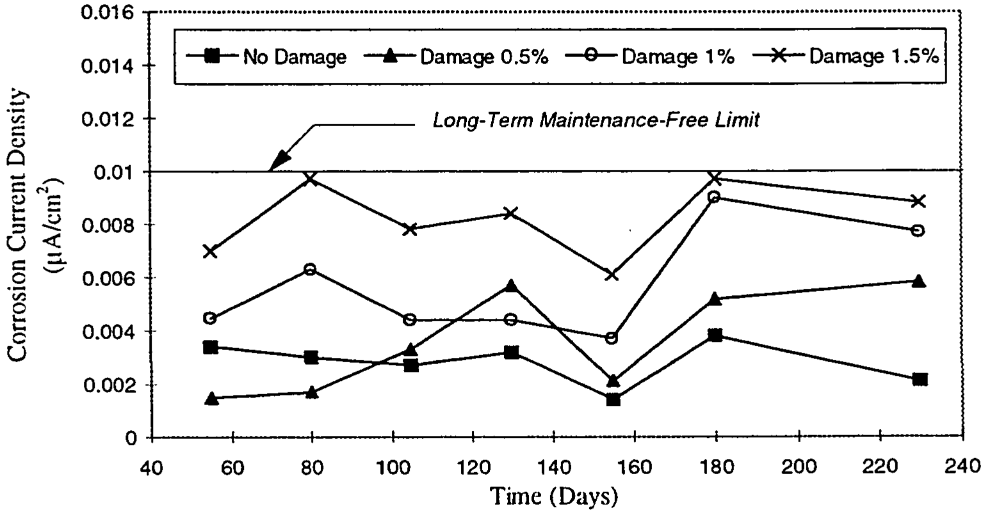

Fig 5.22: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 1% Chlorides

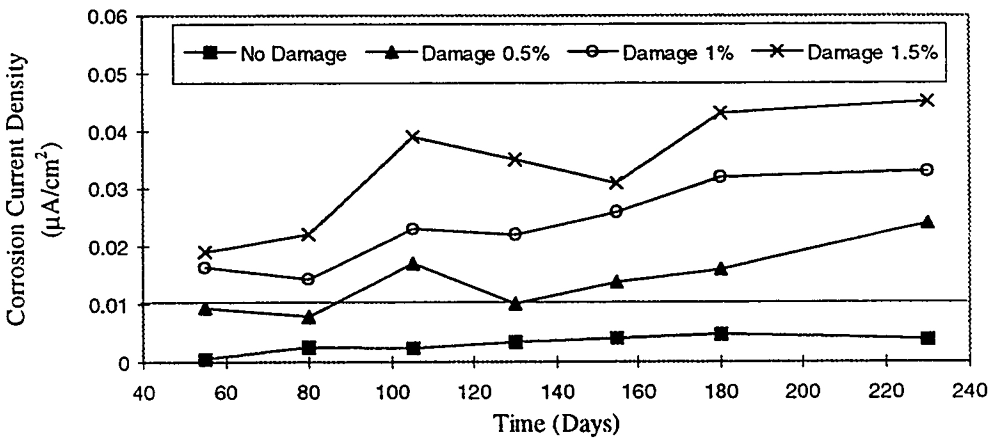

Fig 5.23: Effect of Surface Damage on Reinforcement Corrosion in Specimens Contaminated with 2% Chlorides

chloride level. Rasheeduzaffar et. al. [59], reported that epoxy-coated bars performed exceedingly well as corrosion resistant steel in specimens contaminated with 0.6 and 1.2% of chlorides. However, a significant corrosion was noted in specimens contaminated with 4.8% chlorides. Though their study did not include FBEC bars with damage, the results indicated that the epoxy-coatings have a finite tolerance limit for chlorides. Erodogdu and Bremner [54] indicated that the Icorr on specimens with 1% surface damage bars exposed to a natural marine environment for two years were slightly higher than the 0.01 μA/cm^2.

Fig. 5.24 summarizes the data on Icorr obtained in this study after 230 days of testing. These data represent Icorr on FBEC steel bars with the different levels of surface damage (0, 0.5, 1 and 1.5%) and three contamination levels of 0.4, 1 and 2%.

5.3.3 Effect of Holidays on Reinforcement Corrosion

The data in Tables 5.11 through 5.13 and Figs. 5.25 through 5.28 present the Icorr values on FBEC bars with and without holidays for the different levels of chlorides (0.4, 1 and 2% chlorides by weight of cement). All FBEC steel bar specimens were in passive state, with insignificant corrosion rates. In terms of the maximum corrosion current density normally considered for long-term maintenance-free performance, the only specimens which exceeded that limit, after 180 days of exposure, were those with 3 holidays and contaminated with 2% chlorides.

A maximum of two holidays per linear foot is specified by ASTM A 775 [60]. The above results indicated that all the FBEC steel bars were in a passive state.

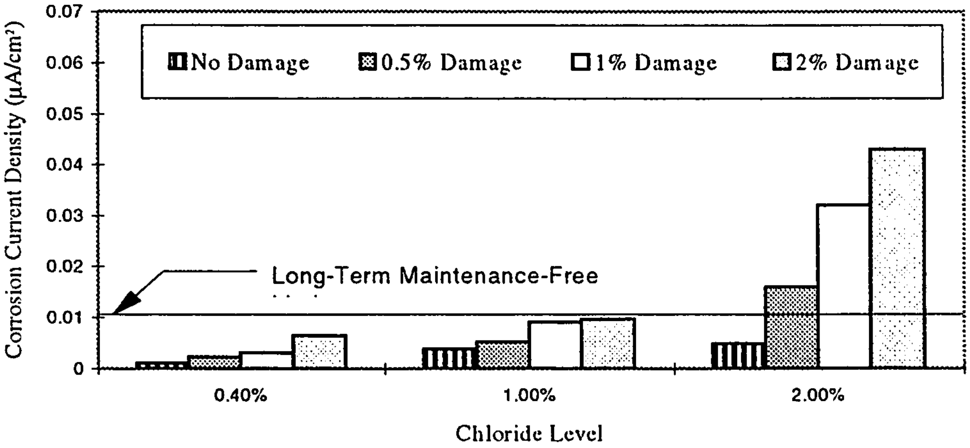

Fig 5.24: Effect of Surface Damage and Chloride Contamination on Reinforcement Corrosion

Table 5.11: Effect of Holidays on Reinforcement Corrosion in Specimens Contaminated with 0.4% Chlorides

Time (Days)	Corrosion Current Density ($\mu A/cm^2$)			
	No Holidays	One Holiday	Two Holidays	Three Holidays
55	6.6 E-04	1.0 E-03	0.8 E-03	1.8 E-03
80	8.0 E-04	0.8 E-03	1.2 E-03	1.4 E-03
105	8.3 E-04	1.0 E-03	0.8 E-03	2.1 E-03
130	8.7 E-04	1.7 E-03	1.2 E-03	1.6 E-03
155	10.0 E-04	1.1 E-03	1.2 E-03	1.9 E-03
180	10.0 E-04	1.2 E-03	1.8 E-03	2.3 E-03
230	11.0 E-04	1.2 E-03	1.5 E-03	3.1 E-03

Table 5.12: Effect of Holidays on Reinforcement Corrosion in Specimens Contaminated with 1% Chlorides

Time (Days)	Corrosion Current Density ($\mu A/cm^2$)			
	No Holiday	One Holiday	Two Holidays	Three Holidays
55	3.4 E-03	0.5 E-03	0.6 E-03	2.1 E-03
80	3.0 E-03	0.5 E-03	0.5 E-03	2.7 E-03
105	2.7 E-03	0.6 E-03	0.6 E-03	2.8 E-03
130	3.2 E-03	1.2 E-03	1.4 E-03	33 E-03
155	1.4 E-03	1.5 E-03	1.9 E-03	3.8 E-03
180	3.8 E-03	1.9 E-03	4.2 E-03	4.9 E-03
230	2.1 E-03	2.1 E-03	2.2 E-03	4.9 e-03

Table 5.13: Effect of Holidays on Reinforcement Corrosion in Specimens Contaminated with 2% Chlorides

Time (Days)	Corrosion Current Density ($\mu A/cm^2$)			
	No Holidays	One Holiday	Two Holidays	Three Holidays
55	0.5 E-03	7.9 E-03	5.8 E-03	3.8 E-03
80	2.5 E-03	4.9 E-03	0.6 E-03	4.9 E-03
105	2.4 E-03	4.9 E-03	2.0 E-03	3.0 E-03
130	3.4 E-03	5.2 E-03	2.8 E-03	4.7 E-03
155	4.0 E-03	7.7 E-03	3.8 E-03	5.9 E-03
180	4.8 E-03	6.5 E-03	6.2 E-03	26.0 E-03
230	3.9 E-03	5.2 E-03	6.3 E-03	27.0 E-03

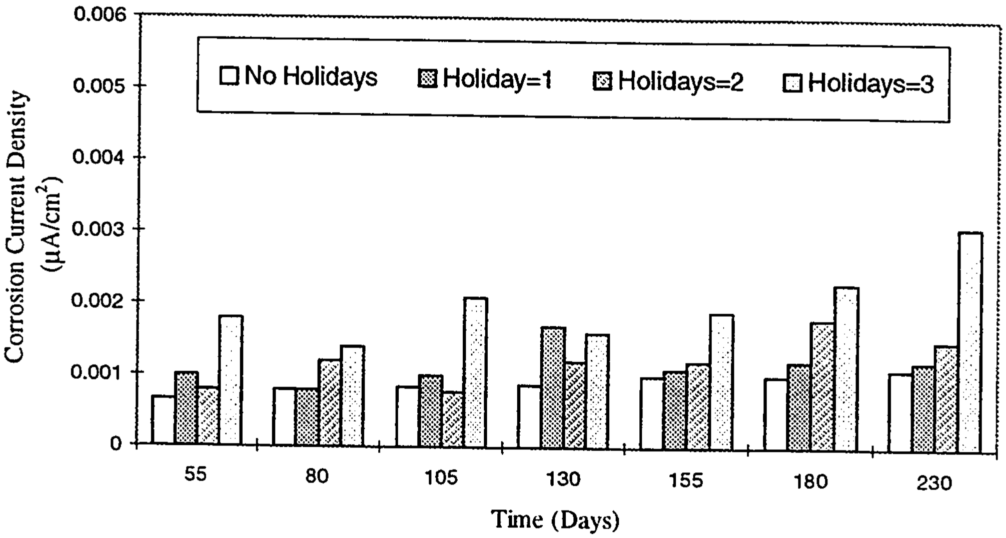

Fig 5.25: Effect of Holidays on Reinforcement Corrosion in Specimens Contaminated with 0.4% Chlorides

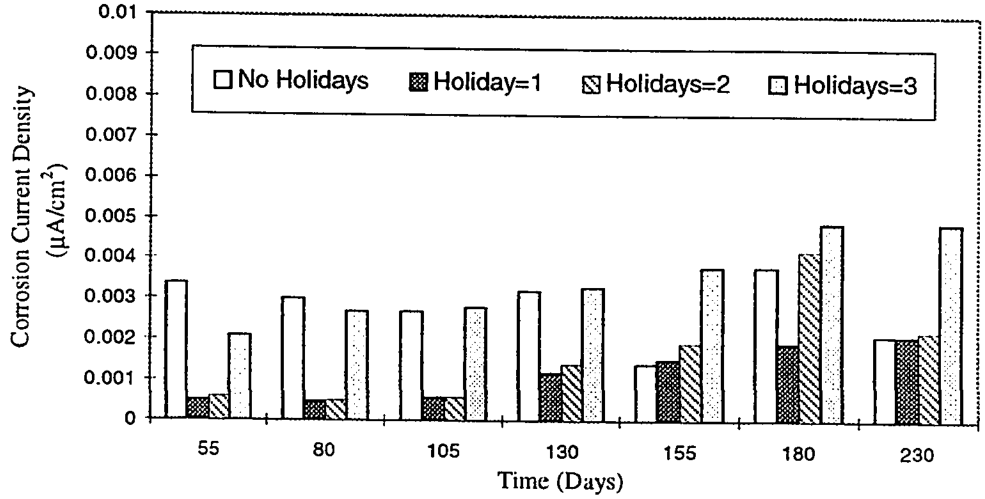

Fig 5.26: Effect of Holidays on Reinforcement Corrossion in Specimens Contaminated with 1% Chlorides

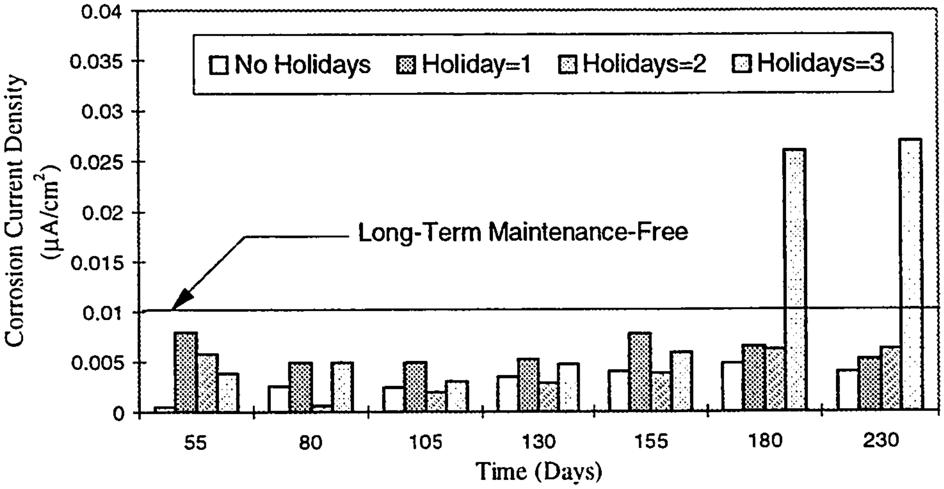

Fig 5.27: Effect of Holidays on Reinforcement Corrossion in Specimens Contaminated with 2% Chlorides

The only specimens which exceeded the long-term maintenance-free limit were the specimens with 3 holidays and contaminated with 2% chlorides.

5.3.4 Effect of Distributed Surface Damage

In addition to the 72 concrete specimens cast to evaluate reinforcement corrosion, three more specimens were cast to address the effect of concentrated and distributed surface damage on reinforcement corrosion. These three specimens were contaminated with 1% chloride by weight of cement and had 1.5% surface damage distributed in two places. Table 5.14 illustrates the corrosion current density on steel in specimens with 1.5% localized and distributed damage. It can be seen that the corrosion current density on steel with localized damage was more than that with a similar degree of damage but distributed at two locations. This may be attributed to the fact that in the former case a higher cathodic area is available compared to the latter. Therefore, ASTM A775 specification for evaluating the allowable surface area per linear foot is a step in the right direction.

Table 5.14: Effect of Localized and Distributed Surface Damage on Corrosion Current Density (1% Chlorides, 1.5% Surface Damage)

Time (Days)	Corrosion Current Density (μA/cm^2)	
	1.5 Surface Damage (Localized)	1.5% Surface Damage (Distributed)
55	7.0 E-03	4.1 E-03
80	97 E-03	5.4 E-03
105	7.8 E-03	4.8 E-03
130	8.4 E-03	4.3 E-03
155	6.1 E-03	3.7 E-03
180	9.7 E-03	4.0 E-03
230	7.8 E-03	3.9 E-03

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

This study was conducted to assess the bond strength and corrosion-resistance characteristics of fusion-bonded epoxy-coated rebars in concrete. The experimental program was designed to evaluate the effect of heat-cool cycling, bar diameter (12 and 20 mm) and coating thickness (150 and 300 μm) on the bond strength. The critical and ultimate bond strength of concrete specimens made with epoxy-coated bars were compared with those made with reference (black) steel bars. Moreover, the effects of holidays and coating damage on the corrosion resistance of FBEC reinforcement were investigated for chloride contamination of 0.4, 1 and 2% by weight of cement. Three levels of surface damage (0.5, 1 and 1.5%) and varying number of holidays (0, 1, 2 and 3 per linear foot) were studied. The corrosion of steel reinforcement was monitored by measuring the corrosion potentials and corrosion current densities at periodic intervals.

6.1 CONCLUSIONS

Based on the results of this study, the following conclusions are made:

1. The epoxy-coated steel bars (150 and 300 μm) have almost the same ultimate bond strength as the uncoated bars, however higher slips are expected for the

coated bars. The application of epoxy-coating results in a reduction in the critical bond strength, particularly for bars with 300 μm coating.

2. A marginal increase in the ultimate and critical bond strength is expected during the earlier heat-cool cycles for. Thereafter, bond strength declines progressively with increasing number of heat-cool cycles.
3. The critical bond in the coated bars is less than that in the uncoated bars and the differential in the critical bond strength between coated and uncoated bars increases with the coating thickness. However, the difference in critical bond strength due to coating becomes less significant with the increasing number of heat-cool cycles.
4. The corrosion potentials indicate an active corrosion of uncoated steel bars. These measurements correlate well with those of corrosion current density.
5. For FBEC bars, the potential measurements are more negative than the uncoated bars, which may be attributed to the high resistance of the coating. This implies the need of development of potential criterion for detection of corrosion activity in FBEC bars.
6. Defect-free FBEC bars indicate passivity in chloride bearing concrete for a prolonged period of time. However, the presence of defects or flaws in the form of surface damage and holidays may lead to earlier development of corrosion activity in a highly chloride contaminated concrete. As the chloride concentration increases over 2%, the FBEC bars with 3 holidays or with 0.5, 1 or 2% surface damage are likely to encounter corrosion much earlier than the damage-free FBEC bars.

6.3 RECOMMENDATIONS

1. The data developed in this study indicated that the chloride level plays an important role in determining the maximum allowable surface damage and holidays. As such, specifications should define the maximum allowable damage and holidays under conditions of different concrete qualities and levels of chloride contamination.

2. There is a need for an appropriate interpretation of half-cell measurements when the epoxy-coated rebars are used. Further investigation is needed to interpret such measurements.

3. As the concrete quality controls the performance of FBEC bars, the reinforcement corrosion in concrete of varying quality needs also to be evaluated.

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VITA

Wael Mohamed Ali Elleithy was born in Egypt, in 1967. He finished his high school in 1985. He joined Faculty off Engineering, Alexandria University in October 1985 and completed his B.Sc. in Civil Engineering in 1990.

He joined King Fahd University of Petroleum and Minerals as a research assistant in the Department of Civil Engineering in 1993, where he obtained his M.Sc. in the field of Structures.

He hopes that this work of his thesis be of some benefit in the field of durability of reinforced concrete structures.