CORROSION DAMAGE OF REINFORCEMENT EMBEDDED IN REINFORCED CONCRETE SLAB

Zhicheng Gao

Dissertation

Approved: _________________________________
Advisor
Dr. Robert Liang

Accepted: _________________________________
Department Chair
Dr. Wieslaw Binienda

Co-Advisor
Dr. Anil K. Patnaik

Interim Dean of College
Dr. Eric Amis

Committee Member
Dr. Zhe Luo

Dean of the Graduate School
Dr. Chand Midha

Committee Member
Dr. Yalin Dong

Date

Committee Member
Dr. En Cheng
ABSTRACT

Corrosion of reinforcements embedded in concrete is a worldwide problem that affects numerous reinforced concrete (RC) structures. While corrosion has always been problematic since the beginning of mining and refinery of metals, corrosion in RC structures only gained research attention during the 1960s and 1970s, following widespread use of de-icing salts on highways in United States. Since then, research has been undertaken worldwide to address corrosion issues. In this dissertation, an experimental study was conducted to characterize the structural behavior of reinforced concrete slabs subjected to accelerated corrosion in the lab.

In order to make the experimental condition similar to the real service environment, the test specimens were introduced with pre-existing cracks and sustained loading was applied during the corrosion process. Accelerated corrosion of tensile steel reinforcements in RC slabs was facilitated by an accelerated corrosion process. Three different test conditions were induced in the corrosion test program: specimens without pre-existing cracks and sustained loading, specimens with pre-existing cracks but no sustained loading, and specimens with pre-existing cracks and sustained loading. In addition, different wetting and drying cycles were incorporated in the corrosion process. Expansion of longitudinal cracks along the tensile reinforcements and transverse cracks crossing the tensile reinforcements were recorded during the corrosion testing. Multiple desired corrosion levels—from low level (1%) to high level (20%)- were applied to different specimens. The
gravimetric metal loss along the longitudinal direction of reinforcements was measured after the bending test. An empirical relationship was developed based on the representative specimens corroded with pre-existing cracks and sustained loading conditions for all desired corrosion levels.

The epoxy-coated reinforcements and polypropylene (PP) fibers were used during casting experimental specimens to assess their corrosion resistance properties. By using the constant electric current, 10 and 20% desired corrosion levels were applied to most specimens, and 40% desired corrosion level was also applied to several test specimens with PP fibers additives to simulate the severe corrosion condition. The surface defects of epoxy-coated reinforcements and two different quantity ratios of PP fibers - $4.5 \text{ kg/m}^3$ and $6 \text{ kg/m}^3$ were considered in this study.

The ultimate capacity of corroded specimens was tested after corrosion process. The average metal loss, the reduced yield strength of corroded reinforced bars and the effective cross section of the specimens can accurately predict the theoretical ultimate capacity loss compared with testing results. The critical inner expansive pressure from XFEM was applied to the developed numerical model to predict the cracking time of the cover concrete, which is defined as the severability limitation of the corroded RC structures. The proposed prediction model had been validated by comparing with the existing experiment data of uniform corrosion condition and the results show that the accuracy of developed model was acceptable to predict the serviceability of reinforced structures with corrosion damage. The effect of non-uniform corrosion condition on cracking pressure from XFEM and cracking time using developed prediction model was discussed.
ACKNOWLEDGEMENTS

I would like to thank my academic advisor, Prof. Robert Liang and Prof. Anil Patnaik, for their support and kind encouragements in the past four years. Indeed, there are many people at The University of Akron that provides a lot of support when needed. Among them, I would like to thank Dr. Zhe Luo, Dr. Yalin Dong, and Dr. En Chen for their help in the PhD program. The research is funding by National Corrosion Center (NCERCAMP) at The University of Akron and the DoD Technical Corrosion Collaboration (TCC), U.S. DoD Office of Corrosion Policy and Oversight. The research is administered by the U.S. Construction Engineering Research Laboratory under agreement number FY12 W9132T-11-C-0035.
TABLE OF CONTENTS

LIST OF TABLES ...................................................................................................................... x

LIST OF FIGURES .................................................................................................................. xii

CHAPTER

I INTRODUCTION .................................................................................................................... 1

1.1 Overview ............................................................................................................................ 1

1.2 Statement of the Problem ................................................................................................... 4

1.3 Objectives and Scope of Work .......................................................................................... 7

1.4 Dissertation Outline .......................................................................................................... 14

II LITERATURE REVIEW ..................................................................................................... 16

2.1 State-of-the-Art Review for corrosion damage of reinforced concrete structures .......... 16

2.2 Accelerate corrosion test without sustained loading condition ........................................ 18

2.3 Accelerated corrosion test with sustained loading conditions [84] ................................. 19

2.3.1 Yoon et al. [62] ........................................................................................................... 20

2.3.2 Ballim et al. [61] ....................................................................................................... 23

2.3.3 El Maaddawy et al. [54] ........................................................................................ 25

2.3.4 Vidal et al. [43] ........................................................................................................ 28

2.3.5 Malumbela et al. [55, 63] ....................................................................................... 29

2.4 Epoxy-coated reinforcements an PP fibers ........................................................................ 31

2.5 Theoretical prediction methodology of ultimate capacity loss of corroded RC specimens ................................................................. 32
2.6 Prediction model for cracking of cover concrete (serviceability) ...................33

III EFFECTS OF SUSTAINED LOADING AND PRE-EXISTING CRACKS ON CORROSION BEHAVIOR OF REINFORCED CONCRETE SLABS .............35

3.1 Introduction ........................................................................................................35

3.2 Motivation of experiment work related to corrosion of slab specimens ..........37

3.3 Test setup ...........................................................................................................38

3.4 Specimen groups and their designation ..........................................................40

3.5 Procedure for introducing pre-cracked condition .........................................41

3.6 Accelerated corrosion procedure .....................................................................44

3.6.1 Theoretical determination of electrical current ........................................44

3.6.2 Corrosion test setup ....................................................................................45

3.7 Test results and discussion .............................................................................46

3.7.1 Capacity and metal loss after accelerated corrosion .....................................46

3.7.2 Observation of Corrosion cracks ...............................................................51

3.8 Conclusions .......................................................................................................56

IV EXPERIMENTAL STUDY OF RELATION BETWEEN CORROSION CRACK WIDTH AND METAL LOSS FOR REINFORCED CONCRETE SLABS .......58

4.1 Introduction .......................................................................................................58

4.2 Background .......................................................................................................60

4.3 Experimental program .....................................................................................62

4.3.1 Reinforcement configuration and material properties ................................62

4.3.2 Test program ................................................................................................63

4.4 Procedure for introducing cracks prior to accelerated corrosion ....................63

4.5 Accelerated corrosion procedure .....................................................................64

4.5.1 Theoretical determination of electrical current ........................................64

4.5.2 Corrosion test setup for different groups ....................................................65
6.4.2 Ultimate capacity prediction method of corroded specimens...............108
6.4.3 Theoretical Validation.................................................................113
6.5 Conclusion ......................................................................................117

VII PREDICTING THE TIME FOR THROUGH-CRACKING IN CONCRETE COVER
CONSIDERIG UNIFORM AND NON-UNIFORM CORROSION
CONDITIONS..............................................................................................119
7.1 Introduction.....................................................................................119

7.2 Review of previous predictive models.............................................121
  7.2.1 Predictive model for critical expansion pressure..........................121
  7.2.2 Predictive model for time to through-cracking due to rebar corrosion..........122
7.3 Critical expansion pressure calculate from extend finite element method ......123
  7.3.1 Extend finite element method.......................................................123
  7.3.2 Critical expansion pressure results..............................................126
7.4 New prediction model.......................................................................137
  7.4.1 Modification based on existing model........................................137
  7.4.2 Model Validation and Discussion..............................................138
7.5 Conclusion ......................................................................................141

VIII SUMMARY AND CONCLUSIONS .......................................................143
  8.1 Summary of Work Accomplished..................................................143
  8.2 Conclusions...................................................................................145
  8.3 Recommendations for Future Research ........................................148
REFERENCES ..............................................................................................150
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-1</td>
<td>39</td>
</tr>
<tr>
<td>3-2</td>
<td>42</td>
</tr>
<tr>
<td>3-3</td>
<td>44</td>
</tr>
<tr>
<td>3-4</td>
<td>45</td>
</tr>
<tr>
<td>3-5</td>
<td>48</td>
</tr>
<tr>
<td>4-1</td>
<td>64</td>
</tr>
<tr>
<td>4-2</td>
<td>65</td>
</tr>
<tr>
<td>4-3</td>
<td>70</td>
</tr>
<tr>
<td>4-4</td>
<td>76</td>
</tr>
<tr>
<td>4-5</td>
<td>77</td>
</tr>
<tr>
<td>5-1</td>
<td>81</td>
</tr>
<tr>
<td>5-2</td>
<td>81</td>
</tr>
<tr>
<td>5-3</td>
<td>83</td>
</tr>
<tr>
<td>5-4</td>
<td>84</td>
</tr>
<tr>
<td>5-5</td>
<td>87</td>
</tr>
<tr>
<td>5-6</td>
<td>93</td>
</tr>
<tr>
<td>6-1</td>
<td>101</td>
</tr>
<tr>
<td>6-2</td>
<td>108</td>
</tr>
<tr>
<td>6-3</td>
<td>116</td>
</tr>
</tbody>
</table>
6-4 Validation results of specimens with PP fibers .......................................................... 117

7-1 Basic experimental parameters .................................................................................. 131

7-2 Critical expansive pressures comparison between existing models and XFEM results .......................................................................................................................... 132

7-3 Cracking pressures of XFEM for uniform and non-uniform corrosion conditions.. 137

7-4 Comparison between predicted and experimental results with uniform corrosion condition ......................................................................................................................... 140

7-5 Comparison of predicted cracking time between uniform and non-uniform corrosion conditions .......................................................................................................................... 141
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1–1</td>
<td>Corrosion damage observed from in-service structures (<a href="http://www.google.com">www.google.com</a>)</td>
<td>2</td>
</tr>
<tr>
<td>1–2</td>
<td>Proposed durability model of corroding RC structures [6]</td>
<td>3</td>
</tr>
<tr>
<td>1–3</td>
<td>Flowchart depicting the scope of work</td>
<td>13</td>
</tr>
<tr>
<td>1–4</td>
<td>Flowchart depicting the scope of work on epoxy-coated and PP fibers</td>
<td>14</td>
</tr>
<tr>
<td>2–1</td>
<td>The differences between the beam and slab section reinforcement arrangement</td>
<td>18</td>
</tr>
<tr>
<td>2–2</td>
<td>Loading frame used by Yoon et al. [62]</td>
<td>21</td>
</tr>
<tr>
<td>2–3</td>
<td>Loading frame used by Ballim et al. [61]</td>
<td>24</td>
</tr>
<tr>
<td>2–4</td>
<td>Loading frame used by El Maaddawy et al. [54]</td>
<td>27</td>
</tr>
<tr>
<td>2–5</td>
<td>Loading frame used by Vidal et al. [43]</td>
<td>29</td>
</tr>
<tr>
<td>2–6</td>
<td>Loading used by Malumbela et al. [55, 63]</td>
<td>30</td>
</tr>
<tr>
<td>3–1</td>
<td>The differences between the beam and slab section reinforcement arrangement</td>
<td>38</td>
</tr>
<tr>
<td>3–2</td>
<td>Reinforcement configuration within test slabs (mm)</td>
<td>39</td>
</tr>
<tr>
<td>3–3</td>
<td>Flexural test for un-corroded slab specimen</td>
<td>39</td>
</tr>
<tr>
<td>3–4</td>
<td>Appearance of first crack during the stage of generating pre-existing cracks</td>
<td>43</td>
</tr>
<tr>
<td>3–5</td>
<td>Accelerated corrosion test setup, the dimensions are in mm</td>
<td>46</td>
</tr>
<tr>
<td>3–6</td>
<td>Accelerated corrosion test setup: (a) without pre-cracked and sustained loading; (b) with pre-cracked and sustained loading</td>
<td>46</td>
</tr>
<tr>
<td>3–7</td>
<td>Crack pattern and load-deflection behavior for specimens without corrosion and with corrosion</td>
<td>47</td>
</tr>
<tr>
<td>3–8</td>
<td>Steel coupons of corroded bars and division of the surface of the specimen</td>
<td>49</td>
</tr>
<tr>
<td>3–9</td>
<td>Variation of mass loss of steel along the specimens within the tank</td>
<td>51</td>
</tr>
</tbody>
</table>
7–3 FE simulation of the cylinder model ............................................................... 128
7–4 Uniform corrosion pattern condition ............................................................ 133
7–5 Basic description for uniform and non-uniform corrosion patterns .............. 135
7–6 Non-uniform corrosion pattern ..................................................................... 136
CHAPTER I  
INTRODUCTION  

1.1 Overview  

Corrosion of reinforcements embedded in concrete is a serious problem that affects deterioration of reinforced concrete (RC) structures [1, 2]. Whilst corrosion has always been problematic since the beginning of mining and refinery of metals, it only gained research relate to RC structures during the 1960s and 1970s following de-icing salts used in the US highways and a construction boom in the Arabian Gulf [3]. The typical corrosion damage detected from in-service structures is shown in Figure 1–1. Many articles continue to be published worldwide addressing corrosion issues. Theoretical models have also been developed and calibrated with experimental results to predict the behavior of RC structures with corroding reinforcements as well as their service lives. 

Based on NACE corrosion cost study [4], there are 607,380 bridges in the United States (2013). Of this total, 200,000 bridges are steel, 235,000 are conventional reinforced concrete, 108,000 bridges are constructed using pre-stressed concrete, and the balance is made using other materials of construction. Approximately 30 percent of the bridges are structurally deficient or functionally obsolete. The annual direct cost of corrosion for highway bridges is estimated to be $13.6 billion.
For those responsible for monitoring the service life of corrosion affected RC structures such as structural engineers and asset managers, it is important to identify the easy-to-measure damages that can be directly related to the loss in the section of reinforcements and eventually, the loading carry capacity of the RC structures [5, 6]. In order to relate these easy-to-measure damages to the capacity of the RC structures, there are two general methods can be used: (i) monitor the real structures serviced in corrosive environment with nature corrosion rate and (ii) accelerated corrosion test in laboratory to develop empirical relationship.

Figure 1–2 represents the service life model for a reinforced concrete structure exposed to corrosive environment [6]. This model presents the structural ultimate capacity as a function of its service time. $T_1$ is the corrosion initiation stage from the time of construction. $T_2$ is the corrosion propagation stage during which the steel starts to corroded until an unacceptable level is defined when crack appearing on the concrete surface is less
than the limitation of the standard (0.18 mm in ACI-224R-01 [7]). The third stage ($T_{RL}$) starts from the time of external visual degradation until a degradation level is such that the residual ultimate capacity less than the design ultimate capacity based on design code.

![Proposed durability model of corroding RC structures](image)

Figure 1–2 Proposed durability model of corroding RC structures [6]

Cracking of cover concrete caused by corrosion of steel reinforcement can result in shorter service life of structural component. First, cracking of concrete, attributable to volume increase of corrosion products (i.e., rust), indicates the loss of cross-sectional steel area of reinforcements. Second, corrosion-induced cracking would induce bond strength deterioration at the steel-concrete interface, as the cracking of cover concrete indicating the structural deterioration process. As a result, cracking of concrete cover is widely treated as a warning sign of loss of serviceability and the need for taking the required maintenance actions, such as corrosion prevention, crack repair, or component replacement, in routine inspection and monitoring of structures exposed to corrosive environments. Predicting the cracking time of cover concrete induced by corrosion of steel reinforcements can help engineers select an appropriate time to take maintenance actions, thereby minimizing or avoiding further deterioration and potential economic loss.
1.2 Statement of the Problem

De-icing salt is one of the most serious chloride sources to cause corrosion damage of the reinforcements embedded in concrete. Continuous freeze and thaw actions coupled with chloride ingress from de-icing salts used during winter season deteriorate bridge decks and other components. Especially, for continuous span structural slab bridges, there is large tensile area at the top surface near the support locations which is directly exposed to the de-icing salts and remains in tension under service loads (dead load and live loads). RC bridges in the states using de-icing salts only in winter times (like in Ohio) are exposed to severe corrosive environment. This is somewhat different from RC bridges located in marine environment. For the structures subjected to de-icing slats only during certain time of the year, the structure is exposed to corrosive environment for certain length of time in the year and almost without corrosive environment in other time. Even within winter time, the de-icing is not applied every day. During the season when the de-icing salts are applied, the structure already has cracks caused by service load and chlorides can penetrate the concrete through these pre-existing cracks. Therefore, the pre-cracked initial condition and wet–dry period cycles are important conditions to be considered in setting up experiments with accelerated corrosion process to simulate realistic conditions.

Cracks in reinforced concrete elements are a common occurrence due to the low tensile strength of concrete. Cracking that occurs in reinforced concrete structures at service load (i.e., during the normal use of the structures) cannot generally be avoided. Sustained service loads on structures, surface cracks, or a combination of both conditions can affect the rate of corrosion, the residual moment strength of corroded RC members, and the effectiveness of repairs. Experiments conducted and theoretical models developed
without consideration of pre-existing cracks and sustained loading conditions do not capture the true behavior of in-service RC structural elements that are subjected to corrosion.

The usage of epoxy-coated reinforcements is thought of as one of the most effective methods for protecting reinforced concrete structures from corrosion damages. Such protection primarily depends on the integrity of the coatings. However, the coating surface may sustain damages during the handling of the epoxy-coated reinforcements. All the previous studies did not consider surface defects condition of epoxy-coated reinforcements before embedded into concrete. The effectiveness of epoxy-coated reinforcement with surface defects for corrosion prevention has not been fully investigated in the past. Expect using epoxy-coating reinforcements, fibers adding into plain concrete is another way to help the RC structures against the corrosion damage. PP fibers is a common fibers used in many research. The corrosion resistance effectiveness of PP fibers adding into reinforced concrete structures serviced in chloride corrosive environment has not been fully investigated in the past.

Previous models used to predict residual ultimate capacity are mostly based on either the average gravimetric mass loss or the maximum pit metal loss of corroded reinforced bars [6, 8, 9]. So far, no research considered the reduction yield strength of the corroded reinforced bars to predict the ultimate capacity loss of corroded RC structures. However, a lot of researchers [10-20] did studies on mechanical properties of corroded reinforced bars. The general conclusion was that the corroded reinforcing steels exhibited less ductility compared with un-corroded ones and the ultimate strain was significantly decreases with the corrosion section loss [10, 11]. The corrosion deterioration of reinforced bars resulting
decrease in the yield strength and the loss of cross section of reinforced bars needs to be evaluated to assess the ultimate capacity loss of the RC member and formulate repair strategies.

The corrosion experiment is time consuming and often difficult to simulate the actual corrosion environment in the field. The theoretical models are constraint by their inability to model the realistic material properties of concrete. Furthermore, it appears that most theoretical models have been developed based on the assumption of uniform corrosion pattern, i.e. corrosion product around the perimeter of rebar is uniform [2, 21-30]. Although the assumption of uniform corrosion in the past studies has contributed to advancement on research and practice of corrosion in RC structures; nevertheless, it has been recently observed that non-uniform corrosion pattern could exist in the real-world built RC structures exposed to field corrosion environment. Non-uniform distribution of expansion pressure may cause adverse effects to the cracking behavior of concrete cover due to the fact that higher pressure is concentrated at the outer part of rebar toward concrete cover. This may cause higher tensile stress development in concrete cover, leading to fast occurrence of cracks in concrete cover and resulting in reduction of time-to-cracking and eventually shorten service life of concrete structures.

Finite element (FE) method has been used as an effective tool to study the effect of steel reinforcement corrosion in concrete [31-39]. The past FEM study, in general, does not attempt to compare the numerical simulation results with test observations [32, 37, 38]. Some comparison between FEM modeling and test results was just qualitative [39], or the quantitative comparison showing poor agreement with the test results [31, 34]. Different researchers had different explanations for the poor agreement between FEM analysis and
test observations. Williamson and Clark [40] offered some insights on the poor agreement of FEM in modeling corrosion process of RC structures by pointing out the importance of considering the fracture energy of the concrete. While Val et. al. [34] used different testing methods of tensile strength of concrete to explain the poor agreement results. Usually, the previous FE models [34-36] can only adopted the stress distribution and path to represent the corrosion crack path of RC structure.

1.3 Objectives and Scope of Work

1) The objective of this study is to highlight that two important factors - (i) pre-existing cracks and (ii) sustained loading - that need to be considered in accelerated corrosion tests and to determine the influence of these two factors on the mass loss of the reinforcements as well as the crack expansion rate and the patterns of cracks on the surface of the concrete specimens. To address these factors, concrete slab specimens having reinforcements of the same size that were prepared using a single concrete mix were loaded to introduce cracks prior to the application of sustained loading and external constant current to accelerate the corrosion process. The experimental results were used to compare the theoretical calculations of metal loss and the actual measured metal loss. The ultimate load carrying capacity of the corroded specimens with various experiment conditions (different wetting and drying cycles and sustained loading levels) were measured and compared.

2) Fill aforementioned gap by providing relevant experimental data and examination the corrosion-induced cracks in RC slabs. The different crack patterns and the interaction mechanism of transverse and longitudinal cracks observed in this experimental program were discussed in detail. The relationship between corrosion-
induced crack width and the corresponding metal loss was also developed from experimental data. The observation and empirical relationship between corrosion crack width and metal loss can be useful for field inspection of bridge deck condition considering corrosion-induced damage.

3) Present findings of accelerated corrosion tests on RC slabs without any additional materials and with epoxy-coated reinforcements and PP fibers as additive materials. In particular, the effect of surface defects of epoxy-coated reinforcements on corrosion resistance properties, when compared with specimens casted with black reinforcements, is one of main interests. The corrosion mitigation effect on corrosion crack width expansion and metal loss of PP fibers added into concrete, when compared with test specimens casted without PP fibers, is another main finding in this program.

4) Conduct the extended experimental work on RC slabs that were subjected to different experimental conditions with different corrosion desired levels. Except the regular test RC specimens with black reinforced bars and plain concrete, there were also several test specimens casted with epoxy-coated reinforcements or adding PP fibers into plain concrete to assess the effect of the additive materials on the ultimate capacity loss. The corrosion crack patterns detected from different specimens were assessed and related with the effective cross section of the corroded specimens. This study tried to use the net area of corroded reinforced bars with decreased yield strength of corroded reinforced bars to predict the ultimate capacity loss of corroded specimens. The proposed ultimate capacity prediction model can be used to find the ultimate capacity loss of testing slabs suffered corrosion damage.
Developed a numerical model combined with XFEM to predict the cracking time of cover concrete. In this study, the XFEM model was validated using existing experiment data in the literature for uniform corrosion pattern. From which the XFEM model was used to study the effect of non-uniform corrosion pattern on time to through-crack in concrete cover. A new predictive equation was suggested to compute time to through-crack for uniform and non-uniform corrosion pattern.

The goal of this research is to develop accelerated corrosion experimental program and numerical model used to assess the ultimate and serviceability limitation of RC structures. The specific scope of work is enumerated as follows:

1) Accelerated corrosion test for RC slabs was used to assess the pre-existing cracks and sustained loading experimental condition. Different experimental conditions were identified as: specimens without pre-existing cracks and sustained loading, specimens with pre-existing cracks but no sustained loading, and specimens with pre-existing cracks and sustained loading. In addition, different wetting and drying cycles were incorporated in the corrosion process. Expansion of longitudinal cracks along the main reinforcements and transverse cracks crossing the main reinforcements was recorded during the corrosion testing. The final mass loss and the ultimate load carrying capacity after corrosion were also determined. The results highlighted that pre-existing cracks initial condition and sustained loading during the corrosion process had a noticeable influence on the experimental results and caused a more severe reduction in ultimate load carrying capacity of the specimens. The transverse cracks allowed chloride ions to reach the surface of the reinforcements to induce corrosion damage, which induced higher longitudinal corrosion cracking at the initial
corrosion stage. The transverse cracks also provided more free space and paths for the corrosion products to fill and come out, which released the hoop stress induced by the corrosion products and reduced the longitudinal corrosion cracking at the final stage of the corrosion test.

2) In experimental program, multiple desired corrosion levels-from low level (1%) to high level (20%)-based on Faraday’s Law were applied to different specimens. The wetting–drying cycles during the accelerated corrosion duration were also adopted in this study. The corrosion cracks width growth rates within exposure area in both longitudinal and transverse directions were monitored during the dry period. The experimental results indicated the corrosion cracks width growth rate in the longitudinal direction was related with the overall crack patterns. The gravimetric metal loss along the direction of reinforcements was measured by removing the concrete. An empirical relationship that 1% metal loss induced 0.03 mm longitudinal crack width was developed based on the representative specimens corroded with pre-existing cracks and sustained loading conditions for all desired corrosion levels.

3) In particular, the effect of surface defects of epoxy-coated reinforcements on corrosion resistance properties, when compared with specimens casted with black reinforcements, is one of main interests. The corrosion mitigation effect on corrosion crack width expansion and metal loss of PP fibers added into concrete, when compared with test specimens casted without PP fibers, is another main finding in this program. In this study, accelerated corrosion tests were carried out by means of passing the electrical to the specimens and exposing limited area of specimens to slat
solutions. The electrical current values used to achieve 10%, 20% and 40% desired corrosion levels were based on Faraday’s law.

4) Slabs were tested whilst under a sustain loading, which was set as 60% of the uncorroded specimen’s ultimate capacity. 1%, 5%, 10%, 20% and 40% corrosion damage levels were used in this experiment. The epoxy-coated reinforcement with surface defects and polypropylene (PP) fibers concrete were also used in this study to assess their effects on ultimate capacity loss. The results indicated: i) the corrosion-induced crack propagation patterns had influence on the effective specimens cross section used to calculate the ultimate capacity; ii) the average metal loss and the reduced yield strength of corroded reinforced bars can accurately predict the ultimate capacity loss compared with testing results; iii) the specimens of epoxy-coated reinforcement with surface defects have similar ultimate capacity loss as the specimens with black reinforcement; iv) PP fibers can help to mitigate the corrosion crack expansion and the ultimate capacity loss.

5) The XFEM was used to simulate the cracking process of concrete cover from initial cracking to the completely cracking. The thick-walled cylinder model adopted in XFEM considering the realistic properties of the corrosion-induced cracked concrete such as fracture energy, tensile strength and elastic modulus of concrete. The critical inner expansive pressure from XFEM was applied to the developed numerical model to predict the cracking time of the cover concrete. The proposed prediction model had been validated by comparing with the existing experiment data of uniform corrosion condition and the results show that the accuracy of developed model was accepted to predict the serviceability of reinforced structures.
with corrosion damage. The effect of non-uniform corrosion condition on cracking pressure from XFEM and cracking time using developed prediction model was discussed. Based on selected reference parameters in this study, the cracking pressure and predicted cracking time of non-uniform condition decreases up to about 35% and 26% respectively.
The scope of work to achieve the stated objectives of this research has been shown in Figure 1–3.

Figure 1–3 Flowchart depicting the scope of work
The special scope of work related with Epoxy-coated reinforcements and PP fibers of this research has been shown in Figure 1–4.

![Flowchart](image)

**Figure 1–4 Flowchart depicting the scope of work on epoxy-coated and PP fibers**

1.4 Dissertation Outline

Chapter 2 presents a brief summary of state-of-the-art studies on the corrosion damage of reinforced bars.

Chapter 3 presents the per-existing cracks and sustained loading effect on the corrosion experimental results and shows that a more severe reduction in ultimate load carrying capacity of the specimens is related to these conditions. Different wetting-drying cycles are also considered and discussed.
Chapter 4 provides empirical relationship between metal loss and crack width based on the representative specimens corroded with pre-existing cracks and sustained loading conditions for all desired corrosion levels (1%, 5%, 10% and 20%).

Chapter 5 presents the corrosion resistance properties of the specimens casted by epoxy-coated reinforcements with surface defects and different quantity of PP fibers.

Chapter 6 develops the theoretical prediction methodology to predict the ultimate capacity loss of corroded specimens by considering the reduced yield strength and area loss of corroded reinforced bars.

Chapter 7 presents a time prediction model of corrosion-induced cover concrete through-crack using extent finite element method (XFEM) analysis results.

Chapter 8 provides a summary of work completed as a part of this study, as well as conclusions. Recommendations for future research are presented at the end.
CHAPTER II
LITERATURE REVIEW

2.1 State-of-the-Art Review for corrosion damage of reinforced concrete structures

Steel corrosion causes the most damage in in-service RC structures near the marine environment. However, in laboratory terms, the process of natural steel corrosion is very slow, needing tens of years to cause reasonable structural damage. For example, François & Arliguie (1998), Castel et al (2003), Vidal et al (2007) and Zhang et al (2009a,b; 2010) [41-46] who allowed their laboratory specimens to corrode naturally, had to wait for four years for steel corrosion to start and an additional two years for first cracking to occur. They only obtained reasonable structural damage after 20 years. These times are not often afforded in laboratory tests. Researchers, understandably, have and continue to use various techniques to accelerate steel corrosion so as to shorten the needed testing time. In doing so they anticipate that structural damage under accelerated tests is proportional to damage caused by natural steel corrosion. It should be pointed out that results obtained by researchers on laboratory specimens that are subjected to accelerated corrosion tests are often passed on to structural engineers and asset managers to apply them to real RC structures which corrode in the field. If they are not applicable to those structures then there is likelihood for engineers to authorize repairs of corroding RC structures at dangerous levels of steel corrosion or when load-bearing capacities of structures are still adequate. For the safety of occupants of corroding RC structures, as well as to minimize
costs from unnecessary repairs, there is need to understand well how to apply (if at all applicable) results from accelerated laboratory tests to in-service structures.

The current research objectives can be divided into three main categories: 1) Compare the theoretical steel mass loss (Faraday’s law) with actual gravimetric mass loss measured in experiment [8, 43, 47-51]; 2) Develop the prediction methodology to predict the experiment work about ultimate capacity loss caused by metal loss [6, 8, 49, 52-55]; 3) Propose the analytical and numerical models to predict cracking time of the cover concrete and validate with experimental results [6, 22, 48, 52, 56].

Based on the literature review on the experimental programs, there are three typical method used simulate the corrosive environment: i) mix chloride ion into concrete or spray chloride solution in the chamber [6, 22, 48, 49, 52, 57, 58]; ii) immerse the specimen in tanks with NaCl solution [43, 47, 50, 59-61]; iii) expose the selected faces of concrete elements to chlorides [62, 63] or by selectively spraying them with salt solution [44, 45, 53]. For example, some researchers opted to mix concrete with chlorides ranging from 1% [52] to 5% [58] by weight of cement. Others immersed their cured samples in tanks with NaCl solution with concentration from 3% [50] to 5% [47] by weight of the solution. Levels of concentration of chlorides were often selected to simulate chloride concentration of seawater which has a salt concentration of about 3.5%. Following discussions by Poursae & Hansson (2009) [64], Yuan & Ji (2009) [65], Yuan et al(2007) [66], Jang & Oh (2010) [35] and Malumbela et al(2011) [29], it is recommended that in accelerated corrosion tests: 1) Steel should be allowed to passivate before adding chlorides to concrete. This is equivalent to saying chlorides should be added externally and not be mixed with concrete. 2) Only selected faces of concrete elements should be contaminated with chlorides. Specimens should not be submerged in salt solutions. This point will be further discussed later.
So far, few corrosion-related experimental studies [62, 67] on slab specimens are reported in the literature. Structural slabs do not have shear reinforcement (stirrups) around the main tensile reinforcing bars and therefore behave differently from beams, as shown in Figure 2–1. The failure modes of corroded RC slabs need to be assessed and compared with un-corroded RC slabs.

The usage of epoxy-coated reinforcements is thought of as one of the most effective methods for protecting reinforced concrete structures from corrosion damages [68-74]. Many studies [75-83] have been conducted to characteristics in the mechanical properties of reinforced concrete with fiber additives. Such concrete is used commonly in retrofitting and repairing the covering of concrete structures. According to these studies [75-83], the increase of formability and bending strength are considered as advantages of adding fibers into the concrete.

2.2 Accelerate corrosion test without sustained loading condition

Malumbela et al. [29, 84, 85] performed a critical review and experimental work with beam specimens and found that while in-service, RC structures normally corrode under sustained loading. The majority of articles on corrosion have focused on structures that
corroded in the absence of sustained loading [1, 5, 6, 8, 21, 23, 42, 43, 47, 50, 52, 53, 61, 62, 67, 86-96]. Their continued production due to corrosion of steel bars embedded in concrete applies tensile stresses on the surrounding concrete which leads to cracking of the cover concrete. Steel corrosion therefore reduces the structural integrity of concrete by the loss in the area of steel, cracking of the cover concrete and the loss in the bond between the corroding steel and the surrounding concrete. Some experimental work has been carried out on the relation between the actual rate of corrosion measured as the gravimetric mass loss of corroded steel bars and the theoretical mass loss of corroded steel bars calculated from Faraday's Law. It was generally found that at low levels of corrosion (<5%), Faraday's Law under predicted the gravimetric mass loss whilst at larger levels of corrosion (>10%), it over predicted the gravimetric mass loss. Numerical models [48, 49, 96] and analytical models [24, 91] have also been developed to relate the corrosion crack widths with the level of corrosion. Mangat & Elgarf [52] and Azad et al [8] attributed the poor predictions of ultimate capacity of beams at high mass losses of steel to losses in the bond between corroded steel bars and the surrounding concrete. They therefore developed necessary correction factors.

2.3 Accelerated corrosion test with sustained loading conditions [84]

The behavior of RC members that are corroded under simultaneous load and steel corrosion is expected to be different to the behavior of beams based on sequential corrosion and load application due to the following [84]:

1) For RC members that are under load, tensile steel bars and the surrounding concrete are under tension and conditions of equilibrium of internal forces and compatibility of deformations must be satisfied. If the area of the tensile steel bars is reduced due to
steel corrosion, larger tensile strains are applied on the steel and compatibility requires an increase in the curvature and hence a reduction in the stiffness. At the early corrosion stages, a RC member corroded under a sustained load therefore has two contrasting mechanisms that control its structural response namely; a decrease in the stiffness due to a loss in the area of steel and an increase in the stiffness due to a gain in the bond between the corroding steel and the surrounding concrete.

2) Flexural cracks due to the applied load provide clear channels of discharge of corrosion products and hence may limit the confinement of concrete around the corroding bar.

2.3.1 Yoon et al. [62]

This work involved testing RC specimens of dimensions $100 \times 150 \times 1170$ mm. The beams were tested whilst under a sustained load using a mechanical loading frame shown in Figure 2–2, in the absence of a sustained load but having been previously loaded and in the absence of a sustained load with no previous loading. Loading on the beams tested under a sustained load was induced by hanging weights on a lever beam and the load was then transferred to the test beams by a load distribution beam. A lever arm was employed to amplify the load from the applied weights. The frame applied tensile stresses at the top part of the beams so that they were tested with the tensile face up. The arrangement to test inverted beams was mainly done to enable a plastic dam of NaCl solution to be built at the top of the beams. This arrangement also exposed the tensile face of the beams where the severest level of corrosion was expected and repair of damaged concrete would often be carried out. Even though it was not reported in the paper, the testing procedure used made it clearly easy to monitor various other properties of corroding RC structures such as the propagation of corrosion cracks during the corrosion process.
Furthermore, the loading frame was a mechanism which relied on the test specimens for its stability and hence it followed the beams as they deflected. This allowed the system to always apply a constant load on the beams.

![Figure 2–2 Loading frame used by Yoon et al. [62]](image)

The overall configuration of the loading system was that specimens were tested over a span of 1050 mm using a 4-point bending configuration with a constant moment region of 230 mm at the middle part of the beams. They were tested under sustained load levels equivalent to 0%, 20%, 45%, 60% and 75% of the ultimate capacity of a virgin beam. During the corrosion initiation stage, beams were subjected to cycles of four days wetting with 5% solution of NaCl and three days drying in natural air. The results indicated that beams that were previously loaded to high loads had shorter corrosion initiation periods (2 days and 4 days for beams previously loaded to 75% and 45% loads respectively). The results also showed that beams with high sustained loads had shorter corrosion initiation periods (3 days, 10 days and no corrosion initiation after 30 days for beams tested under 75%, 45% and 0% loads respectively). Since the drying and wetting cycles with NaCl solution used in the program does not overly accelerate the corrosion process and yet the
service life of in-service structures is in the order of tens of years, the differences in the
periods of corrosion initiation found by [62] are insignificant.

Corrosion current passing through the corroding steel bars was monitored during the
corrosion propagation stage and converted to the rate of corrosion using Faraday's Law.
The results showed that beams subjected to high levels of sustained loads and high previous
loads had higher corrosion rates. These rates were found to generally increase with the
duration of electrolysis and at an increasing rate. Mass loss of steel was also found to be
higher for beams tested under a sustained load compared to beams that were previously
loaded and then corroded in the absence of a sustained load. This was attributed to larger
crack widths in specimens that were corroded under a sustained load.

Clearly this work indicates that the behavior of beams corroded under load is different from
the behavior of beams corroded in the absence of a sustained load. There are however a
number of key areas that the work did not fully address:

1) Even though deflections were measured, it was difficult to explain why they suddenly
   increased with accelerated corrosion. To fully characterize structural performance, it
   was necessary to also monitor variations of strains across the beams.

2) It is well documented that Faraday's Law does not accurately predict mass loss of steel
   bars corroded whilst embedded in concrete. It was as such necessary to verify the
   calculated corrosion rates with actual gravimetric mass loss of the steel bars.

3) Other easy to measure properties of corroding RC structures such as corrosion crack
   widths and their propagation were not monitored during the experiment.
2.3.2 Ballim et al. [61]

The program involved testing RC specimens of dimensions $100 \times 160 \times 1550$ mm. The beams were tested under load over a span of 1050 mm in a 4-point bending configuration with a constant moment region of 350 mm in the middle span. The load on the beams was applied using a compressed spring and then transferred to the test specimens via a load distributor as shown in Figure 2–3. The bottom supports of the beams and underside of the beams (tensile region) were placed inside a tank containing a 3% solution of NaCl. Corrosion of tensile steel bars was accelerated by impressing current. This work was divided into two main sets; set 1 beams were tested under a load equivalent to 23% of the ultimate load; and set 2 beams were tested under a load equivalent to 34% of the ultimate capacity of beams. Central deflections were measured on the beams, and after testing the beams to failure, corroded bars were retrieved from the concrete, cleaned and weighed to determine the gravimetric mass loss.

The gravimetric mass loss of steel bars was found to be consistently lower than the mass loss predicted using Faraday's Law. Following the corrosion process, beams were removed from the load rigs and corrosion crack maps were prepared. It was found that whilst corrosion crack maps were similar for both beams, beams corroded under higher load showed more extensive and wider cracks. Some spalling of the concrete cover was even observed in these beams. Since the frame was in contact with NaCl solution, it was concluded in the paper that some of the applied current was lost to the corrosion of the frame. It was however observed that beams subjected to higher load levels experienced larger mass loss. These results are not conclusive enough to compare with mass loss on
beams corroded in the absence of a sustained load where there would be no loss in current to the frame.

![Diagram of loading frame](image)

**Figure 2–3 Loading frame used by Ballim et al. [61]**

Whilst this work provided further insight on the behavior of beams corroded under a sustained load, some of the results obtained are questionable due to the nature of the testing program and there were some critical research areas that were not addressed:

1) As stated in the publication, there was corrosion of the frames which affected deflection measurements and reduced the total current to corrode the steel bars.

2) From basic mechanics, it is clear that the spring system used was such that when the beam deflected, the springs relaxed and hence the force applied on the beams reduced. Based on the provided spring constant and the deflection of beams, beams in sets 1 and 2 lost loads of about 0.6 kN and 1 kN respectively.

3) Even though deflections were clearly found to increase with an increase in the degree of corrosion and the level of the applied load, other parameters of structural behavior such as variation of strains were not continuously measured. Deflections could therefore only be associated with the loss in the area of steel predicted from Faraday's
Law and the level of applied loads but not with curvatures and stiffness. There was as such no basis for explaining why higher load level yielded larger deflection ratios.

4) Crack maps were conducted at the end of the experiment such that there was no indication of sequence and propagation of the cracks. This is especially because beams were always immersed in a NaCl solution and the corroding section of the beams was inaccessible.

5) In upcoming research where the focus on corrosion of beams is shifting towards the effectiveness of repair and strengthening of corroded structures, it is very difficult to break the concrete and repair it whilst under load using this test frame.

2.3.3 El Maaddawy et al. [54]

This work involved corroding eight quasi-full-scale RC beams with dimensions of 152 × 254 × 3200 mm to different corrosion levels. Four beams were tested under a sustained load equivalent to 60% of the ultimate load of a virgin beam whilst corresponding four beams were tested in the absence of a sustained load to act as controls. The load was applied using a mechanical loading frame shown in Figure 2–4. The frame was such that two beams were loaded simultaneously; the bottom beam with tensile face up and the top beam with tensile face down. The frame applied load on specimens by hanging weights on a lever beam, and the load was then transferred to another lever beam before being transferred to the test beams by a load distribution beam. The two lever arms were designed to amplify the applied load. The loading system self-adjusts as the beams deflect and hence always applied a constant load on the beams. The loading arrangement was such that the beams were tested with a 4-point bending over a span of 3000 mm with a constant moment region of 1000 mm in the middle. Corrosion was accelerated using an impressed current
under a controlled humidity and with a continuous spray of mist on beams. It was restricted
to the tensile steel reinforcement in the middle 1400 mm of the specimen by casting
concrete with salt only on the required corrosion region. At the early testing stages, beams
were constantly monitored for the appearance of the first corrosion crack. It was found that
for beams corroded under load, corrosion cracks appeared after 53 h whilst for beams
corroded in the absence of a sustained load, cracks were observed after 95 h. It is worth
noting that the difference between 95 h and 53 h seem small but the beams in the program
were corroded at a corrosion density that is up to a thousand times more than the corrosion
density of in-service structures. If the rate between the two periods (≈ 2) is also the same
for in-service structures where the time to cover cracking is in the order of tens of years,
then a 50% reduction of the time to cover cracking due to a sustained load is very
significant. This indicates that various models developed to estimate the time from
corrosion initiation to the time of cracking as an attempt to model the service life of
corroding structures are probably calibrated with inaccurate data. Current models are
therefore likely to overestimate the service life of real structures. During the corrosion
process, corrosion crack widths at the middle of two beams to have the highest level of
corrosion were measured using a demec gauge. The results indicated that the crack width
at the center of the beams monotonically increased overtime but at a decreasing rate,
regardless of the applied load. A decrease in the rates of crack opening was ascribed to the
accumulation of corrosion products around the reinforcing bar. This argument implies that
the rates of corrosion decreased with an increase in the level of corrosion, which is in
contrast with studies by [62].
Figure 2–4 Loading frame used by El Maaddawy et al. [54]

Clearly this work was more detailed than the previous two works [61, 62] on the effects of loading on the behavior of corroded beams. However, there still remain some areas that were left unclear:

1) The arrangement of the beams on the test frames was such that there was limited access to the tensile faces of the beams (especially for the top beam) during the corrosion process which is where the severest level of corrosion cracking was found. It is therefore difficult to use the test frame to repair structures whilst under a sustained load.

2) Whilst corrosion cracks were continuously monitored at the center of the corrosion region for selected beams, it is not clear how they propagated along other sections of the regions over time. The crack maps provided also clearly indicate that the maximum crack width did not necessarily occur at the center of the corrosion region. This casts doubts over the influence of loading on the variation of central corrosion crack widths reported in the paper.

3) Even though mass loss was measured at different areas along the bar, it was averaged so that the crack maps drawn at the end of the corrosion process cannot be related to
the mass loss at every point along the beam. It is therefore difficult to associate the location of the maximum crack width with the location of the maximum mass loss. This relation is however critical for associating ultimate capacity with easy to measure properties of a corroding structure [6].

2.3.4 Vidal et al. [43]

In this work, 72 quasi-full-scale RC beams (150 × 280 × 3000 mm) were tested under load. The load was applied as a three point flexure by placing beams back-to-back horizontally; bottom beams were tested face up whilst top beams were tested face down. The beams were fastened onto each other at the middle as shown in Figure 2–5. Two load levels were induced; 50% and 80% of the ultimate capacity of a virgin beam. 36 beams were corroded under load whilst the other 36 were used as controls and were not corroded but were also placed under load. The accelerated corrosion process was archived by:

1) 0 to 6 years: continuous spraying under laboratory conditions ($T^\circ \approx 20 ^\circ C$).

2) 6 to 9 years: cycles spraying under laboratory conditions ($T^\circ \approx 20 ^\circ C$), one week of spraying and one week of drying.

3) 9 years to now: cycles spraying, one week of spraying and one week of drying, however the confined room was transferred outside, so the beams were exposed to the temperature of the south-west of France climate, ranging from $-5 \ ^\circ C$ to $35 \ ^\circ C$.

Most of the structural behavior on this test involved removing the beams from the loading systems and testing them to failure on another system. It is as such difficult to appreciate the effects of loading on the behavior of corroded beams. The loading system is not a mechanism such that it did not self-adjust as the beams deflected. The tensile face of the beams was also hidden which made it difficult to continuously monitor corrosion
damage on the tensile face of the beams or to use the frame to repair the beams whilst under load

Figure 2–5 Loading frame used by Vidal et al. [43]

2.3.5 Malumbela et al. [55, 63]

In this work, beams of dimensions $[153 \times 254 \times 3000 \text{ mm}]$ are tested under various loading conditions (0%, 1%, 8% and 12% of the ultimate capacity of a virgin beam). Low loading (1%) are achieved by testing beams whilst supported on two concrete blocks of $100 \times 100 \times 200 \text{ mm}$ placed in the middle third of the beam span whilst higher loads (8% and 12%) are applied using a mechanical loading frame shown in Figure 2–6. The sustained loading level is much lower than that used by other researchers [43, 54, 61, 62], which range from 20% to 80% of the ultimate capacity of beams. The load on the test frame is applied by hanging weights on a loading beam and transferred to the test specimens by a spreader beam. Bearing supports and pinned struts are used to transfer the load from the loading beam to the spreader beam and to allow for a free vertical movement of the loading system. A free vertical movement of the loading system ensures that the test specimens are always under a constant sustained load.
Corrosion of beams is accelerated using an impressed current and cycles of four days wetting with a 5% solution of NaCl and two days drying in natural air. It is limited to a length of 700 mm in the middle part of the beams by building a NaCl pond on the tensile face of the beams. Longitudinal strains on the tensile face of the beams and on the compression side of the beams are measured before and after each wetting cycle using a 100 mm demec gauge. Lateral strains on the top tensile faces and side faces of the beams are also monitored and converted into corrosion crack widths.

The test results corrosion crack widths were found to be larger on beams corroded under load than beams corroded in the absence of a sustained load. It is therefore apparent that a sustained load has a major influence on the structural behavior of beams corroded under load.

![Figure 2–6 Loading used by Malumbela et al. [55, 63]](image)

This work is the newest experimental program considering the sustained loading conditions during the accelerated corrosion during. However, there still some areas that were unclear:
1) The strain gauge used to monitor the corrosion crack process. The problem is that the corrosion crack appeared randomly on the surface of the specimens, which is hard to choose the location to place the strain gauges.

2) The test concluded that the ultimate capacity of corroded specimens was control by the maximum metal loss, which may be related with its experimental setup. The corrosion area was limited in the middle part of the beams, which caused the serious corrosion damage to local area of the reinforcements.

3) The sustained loading level is lower than real service condition, which is around 60% of the ultimate capacity of un-corroded RC structures.

2.4 Epoxy-coated reinforcements an PP fibers

To prevent corrosion damage of reinforcing bars in reinforced concrete (RC) structures exposed to aggressive corrosion environments, there are two general methods used widely in practice – i) using various kinds of coated reinforcements and ii) adding fibers into plain concrete. Among the coated reinforcements, epoxy coatings have been applied to surfaces of reinforcements since mid-1970s [97]. The usage of epoxy-coated reinforcements is one of the most effective methods for protecting reinforced concrete structures from corrosion damages [68-74]. The protection of epoxy coatings primarily depends on the integrity of the coatings. However, the coating surface may sustain damages during the handling of the epoxy-coated reinforcements.

Except using coated reinforcements, adding fibers into plain concrete is another way to improve the corrosion resistance of RC structures by reducing the crack width growing of cover concrete. Many studies [75-83] have been conducted to characterize in the mechanical properties of reinforced concrete with fiber additives. Such concrete is used
commonly in retrofitting and repairing the covering of concrete structures. According to these studies [75-83], the increase of formability and bending strength are considered as advantages of adding fibers into the concrete. Most attention among previous research has been focused on polypropylene (PP) fibers because of its low cost, outstanding toughness and enhanced shrinkage cracking resistance of the concrete [75-83, 98]. Sanjuan et al. [98] added PP fibers into pure concrete cylinder specimens in carbonation corrosive environment to control the cracking and improve the durability of concrete. Low amounts of PP fibers were used in their study with a finding of beneficial effect of fiber addition on the corrosion rate.

2.5 Theoretical prediction methodology of ultimate capacity loss of corroded RC specimens

Usually, the corrosive environment along the structures cannot be exact same which means the corrosion rate is changing place to place, especially for the structures contacted with de-icing salts. If the corrosion rate varies along the corroding length, which will cause the variation of the mass loss along the corroding reinforced bars. So far, different researchers have different opinions about prediction the residual ultimate capacity of corroded specimens by using average gravimetric mass loss or using maximum local mass loss. Azad et al. [8] shew that at high desired corrosion level (20%), average gravimetric mass loss overestimates the residual ultimate capacity of corroded beams. Because Azad et al. [8] did not considered the hook design in his experiment specimens, the authors attributed the large reduced ultimate capacity of beams to the loss in the bond between the steel and the concrete. Torres-Acosta et al. [6] also showed that average radius loss calculated by average gravimetric mass loss had a substandard relation with the ultimate
capacity of corroded RC beams. Azad et al. [8] also concluded that the average section loss can be used to predict residual capacity of tested specimens with low desired corrosion level. Fasl J. et al. [20] adopted the corroded reinforcing bars taken from real RC slabs with new concrete. Based on their test, the localized section loss of a single bar did not control the slab behavior; instead, slab flexural behavior was governed by the average section loss of all the reinforcing bars.

So far, no research considered the reduction yield strength of the corroded reinforced bars to predict the ultimate capacity loss of corroded RC structures. However, a lot of researchers [10-20] did studies on mechanical properties of corroded reinforcing steel bars. The general conclusion was that the corroded reinforcing steels exhibited less ductility compared with un-corroded ones and the ultimate strain was significant decreases with the corrosion section loss [10, 11]. Most studies [10-12, 18] adopted the corroded reinforcing steel bars after removing the surface rust to do standard tensile test, which proved that the nominal yield strength of corroded reinforcing steel bars was decreasing with the increasing of corrosion level. The degree of reinforcement corrosion resulting decrease in the yield strength and the cross section of steel bars needs to be evaluated to assess the ultimate capacity loss of the RC member and formulate repair strategies.

2.6 Prediction model for cracking of cover concrete (serviceability)

Extensive studies have been devoted to predict cover-concrete cracking, including experimental study, theoretical models and finite element simulations. The past experimental programs [22, 48, 95, 96, 99] have mainly focused on finding the critical amount of the steel corrosion needed for cracking concrete cover and developing the corresponding empirical predictive models. Several theoretical models [2, 21-30] were
developed to predict the critical expansion pressure to cause concrete cracking using thick-walled cylinder theory. Most of these theoretical models [22, 25, 27-29] were based on the elastic theory or the elastic-plastic theory and often neglect the tension softening characteristics (fracture energy) of concrete. Furthermore, most theoretical models had been developed based on the assumption of uniform corrosion pattern [2, 21-30].

Finite element (FE) method has facilitated an effective tool as an additional approach to study the effect of steel reinforcement corrosion on concrete [31-39]. However, the previous FE models [34-36] can only adopted the stress distribution and path to represent the corrosion crack path of RC structure. The advantage of extend finite element method (XFEM) [100] offers a better simulation approach to evaluate cracking behavior of RC structure due to corrosion with cracking of the elements.
3.1 Introduction

Since the 1960s and 1970s, following widespread use of de-icing salts on highways within the United States and a construction boom in the Arabian Gulf, research has been undertaken worldwide to address corrosion issues. Theoretical models have also been developed and calibrated with experimental results to predict the behavior and service life of concrete structures with corroding steel bars [6, 8, 22, 23, 52, 89, 94, 96, 101, 102]. Malumbela et al. [29, 84, 85] performed a critical review and experimental work with beam specimens and found that while in-service, RC structures normally corroded under sustained loading [103, 104]. The majority of articles on corrosion have focused on structures that corroded in the absence of sustained loading [1, 5, 6, 8, 21, 23, 42, 43, 47, 50, 52, 53, 61, 62, 67, 86-96, 105-107]. Since 1999, some researchers have examined sustained loading effects [42, 43, 61-63, 88, 89, 91]; however, none of these studies focused on corrosion during the service life of the structural components of RC structures having pre-existing surface cracks.

Cracks in reinforced concrete elements are a common occurrence due to the low tensile strength of concrete. Cracking that occurs in reinforced concrete structures at service load (i.e., during the normal use) cannot generally be avoided. Among the studies
published after 1999, only a study by Yoon (2000) [62] considered preloaded test specimens where cracks existed prior to accelerated corrosion testing, but the test program in this study did not use a sustained loading condition. Sustained service loads on structures, surface cracks, or a combination of both conditions can affect the rate of corrosion, the residual moment strength of corroded RC members, and the effectiveness of repairs. Experiments conducted and theoretical models developed without consideration of pre-existing cracks and sustained loading conditions do not capture the true behavior of in-service RC structural elements that are subjected to corrosion.

This chapter highlights that two important factors — (i) pre-existing cracks and (ii) sustained loading — that need to be considered in accelerated corrosion tests and to determine the influence of these two factors on the mass loss of the reinforcements as well as the crack expansion rate and the patterns of cracks on the surface of the concrete specimens. To address these factors, concrete slab specimens having steel bar reinforcements of the same size that were prepared using a single concrete mix were loaded to introduce cracks prior to the application of sustained loading and external induced current to accelerate the corrosion process. The electrical current applied in the accelerated corrosion test was based on theoretical mass loss design levels (10%) calculated from Faraday’s law. The experimental results were used to compare the theoretical calculations of mass loss and the actual measured mass loss. The ultimate load carrying capacity of the corroded specimens with various experiment conditions (different wetting and drying cycles and sustained loading levels) were measured and compared. The mechanisms regarding the effect of the transverse cracks were developed based on the observations of this experimental program to explain the different characteristics of the longitudinal crack
expansion rate of specimens corroded under different test conditions (i.e., with or without pre-cracking and sustained loading).

3.2 Motivation of experiment work related to corrosion of slab specimens

Few corrosion-related experimental studies [62, 67] on slab specimens are reported in the literature. The motivations for conducting accelerated corrosion experiments on slab specimens with and without pre-existing cracks and sustained loading can be enumerated as follows:

1) Pre-cracked condition has not been considered by researchers in earlier studies for RC slab elements. Consequently, there is a need to study the effects of this condition.

2) Sustained loading (from dead load and live loads) exists during the service of continuous span structural slab bridges exposed to de-icing salts. Exposure to these salts should be simulated in the accelerated corrosion test.

3) The wetting–drying cycle is an important experimental condition for accelerated corrosion tests employed to simulate bridges subjected to a corrosive environment due to the use of de-icing salt, since de-icing salt is not applied continuously during the winter season.

Structural slabs do not have shear reinforcement (stirrups) around the main tensile reinforcing bars and therefore behave differently from beams, as shown in Figure 3–1. The failure modes of corroded RC slabs need to be assessed and compared with those for un-corroded RC slabs.
3.3 Test setup

All specimens were cast using concrete (ASTM Type I Portland cement) with a cement content of 335 kg/m³ (564 lb/yd³), a coarse aggregate to fine aggregate ratio of 1.3, and a water-to-cement ratio (w/c) of 0.44. A total of 21 slab specimens with dimensions of 305 × 76 × 711 mm (12 × 3 × 28 in) were cast in four batches. For each batch of concrete, several cylinders that were 102 mm (4 in) in diameter and 204 mm (8 in) in length were also cast to determine the compressive strength of each batch of concrete. The reinforcement configuration of the specimens is shown in Figure 3–2. The test specimens were placed in a curing room for 28 days approximately 24 hours after casting. The curing room maintained the required humidity and temperature during the entire curing period.
After curing, three un-corroded specimens were subjected to a three-point bending test to determine the ultimate load capacity. The flexural strength is expressed in terms of the critical load when the specimen reached the failure point. The collapse load recorded from the tests and the average of the three tests is shown in Table 3-1. Figure 3–3 shows the test setup and the failure mode of an un-corroded specimen, with typical flexural cracks.

**Table 3-1 Ultimate load carrying capacity for un-corroded specimens**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-corroded specimens</td>
<td>42 kN</td>
<td>36 kN</td>
<td>42 kN</td>
<td>40 kN</td>
</tr>
<tr>
<td></td>
<td>(9,448 lb)</td>
<td>(8,187 lb)</td>
<td>(9,433 lb)</td>
<td>(9,023 lb)</td>
</tr>
</tbody>
</table>

(a) Test setup          (b) Typical test specimen after bending failure

Figure 3–3 Flexural test for un-corroded slab specimen
3.4 Specimen groups and their designation

Concrete cylinders were used for compression testing to determine the compressive strength of the concrete used for making the test specimens. The average compressive strength varied from batch to batch, despite the use of the same mixture proportions, same materials, and a similar casting procedure. All cylinders are cured for at least 28 days but were tested on different days. The measured values, taken as the average strengths of all cylinders, varied from a minimum of 38 Mpa (5484 psi) to a maximum of 58 Mpa (8266 psi) with a standard deviation of 5 Mpa. For the #4 (12.7 mm) grade 60 reinforcing bars used in the test program, the specified yield strength was 414 Mpa (60 ksi).

The remaining 18 specimens of the program were divided into two sets (Set A and Set B), where each set is further divided into three different groups, representing the conditions of no pre-existing cracks and sustained loading (Group 1), pre-existing cracks specimens (Group 2), and both pre-existing cracks and sustained loading (Group 3). For specimens in Set A, the theoretical corrosion level based on Faraday’s law is 10% of steel mass loss, and the applied sustained loading level is 12% of the un-corroded specimens’ capacity. For specimens in Set B, the theoretical corrosion level based on Faraday’s law is 10% of steel mass loss, and the applied sustained loading level is 60% of the un-corroded specimens’ capacity. The detailed description of the different specimen groups based on the pre-existing cracks and sustained loading conditions is in Table 3-2. In addition, specimens in Set A were subjected to two days of wetting and no drying time, while specimens in Set B were subjected to two days of wetting and one day of drying. The total experiment time of specimens in Set A is 14 days (only wetting time) and the total
experiment time of specimens in Set B is 21 days (14 days of wetting and 7 days of drying). The accelerated corrosion process only occurred during the wetting time, so the electrical current used in this program for the two sets to reach the theoretical corrosion level (10%) was the same.

3.5 Procedure for introducing pre-cracked condition

A three-point loading configuration was used to induce bending moments with one load at the mid-span and two supports at the end of each specimen. The distance between the two end supports of the test specimens was 610 mm (24 in). Load was applied on a universal testing machine to introduce cracks in the test specimens. Among the theoretical equations for crack propagation, ACI 318R-95 [108] and Oh and Kang’s [109] equations were reported to be better for predicting surface crack width, as compared to the experimental values of crack width [110, 111]. Therefore, these two equations were used in the test program to control the first crack produced in the test specimen without causing severe damage to the test specimens prior to the initiation of the corrosion process. The expression of ACI 318R-95 and the expression of Oh and Kang are shown in Equations (3.1) and (3.2), respectively.

\[
\text{ACI:} \quad W_{max} = 0.011\beta f_s \sqrt[3]{d_c A_0}
\]

\[
\text{Oh and Kang:} \quad W_{max} = \phi a_0 (\epsilon_s - 0.0002)\beta
\]

where \( W_{max} \) is the maximum crack width; \( \beta \) is the ratio of the distance between neutral axis and extreme tension face to the distance between the neutral axis and the centroid of reinforcing steel; \( f_s \) is the stress in the tension reinforcement calculated on the basis of a cracked section; \( d_c \) is the distance measured from the centroid of tensile steel to the edge.
<table>
<thead>
<tr>
<th>Group Name</th>
<th>Specimen label</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Set A</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Group 1</td>
<td>N-1</td>
<td>No pre-cracked or sustained loading applied during accelerated corrosion test</td>
</tr>
<tr>
<td></td>
<td>N-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>N-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-1</td>
<td>Pre-cracked before applying accelerated corrosion current</td>
</tr>
<tr>
<td>Group 2</td>
<td>P-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td></td>
</tr>
<tr>
<td>Group 3</td>
<td>PS-1</td>
<td>Pre-cracked and sustained loading during corrosion process</td>
</tr>
<tr>
<td></td>
<td>PS-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS-3</td>
<td></td>
</tr>
<tr>
<td>Subject to wetting and no drying time</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Set B</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Group 1</td>
<td>N-1</td>
<td>No pre-cracked or sustained loading applied during accelerated corrosion test</td>
</tr>
<tr>
<td></td>
<td>N-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>N-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-1</td>
<td>Pre-cracked before applying accelerated corrosion current</td>
</tr>
<tr>
<td>Group 2</td>
<td>P-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td></td>
</tr>
<tr>
<td>Group 3</td>
<td>PS-1</td>
<td>Pre-cracked and sustained loading during corrosion process</td>
</tr>
<tr>
<td></td>
<td>PS-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS-3</td>
<td></td>
</tr>
<tr>
<td>Subject to two days wetting and one day drying</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note: N – No pre-cracked and sustained loading; P – Pre-cracked condition; PS – Pre-cracked and sustained loading condition*
of the section on tension side; $A_0$ is the concrete area surrounding each reinforcing bar; $\Phi$ is the diameter of tensile reinforcing bar; and $a_0$ is the coefficient that is a function of certain variable as defined in the reference; and $\epsilon_s = \frac{f_s}{E_s}$. A more detailed explanation of all items in two formulas can be found in ACI code [108] and Oh and Kang [109].

Typically, the minimum width of visible cracks under normal conditions is approximately 0.13 mm (0.005 in), as shown Figure 3–4. If the point load is removed, the bending crack will usually close and the crack will not be visible. Table 3-3 shows the point load value when the first visible bending crack appeared on the surface of the test specimens. The theoretical value is approximately 13.3 kN (3000 lb), which is calculated based on the theoretical equations. The load applied to the slabs in the testing program to create the first crack was $13.3 - 17.8$ kN (3000 – 4000 lb), which means the pre-cracked load used in this testing program did not induce the tensile reinforcements to yield prior to corrosion testing.

Figure 3–4 Appearance of first crack during the stage of generating pre-existing cracks
Table 3-3 First cracking load from tests and the theoretical cracking load

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Set A</th>
<th>Set B</th>
<th>ACI</th>
<th>Oh and Kang [33]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-1</td>
<td>15 kN (3,300 lb)</td>
<td>14 kN (3,100 lb)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-2</td>
<td>16 kN (3,500 lb)</td>
<td>14 kN (3,100 lb)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-3</td>
<td>14 kN (3,200 lb)</td>
<td>13 kN (3,000 lb)</td>
<td>14 kN</td>
<td>13 kN</td>
</tr>
<tr>
<td>PS-1</td>
<td>15 kN (3,400 lb)</td>
<td>16 kN (3,500 lb)</td>
<td>(3,158 lb)</td>
<td>(3,039 lb)</td>
</tr>
<tr>
<td>PS-2</td>
<td>15 kN (3,500 lb)</td>
<td>13 kN (3,000 lb)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PS-3</td>
<td>13 kN (3,000 lb)</td>
<td>16 kN (3,500 lb)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.6 Accelerated corrosion procedure

Because the nature corrosion test is time consume, most corrosion experimental researchers will apply accelerated corrosion test to get the results.

3.6.1 Theoretical determination of electrical current

Faraday’s law was used to calculate the theoretical steel mass loss of reinforcements embedded in each specimen when subjected to accelerated corrosion using impressed current. The relation between the steel loss and current flow is as follows:

\[ \Delta m = \frac{MIt}{zF} \]  

where \( \Delta m \) is the mass of steel consumed in grams; \( M \) is the atomic weight of the metal (56 grams or 0.1232 lb for Fe); \( I \) is the current (amperes); \( t \) is time (seconds); \( z \) is the ionic charge (which is equal to 2); and \( F \) is Faraday’s constant (96,500 amperes/seconds).

In this study, the total corrosion time of 14 days (1,209,600 seconds) was used. The applied current value determined from Faraday’s law that is required to reach the theoretical corrosion level in two weeks is shown in Table 3-4. As mentioned in Section 4,
specimens of two sets in this program had the same wetting time (14 days), so the electrical
current value used to induce accelerated corrosion was the same for all specimens. After
reaching the desired corrosion level, the specimens were then subjected to a three-point
bending test to determine the residual ultimate load carrying capacity.

Table 3-4 Accelerated current value for theoretical steel mass loss levels

<table>
<thead>
<tr>
<th>Theoretical mass loss</th>
<th>Corrosion length</th>
<th>Original mass</th>
<th>Mass loss</th>
<th>Time (seconds)</th>
<th>Current value (amperes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>1,118 mm (44 in)</td>
<td>1,100 grams (2.45 lb)</td>
<td>111 grams (0.245 lb)</td>
<td>1,209,600</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Note: The mass loss is represented by the percentage of the original steel weight

3.6.2 Corrosion test setup

Figure 3–5 shows the schematic representation of the accelerated corrosion test setup.
The DC power supply is used to generate electrical current to facilitate accelerated
corrosion, where a stainless steel plate is submerged in the salt solution and connected to
the cathode of the DC power supply and the reinforcements embedded in the concrete are
connected to the anode of the DC power supply. The sustained loading was applied from
the bottom of the test specimen. Photos of test specimens in the corrosion process with and
without sustained loading are shown in Figure 3–6a and Figure 3–6b, respectively. The RC
slab is partially exposed to 5% NaCl solution to allow for chloride ions to permeate the
concrete cover and reach the reinforcements embedded in concrete. The sustained load was
monitored by a load cell, while the crack widths on the surface of the specimen were
measured using a micrometer.
3.7 Test results and discussion

After the experimental work, the entire test data were collected and analyzed. Based on collected data, several important results can be summarized.

3.7.1 Capacity and metal loss after accelerated corrosion

After the specimens were subjected to the accelerated corrosion within the specified corrosion period at different test conditions, the specimens were removed from the corrosion cells and were subjected to bending tests to determine their ultimate load carrying
capacity. For comparison purposes, the crack patterns and load–deflection curves for un-corroded specimens and corroded specimens at the collapse load are shown in Figure 3–7a and Figure 3–7b, respectively. By comparing the crack patterns, it was observed that the failure modes were different: the un-corroded slab specimen exhibited several flexural cracks at collapse load, while the corroded slab specimen only exhibited one wide crack running in the transverse direction. Figure 3–7b shows the load-deflection curves for the corroded specimens and un-corroded specimens. The corroded specimens were observed to suffer a larger deflection with a smaller load than those of the un-corroded specimens.

The ultimate load carrying capacity for the corroded specimens of different groups in different sets is listed in Table 3-5. The ultimate load carrying capacity reduction values listed in Table 3-5 were calculated by using the following equation:

\[
\text{Ultimate loading carrying capacity reduction} = (1 - \frac{\text{Ultimate load of corroded specimens}}{\text{Ultimate load of un–corroded specimens}}) \times 100 \quad (3.4)
\]
Table 3-5 Mass loss and ultimate load carrying capacity for corroded specimens

<table>
<thead>
<tr>
<th>Experiment Set</th>
<th>Experiment Group</th>
<th>Number of the specimens</th>
<th>Mass loss</th>
<th>Ultimate load carrying capacity reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Average mass loss</td>
<td>Maximum mass loss</td>
</tr>
<tr>
<td>Set A</td>
<td>Group 1</td>
<td>3</td>
<td>5.2%</td>
<td>6.1%</td>
</tr>
<tr>
<td>Set B</td>
<td>Group 1</td>
<td>3</td>
<td>7.7%</td>
<td>8.8%</td>
</tr>
<tr>
<td>Set A</td>
<td>Group 2</td>
<td>3</td>
<td>7.6%</td>
<td>11.2%</td>
</tr>
<tr>
<td>Set B</td>
<td>Group 2</td>
<td>3</td>
<td>9.5%</td>
<td>14.8%</td>
</tr>
<tr>
<td>Set A</td>
<td>Group 3</td>
<td>3</td>
<td>9.7%</td>
<td>12.6%</td>
</tr>
<tr>
<td>Set B</td>
<td>Group 3</td>
<td>3</td>
<td>12.2%</td>
<td>17.6%</td>
</tr>
</tbody>
</table>

Note: Moment capacity reduction is represented by the percentage of un-corroded samples shown in Table 3-1.

In order to evaluate the steel mass loss variation along the length of the rebar, the specimen surface that was exposed to the salt solution was divided into ten segments of 50.8 mm (2 in) in length, as shown in Figure 3–8a. The corroded reinforcements obtained by removing concrete indicated that the corrosion products were concentrated on the top surface of the bars (i.e., the surface closer to the slab face that was exposed to the salt solution). From the visual comparison of the corrosion concentration for specimens with different corrosion conditions, it was found that specimens corroded with pre-existing cracks and sustained loading exhibit more corrosion products.

After cleaning the corrosion products from corroded reinforcements using a special acid solution, the reinforcements were cut into small coupons that were 50.8 mm (2 in) in length, corresponding to the division of the specimen surface, as shown in Figure 3–8b and
Figure 3–8c. The steel mass loss of reinforcements in the coupons (Figure 3–8c) was determined by the following equation:

\[
\text{Steel mass loss} = (1 - \frac{\text{residual weight of steel coupon}}{\text{original weight of steel coupon}}) \times 100 \tag{3.5}
\]

Figure 3–8 Steel coupons of corroded bars and division of the surface of the specimen

The variation of steel mass loss along the reinforcement length is shown Figure 3–10a and Figure 3–10b for Set A and Set B, respectively. It can be observed from Figure 3–10 that the general trend of steel mass loss along the direction of the reinforcements can be represented by a parabola with a maximum vertex close to the middle of the span of the specimens. However, the specimens corroded without pre-existing cracks and sustained loading do not show this type of variation. It was also observed that the steel mass loss
extended to the full length of the reinforcement, even beyond the ends of the exposure region. As shown in Figure 3–10, the level of steel mass loss and its variation along the longitudinal direction of the slab appears to relate to the specimen conditions. The specimens with pre-existing cracks subjected to sustained loading during the accelerated corrosion process exhibited more severe steel mass loss. For the specimens corroded without pre-existing cracks and sustained loading in Group 1, the mass loss along the longitudinal direction of the reinforcement is quite uniform.

Table 3-5 shows the larger mass loss by comparing the specimens in the same group in Set B (10% theoretical corrosion level and with two days of wetting and one day of drying) than the mass loss detected from Set A (10% theoretical corrosion level and without drying time). The most obvious reason why slabs with longer drying cycles exhibit a larger mass loss of steel is that the longer drying cycles could have allowed for more natural corrosion to occur because of the extended time required to reach the desired time of electrolysis [51].

![Graph](image)

(a) Specimens of Set A

Figure 3–9 Variation of mass loss of steel along the specimens within the tank
3.7.2 Observation of Corrosion cracks

One telltale sign of corrosion of reinforcements in concrete slabs/beams is the development of surface cracks. A commonly accepted explanation is that a higher corrosion level induces a larger amount of corrosion products, which in turn causes a larger crack width as a result of volume expansion of the corrosion products. However, few researchers have paid attention to the crack patterns under pre-existing cracks and sustained loading experimental conditions. In our experimental program, we observed the development of both longitudinal cracks and transverse cracks for the specimens under these combined test conditions. Because the appearance of transverse cracks under the loading and corrosion conditions is random, only the maximum transverse crack widths of test specimens are monitored. The maximum transverse crack widths for specimens in Set A and Set B are shown in Figure 3–11a - b, respectively. The specimens in Group 3
(specimens with both pre-existing cracks and sustained loading) exhibit wider transverse cracks than those in Group 2 (specimens with only pre-existing cracks). Clearly, the sustained loading in Group 3 contributed to the larger transverse crack widths.

The maximum longitudinal crack widths of all specimens were also measured and recorded. Based on comparison of the crack widths observed in Set A (10% theoretical corrosion level without drying time) with those in Set B (10% theoretical corrosion level with two days of wetting and one day of drying), the following observations can be made:

1) A longer corrosion time lead to larger crack widths, which can be seen in Figure 3–11 and Figure 3–12 for both longitudinal and transverse cracks. The difference in wetting and drying cycles between Set A and Set B does not significantly affect the crack expansion for both longitudinal and transvers cracks when comparing the final crack widths shown in Figure 3–11 and Figure 3–12.

2) In Figure 3–12a, it can be noticed that specimens in Group 3 (those with both pre-existing cracks and sustained loading) of Set A (10% theoretical corrosion level and without drying time) have the maximum longitudinal crack width. This could be attributed to the fact that the pre-existing cracks and sustained loading leads to the development of transverse cracks, which allow the chloride ions from the salt solution to penetrate into the concrete and reach the surface of reinforcements more rapidly. Therefore, the corrosion reaction occurs at an earlier stage in the specimens of Group 3 than that in the other two groups. A similar phenomenon of longitudinal crack width expansion was detected in the specimens in Set B, as shown in Figure 3–12b.

3) At the end of the experiment, the widest longitudinal cracks observed in specimens in Group 1 (specimens without pre-existing cracks and sustained loading) were similar to
or larger than those in the other two groups. This reverse trend of the longitudinal crack width in the earlier stage and late stage of the experiment may be explained by the hypothesis that the transverse cracks appearing in Group 2 (specimens with only pre-existing cracks) and Group 3 (specimens with both pre-existing cracks and sustained loading) provided additional space for corrosion products to fill, thus releasing the hoop tensile stress caused by the volume expansion of corrosion products. Therefore, transverse cracking could reduce the increase rate of hoop tensile stress caused by corrosion products.

Specimens in Group 2 (specimens with only pre-existing cracks) showed a similar crack width as the specimens in Group 3 (specimens with both pre-existing cracks and sustained loading). It appears that the net effect of sustained loading in creating transverse cracks on the longitudinal crack expansion is the combination of two contradictory mechanisms: accelerated corrosion rate due to faster ingress of chloride ions and reduced hoop tensile stress due to the opening of transverse cracks.
(a) Maximum transverse crack width for Set A
Figure 3–11 Transverse crack width results

(b) Maximum Transverse crack width for Set B
Figure 3–11 Transverse crack width results
(a) Maximum longitudinal crack width for Set A

(a) Maximum longitudinal crack width for Set B

Figure 3–12 Longitudinal Crack width results
3.8 Conclusions

Based on the experimental results that were obtained in this study, the following conclusions were drawn:

1) Corroded RC specimens subjected to three-point bending loads have different failure modes than un-corroded specimens. The un-corroded specimens exhibit flexural failure with several bending-induced cracks in transverse direction, while corroded specimens only exhibit one large transverse crack under bending loads.

2) Maximum mass loss of steel bars was found to be close to the middle span of specimens with pre-existing cracks.

3) Specimens with pre-existing cracks and sustained loading during the accelerated corrosion process exhibited the most severe level of corrosion-induced damage in terms of steel mass loss and the reduction in ultimate load carrying capacity.

4) In the experimental program, the pre-existing cracks and sustained loading conditions have been shown to exert an influence on both the crack width expansion rate and crack pattern. The pre-existing cracks combined with sustained loading were found to induce transverse cracks with greater widths. The development of transverse cracking was further shown to affect longitudinal crack development. At earlier stages of the corrosion duration, the transverse cracks allow a faster corrosion rate, resulting in a faster expansion rate of longitudinal corrosion cracks. At later stages of the corrosion duration, the large transverse cracks provide spaces and paths for the corrosion products to fill, thus releasing the hoop tensile stress, which in turn reduces the rate of longitudinal crack expansion.
5) The wetting and drying cycles incorporated in the corrosion process were shown to have an effect on the steel mass loss, but they did not cause a large difference in longitudinal and transverse crack widths.
CHAPTER IV

EXPERIMENTAL STUDY OF RELATION BETWEEN CORROSION CRACK WIDTH AND METAL LOSS FOR REINFORCED CONCRETE SLABS

4.1 Introduction

Because of the extreme winter weather conditions in Northeast America, where de-icing salt is used frequently in winter time, bridges in this region are more susceptible to deterioration caused by chloride-induced corrosion. Continuous freeze and thaw actions coupled with chloride ingress from de-icing salts used during winter time particularly deteriorate bridge decks. For the continuous slab bridges, there is a large tensile area at the top surface near the support location which is directly exposed to the chloride environment and remains in tension under service load (dead load and live load). Structural engineers and asset managers tried to apply the easy-to-measure damage to evaluate the capacity condition of the RC structures suffering corrosion damage. Experimental tests are usually conducted to understand the corrosion behavior of RC structures to assess the corrosion crack patterns and develop the empirical relationship between corrosion crack width (easy-to-measure damage) with corrosion-induced metal loss and from which to develop prediction method to evaluate the structure capacity loss using corrosion-induced crack width as an indicator.
Service cracks in loaded reinforced concrete elements are a common occurrence due to the low tensile strength of concrete. Therefore, most RC structural components have surface cracks under service loading. Nevertheless, currently there is lack of data on corrosion-induced cracking behavior of RC slabs under realistic bridge deck environment where RC slabs are corroding under pre-existing crack conditions while subjected to sustained loading.

In order to evaluate the corrosion behavior of RC slabs in laboratory within the restriction of shortened experimental time, accelerated corrosion tests have been used [8, 47, 49, 58-60, 62, 112-115]. The well designed accelerated corrosion test apparatus should be applying sustained loading while allowing observation of corrosion crack width and crack patterns during corrosion testing. The mass loss can be measured after the accelerated corrosion duration by removing the cover concrete. From the measurement of crack width and mass loss at the end of corrosion duration, the empirical relationships between crack width and mass loss can be developed.

In previous chapter, the main focus was on assessing the effect of pre-existing cracks and sustained loading on the spatial distribution of crack width and mass loss along the longitudinal direction of the test specimens [104]. On the other hand, the primary objective of this chapter is to provide relevant experimental data and establish useful empirical relationship between crack width and mass loss. The observation and empirical relationship between corrosion crack width and mass loss can be useful for field inspection of bridge deck condition considering corrosion-induced damage. Furthermore, the interaction mechanisms of transverse and longitudinal cracks observed in this experiment program are discussed in detail.
4.2 Background

As mentioned in Section 3.1, the most existing experiment data and analytical models were obtained from RC specimens that were corroded in the condition without pre-existing cracks and sustained loading. In fact, we could not identify previous experimental work that includes the test condition of the pre-existing cracks with sustained loading. Among those reported on the effects of sustained loading on corrosion rate, there is a contradictory observations. For example, in references [61, 62], a higher corrosion rate for RC beams under a sustained loading is observed when compared to the RC beams without sustained loading. On the other hand, in reference [90], it shown that the sustained loading did not exert an effect on the metal loss in the corroded RC beams. The experimental results reported in this paper could provide additional insight on the effects of sustained loading on the metal loss caused by corrosion in RC slabs.

The loading system used by previous researchers did not allow for an easy continuous monitoring of the crack widths during accelerated corrosion duration reported by [54, 61]. The study by [115] proposed that there are several advantages in using stain gages to monitor the tensile strain during accelerated corrosion duration and convert the measured strain to the crack width. However, all these advantages are based on the assumption that strain gages are strategically located in the cracking areas. Actually, the locations of cracks are randomly appearing and hard to predict. The sequence and propagation of the corrosion cracks at multiple corrosion levels and different experimental conditions was as such unknown. It is worth noting that some studies [90, 115] monitored the corrosion crack expansion with RC beams corroded under sustained loading. However, there is no past study to highlight the effects of pre-existing transverse cracks (normal to the longitudinal
tensile reinforcements) condition of the RC specimens on the corrosion crack width growth rate.

In the monitoring of the service life of corrosion-affected RC structures, easy-to-measure damage is needed to directly relate the measured damage to the loss in the section of steel, and eventually to predict or estimate the residual capacity of the structures with these visually measured damage [5, 6]. In an attempt to identify these easy-to-measure damage, numerous experimental studies on the behavior of RC structures with corroding reinforcements were carried out [2, 5, 6, 8, 24, 44, 48-50, 58, 62, 63, 65, 84, 85, 87, 90, 91, 96, 112, 116-122]. For example, Alonso, et al. (1998) [48] involved testing RC specimens of dimensions $150 \times 150 \times 380$ mm with the corrosion damage level is around 20% for all tested specimens. From their experimental data, they developed the empirical relationship is that a 1% metal loss corresponds to a maximum crack width of 0.076 mm. Torres-Acosta A.A et al. (2007) [6] involved testing RC specimens of dimensions $100 \times 150 \times 1500$ mm with the corrosion damage level between 5% to 10%. They developed the empirical relationship is that 1% metal loss corresponds to a maximum crack width of 0.03 mm. Kaersley E.P et al. (2014) [67] involved testing RC slabs of dimensions $350 \times 150 \times 1700$ mm and the average of corrosion level was at 15%. However, they did not proposal any empirical relationship between metal loss and corrosion crack width. Based on the reported experiment data from [67]; however, we can observe that a 1% metal loss corresponds to 0.085 mm crack width.

The experimental work presented in this paper adopted the RC slab specimens without shear reinforcements. The loading system and experimental setup used in this test program allows the crack width be easily monitored during the accelerated corrosion
duration. Different corrosion levels and experimental conditions are considered in this program to allow for comparisons of effects of that conditions as well as for developing the empirical relationship between corrosion crack width and metal loss for different test conditions.

4.3 Experimental program

The dimension of the slab specimens is same as mentioned in Section 3.3. The accelerated corrosion was same as mentioned in Section 3.6. In this chapter, the test was conducted so that various corrosion levels (1%, 5%, 10% and 20%) based on Faraday’s law can be achieved in 21 days. Furthermore, all the test specimens were subjected to two days of wetting period and one day of drying period to simulate corrosive environment due to application of de-icing salt. Only 60% (service condition) of the ultima capacity of the un-corroded specimens was chosen as sustained loading level in this chapter.

4.3.1 Reinforcement configuration and material properties

Each slab specimen was reinforced with two #4 deformed bars with an area of 645 mm$^2$ (0.2 in$^2$) and diameter 12.7 mm (0.5 in) on the tension side of the specimens. Because the slab components used in bridges do not have shear reinforcement, the test specimens in this experiment also do not contain any shear reinforcements, as shown in Figure 3–2. The materials used to cast test specimens have been discussed in Section 3.3. In this chapter, the total forty-one slab specimens were used and cylinders were also cast to determine the compressive strength of concrete. The measured values, taken as the average of 17 cylinder strengths (all cylinders are cured for at least 28 days but tested on different days), ranging from a minimum of 38 Mpa (5484 psi) to a maximum of 58 Mpa (8266 psi) with a standard deviation of 5 Mpa.
4.3.2 Test program

The test program consists of four groups based on the condition of the existence of pre-existing cracks prior to commencing corrosion test and if sustained loading is applied during corrosion process. As summarized in Table 4-1, four corrosion levels are achieved in 21 days of corrosion period: Set A with 1% corrosion level, Set B with 5% corrosion level, Set C with 10% corrosion level, and Set D with 20% corrosion level. Each set also has different groups according to different experiment conditions in terms of pre-existing cracks and sustained loading. Specimens in Group 1 are those subjected to corrosion without pre-existing cracks and sustained loading. Specimens in Group 2 are those subjected to corrosion with pre-existing cracks. Specimens in Group 3 are those with both pre-existing cracks and sustained loading.

4.4 Procedure for introducing cracks prior to accelerated corrosion

Same as previous Chapter 3, we used both ACI 318R-95 [108] and Oh and Kang’s [109] equations, as given in Equations (3.1) and (3.2), to determine the load used to produce pre-existing cracks in the specimens prior to the initiation of the corrosion process.
Table 4-1 Details of test specimens

<table>
<thead>
<tr>
<th>Set Label</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Group 1</td>
</tr>
<tr>
<td>Set A</td>
<td>3</td>
</tr>
<tr>
<td>Set B</td>
<td>2</td>
</tr>
<tr>
<td>Set C</td>
<td>2</td>
</tr>
<tr>
<td>Set D</td>
<td>5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>No pre-existing cracks or sustained loading applied during accelerated corrosion duration</th>
<th>Pre-existing cracks before applying accelerated corrosion current</th>
<th>Pre-existing cracks and sustained loading during accelerated corrosion duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>No pre-existing cracks or sustained loading applied during accelerated corrosion duration</td>
<td>Pre-existing cracks before applying accelerated corrosion current</td>
<td>Pre-existing cracks and sustained loading during accelerated corrosion duration</td>
</tr>
</tbody>
</table>

Note: All specimens subjected to two days wetting and one day drying during the accelerated corrosion duration.

4.5 Accelerated corrosion procedure

Because the nature corrosion test is time consume, most corrosion experimental researchers will apply accelerated corrosion test to get the results.

4.5.1 Theoretical determination of electrical current

Same as in Chapter 3, Faraday’s law, given in Equation (3.3), was used to calculate the desired corrosion level (theoretical metal loss of the reinforcements embedded in each specimen). From which, the direct electrical current value to achieve the desired corrosion level for a given duration time can be calculated.

In this chapter, the total corrosion time was still 21 days, during which we only introduce cycles of two days of wetting and one day of drying. Thus, only two weeks (1,209,600 seconds) wetting period were considered in Equation (3.3) as corrosion time to
calculate the required electrical current value for each desired corrosion level. The applied
electrical current value used to reach the desired corrosion levels (1%, 5%, 10% and 20%) in accelerated corrosion duration is shown in Table 4-2. Since the total accelerated corrosion duration of all specimens is same, therefore, specimens with higher desired corrosion level would be subjected to higher electrical current value.

Table 4-2 Accelerated current value for desired corrosion levels

<table>
<thead>
<tr>
<th>Set Label</th>
<th>Desired corrosion level</th>
<th>Corrosion length</th>
<th>Mass loss</th>
<th>Current value (amperes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set A</td>
<td>1%</td>
<td>1,118 mm (44 in)</td>
<td>11 grams (0.025 lb)</td>
<td>0.03</td>
</tr>
<tr>
<td>Set B</td>
<td>5%</td>
<td>1,118 mm (44 in)</td>
<td>56 grams (0.122 lb)</td>
<td>0.016</td>
</tr>
<tr>
<td>Set C</td>
<td>10%</td>
<td>1,118 mm (44 in)</td>
<td>111 grams (0.245 lb)</td>
<td>0.32</td>
</tr>
<tr>
<td>Set D</td>
<td>20%</td>
<td>1,118 mm (44 in)</td>
<td>222 grams (0.49 lb)</td>
<td>0.63</td>
</tr>
</tbody>
</table>

Note: The mass loss is represented by the percentage of the original steel weight.

4.5.2 Corrosion test setup for different groups

A schematic diagram of the test setup is shown Figure 3–5. Figure 3–6 shows photos of experimental setup for specimens without sustained loading and subjected to a sustained loading during accelerated corrosion duration. The sustained loading will be applied from the bottom surface of the specimens to upward. The tank fixed on the tensile surface of the specimen is used to contain NaCl solution.

4.6 Test results and discussion

After the experimental work, the entire test data were collected and analyzed. Based on collected data, several important results can be summarized.
4.6.1 Corrosion crack patterns

In this chapter, RC slab specimens exhibited different crack patterns on the tensile and compression surface for all specimens. Because the sustained loading is applied from bottom in upward direction, the bottom surface of the specimen is a compression surface. For discussion purpose, we designate the different corrosion-induced cracks patterns into three types. Type I represents a condition where longitudinal cracks occur at the location of the corroded main tensile reinforcement on the extreme tensile face of the specimen. Figure 4–1a shows a photo of such longitudinal crack. Type II represents the combination of the transverse cracks and longitudinal cracks, as shown in the photo in Figure 4–1b. Type III represents the longitudinal cracks detected on the compression surface of the slab during the corrosion process. Figure 4–1c and d show the photos of such longitudinal cracks.

Specimens in Group 1 (i.e., without pre-existing cracks and sustained loading) for all sets (i.e. different corrosion level) only had Type I corrosion cracks. Specimens in Group 2 (i.e., with pre-existing cracks) and Group 3 (i.e., with both pre-existing cracks and sustained loading) of all sets had shown Type II corrosion cracks. Type III corrosion cracks (i.e., the longitudinal cracks on compression surface of the specimens) detected in Group 1 (i.e., without pre-existing cracks and sustained loading) and Group 2 (i.e., with pre-existing cracks) were penetrated from the side to the middle of the span, as shown in Figure 4–1c. The longitudinal cracks on compression surface of the specimens in Group 3 (i.e., with pre-existing cracks and sustained loading) of all sets were detected from the
Figure 4–1 Typical corrosion crack patterns

middle of the span to the side as the corrosion progresses, as shown in Figure 4–1d. The cracks on the compression surface were defected much later than the longitudinal cracks on the tensile surface of the specimen.

Type I crack found in this program had also been reported elsewhere [44, 67, 90, 115, 121]. However, three new findings on corrosion pattern are obtained in the current study: i) Type II cracks were documented on the extreme tensile surface especially at the pre-existing cracks location; ii) Type III cracks were detected on the compression surface of the tested specimens; iii) There is no side face cracking observed in this program. This could be due to the test specimens are RC slab without shear reinforcement compare to the RC beams specimens used by others [44, 90, 115, 121]. Reference [67] used RC slabs as their experimental specimen, but without considering the pre-existing cracks and sustained

67
loading conditions during their corrosion process. Therefore, only Type I crack was found in their tests.

The transverse cracks in Type II corrosion cracks can only be observed from specimens in Group 2 (i.e., with pre-existing cracks) and Group 3 (i.e., with both pre-existing cracks and sustained loading). The transverse cracks that appeared in the exposure region were possibly caused by reinforcement corrosion or sustained loading or a combination of the two effects. Due to the randomly appearing of the transverse cracks under the loading and corrosion conditions, only the maximum transverse crack width of each test specimen was recorded. Two representative transverse crack growth process results - Set A (1% corrosion level) and Set D (20% corrosion level) - are shown in Figure 4–2.

![Figure 4–2](image)

Figure 4–2 Transverse crack width growth rates observed in Type II corrosion cracks

The transverse crack width observed from specimens in Group 3 (i.e., with both pre-existing cracks and sustained loading) was much larger than that observed from specimens in Group 2 (i.e., with pre-existing cracks). It appears that the sustained loading has significant effect on the growth of the transverse crack. The specimens in Group 2 (i.e., with pre-existing cracks) of Set A (1% corrosion level) and Set D (20% corrosion level)
almost had similar transverse crack width (0.1 mm), so the corrosion level has less effect on the transverse crack growth, if the specimens corroded without being subjected to sustained loading. The transverse crack width observed from specimens in Group 3 (i.e., with both pre-existing cracks and sustained loading) of Set D (20% corrosion level) is much larger (0.76 mm) than that (0.35 mm) in Set A (1% corrosion level), which may be due to the fact that the higher corrosion level induce larger steel section loss and stuffiness loss of reinforcements. In fact, the deflection of specimens in Set D (20% corrosion level) was larger than that of specimens in Set A (1% corrosion level) when the same amount sustained loading applied to both test specimens.

4.6.2 Relation between crack pattern and the crack growth rate

The detailed information of Type I and II corrosion cracks on the tensile surface (top surface) of the tested specimens was recorded with micro-crack measurement device during the drying days. However, Type III corrosion crack on the compression surface (bottom surface) of the specimens was difficult to record during the accelerated corrosion duration due to difficulty in access. Since the applied electrical current is constant, which means the corrosion rate and metal loss can be assumed as constant according to Faraday’s Law. Therefore, the average longitudinal corrosion crack growth rate (mm/day) can be calculated as follows:

\[
\text{Average corrosion crack growth rate} = \frac{\text{Corrosion crack width at final stage}}{\text{Total corrosion time}}
\]  \hspace{1cm} (4.1)

where “Corrosion crack width at final stage” is the average value calculated from the measured longitudinal corrosion crack width of all tested specimens at the last drying day.
of the accelerated corrosion duration; “Total corrosion time” is the total accelerated corrosion time used in this program 21 days.

All the calculated average corrosion crack growth rates are listed in Table 4-3. Based on all the values listed in Table 4-3, two main characteristics can be observed. For Set A (1% corrosion level) and Set B (5% corrosion level), the specimens in Group 1 corroded without pre-existing cracks and sustained loading conditions have lowest average corrosion crack growth rate. While the specimens in Group 3 corroded with both pre-existing cracks and sustained loading conditions have highest average corrosion crack growth rate. For Set C (10% corrosion level) and Set D (20% corrosion level), the specimens in Group 3 corroded with pre-existing cracks and sustained loading conditions have the lowest average corrosion crack growth rate, while the specimens in Group 1 corroded without pre-existing cracks and sustained loading conditions have the highest average corrosion crack growth rate.

Table 4-3 Average longitudinal corrosion crack growth rate (mm/day)

<table>
<thead>
<tr>
<th>Set</th>
<th>Group 1</th>
<th>Group 2</th>
<th>Group 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.0007</td>
<td>0.002</td>
<td>0.004</td>
</tr>
<tr>
<td>B</td>
<td>0.004</td>
<td>0.006</td>
<td>0.009</td>
</tr>
<tr>
<td>C</td>
<td>0.02</td>
<td>0.016</td>
<td>0.013</td>
</tr>
<tr>
<td>D</td>
<td>0.05</td>
<td>0.03</td>
<td>0.015</td>
</tr>
</tbody>
</table>

The longitudinal corrosion crack growth results at different corrosion time are shown in Figure 4–3. Based on the experimental results, we can conclude that there are two crack mechanisms interacting during corrosion process:

1) The transverse cracks could allow the chloride ion from the salt solution diffuse into concrete and reach the surface of reinforcements faster, which can speed up corrosion
process and induce more severe corrosion damage. Therefore, one can see that in Set A (1% corrosion level) and Set B (5% corrosion level) and the initial stage of Set C (10% corrosion level) and Set D (20% corrosion level), the longitudinal crack width growth rate of Group 2 (i.e., with pre-existing cracks) and Group 3 (i.e., with both pre-existing cracks and sustained loading) is larger than that in Group 1 (i.e., without pre-existing cracks and sustained loading, thus without transverse cracking).

2) The transverse cracks also provide the space for corrosion products to fill without inducing pressure. The longitudinal cracks were induced by the hoop tensile stress due to the expansion of the corrosion products. Therefore, the transverse crack could reduce the hoop tensile stress because the space for corrosion products to occupy. For specimens in Group 1, where no transverse crack develops, the corrosion product can only fill the space in longitudinal crack, thus causing a larger hoop stress at certain stage of corrosion process. For Set C (10% corrosion level) and Set D (20% corrosion level), the longitudinal crack width growth rate of Group 2 (i.e., with pre-existing cracks) and Group 3 (i.e. with both pre-existing cracks and sustained loading) is smaller than Group 1 (without pre-existing cracks and sustained loading) at later stage of corrosion due to the effect of the transverse crack explained.
4.6.3 Relation between crack widths and metal loss

In order to develop the empirical relationship between corrosion crack width and metal loss, the corrosion exposure area of all the specimens were divided into 10 segments (identified with number from 1 to 10) with 50.7mm (2 inch) length for each segment, as shown in Figure 4-4. The longitudinal crack width of each segment can be recorded during the corrosion process. The corrosion product can be cleaned by a special acid solution after the concrete cover is removed. All the measured the longitudinal crack width and metal
loss values for each segment at the end of the corrosion duration (after 21 days) are listed in Table 4-4. Because there is more than one specimen (Table 4-1) used in this experimental program for each group with different experimental conditions of multiple desired corrosion levels, Table 4-4 lists the average value for all tested specimens within each designated test group and set.

1) The longitudinal crack width and metal loss are almost keep constant for all segments in specimens of Group 1 (without pre-existing cracks and sustained loading) for all sets in this experiment program.

2) The middle-span zone (segments 4-7) of specimens in Group 2 (with pre-existing cracks) and Group 3 (with pre-existing cracks and sustained loading) exhibit to larger longitudinal crack width and metal loss than those in the outer zone (segments 1-3 and segments 8-10) for all sets in this experiment program, as shaded data in Table 4-4.

Figure 4–4 Division of the area of the slab exposed to the salt solution and corresponding reinforcement coupons

The relationship between corrosion crack width and metal loss can be calculated by dividing the longitudinal crack width by the corresponding metal loss shown in Table 4-4.
The empirical value of the longitudinal crack width induced by a 1% metal loss is summarized in Table 4-5. As shown can be observed, Group 1 (i.e., without pre-existing cracks and sustained loading) and Group 2 (i.e., with pre-existing cracks) exhibit different crack width at 1% metal loss for different corrosion levels. However, the crack width at 1% metal loss for group 3 of all corrosion levels remains the same. The empirical relationships developed from specimens corroded without pre-existing cracks and sustained loading can be comparable with this study’s results, as mentioned in introduction section.

As listed in Table 4-5, results developed from specimens in Group 3 (i.e., with pre-existing cracks and sustained loading) have smaller value than that developed from specimens in Group 1 (i.e. without pre-existing cracks and sustained loading) for high corrosion level (i.e., 10% or 20% corrosion level). While the experimental conditions of Group 3 are more representative compare with realistic service conditions of RC structures. If the empirical relationship developed from Group 1 (i.e., without pre-existing cracks and sustained loading) used to guide the structural engineers and asset managers to predict the metal loss based on measured crack width, it would induce underestimate results for RC structures suffered serious corrosion damage.

4.7 Conclusions

1) In this chapter, RC slab specimens corroded under different experiment conditions have shown different crack patterns. Type I is longitudinal crack along the tensile side of the specimen. Type II is a combination of transverse crack and longitudinal crack on the tensile surface of the specimen. Type III is longitudinal crack on the compression surface of the specimen. Different crack patterns developed due to combined interaction of the pre-existing cracks and applied sustained loading during corrosion.
2) The transverse cracks induced by pre-existing cracks and sustained loading conditions have different mechanism effect on the longitudinal crack growth rate. At corrosion initial stage, the transverse cracks sped up the corrosion rate and caused faster longitudinal corrosion crack growth rate. While the larger transverse cracks can also provide spaces and paths for corrosion product to fill and come out after certain corrosion time, which can release the hoop tensile stress to reduce the longitudinal corrosion growth rate.

3) The empirical relationship between metal loss and crack width developed from specimens corroded with pre-existing cracks and sustained loading conditions for all corrosion levels remain the same value. It was therefore recommend that 1% mental loss corresponds to a maximum crack width 0.03 mm (0.001 in), which can be referred in practical.
Table 4-4 Metal loss and longitudinal crack width at the end of corrosion test

<table>
<thead>
<tr>
<th>Segment</th>
<th>Set A</th>
<th>Set B</th>
<th>Set C</th>
<th>Set D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Metal loss (%)</td>
<td>Crack width (mm)</td>
<td>Metal loss (%)</td>
<td>Crack width (mm)</td>
</tr>
<tr>
<td>Group 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.69</td>
<td>0.03</td>
<td>5.30</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>1.26</td>
<td>0.03</td>
<td>5.30</td>
<td>0.08</td>
</tr>
<tr>
<td>3</td>
<td>1.72</td>
<td>0.03</td>
<td>5.74</td>
<td>0.08</td>
</tr>
<tr>
<td>4</td>
<td>1.23</td>
<td>0.03</td>
<td>5.99</td>
<td>0.08</td>
</tr>
<tr>
<td>5</td>
<td>0.90</td>
<td>0.03</td>
<td>5.54</td>
<td>0.1</td>
</tr>
<tr>
<td>6</td>
<td>1.33</td>
<td>0.03</td>
<td>5.05</td>
<td>0.1</td>
</tr>
<tr>
<td>7</td>
<td>1.03</td>
<td>0.03</td>
<td>5.35</td>
<td>0.08</td>
</tr>
<tr>
<td>8</td>
<td>1.46</td>
<td>0.03</td>
<td>5.25</td>
<td>0.1</td>
</tr>
<tr>
<td>9</td>
<td>1.53</td>
<td>0.03</td>
<td>4.71</td>
<td>0.08</td>
</tr>
<tr>
<td>10</td>
<td>1.00</td>
<td>0.03</td>
<td>5.35</td>
<td>0.08</td>
</tr>
<tr>
<td>Group 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.05</td>
<td>0.03</td>
<td>4.31</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>2.83</td>
<td>0.03</td>
<td>4.21</td>
<td>0.15</td>
</tr>
<tr>
<td>3</td>
<td>2.93</td>
<td>0.03</td>
<td>4.61</td>
<td>0.17</td>
</tr>
<tr>
<td>4</td>
<td>2.78</td>
<td>0.05</td>
<td>6.14</td>
<td>0.17</td>
</tr>
<tr>
<td>5</td>
<td>4.31</td>
<td>0.05</td>
<td>9.55</td>
<td>0.25</td>
</tr>
<tr>
<td>6</td>
<td>3.42</td>
<td>0.05</td>
<td>8.61</td>
<td>0.23</td>
</tr>
<tr>
<td>7</td>
<td>2.43</td>
<td>0.03</td>
<td>7.18</td>
<td>0.18</td>
</tr>
<tr>
<td>8</td>
<td>1.54</td>
<td>0.03</td>
<td>5.40</td>
<td>0.15</td>
</tr>
<tr>
<td>9</td>
<td>1.84</td>
<td>0.03</td>
<td>5.74</td>
<td>0.13</td>
</tr>
<tr>
<td>10</td>
<td>1.44</td>
<td>0.03</td>
<td>4.16</td>
<td>0.10</td>
</tr>
<tr>
<td>Group 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.95</td>
<td>0.05</td>
<td>7.08</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>1.69</td>
<td>0.08</td>
<td>7.77</td>
<td>0.15</td>
</tr>
<tr>
<td>3</td>
<td>2.73</td>
<td>0.10</td>
<td>8.06</td>
<td>0.18</td>
</tr>
<tr>
<td>4</td>
<td>4.56</td>
<td>0.10</td>
<td>8.71</td>
<td>0.23</td>
</tr>
<tr>
<td>5</td>
<td>5.10</td>
<td>0.10</td>
<td>10.88</td>
<td>0.25</td>
</tr>
<tr>
<td>6</td>
<td>6.24</td>
<td>0.10</td>
<td>11.62</td>
<td>0.25</td>
</tr>
<tr>
<td>7</td>
<td>4.36</td>
<td>0.08</td>
<td>8.46</td>
<td>0.23</td>
</tr>
<tr>
<td>8</td>
<td>2.28</td>
<td>0.08</td>
<td>8.06</td>
<td>0.15</td>
</tr>
<tr>
<td>9</td>
<td>1.69</td>
<td>0.051</td>
<td>7.52</td>
<td>0.13</td>
</tr>
<tr>
<td>10</td>
<td>0.36</td>
<td>0.051</td>
<td>9.55</td>
<td>0.08</td>
</tr>
<tr>
<td>Segment</td>
<td>Set A</td>
<td>Set B</td>
<td>Set C</td>
<td>Set D</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
</tr>
<tr>
<td></td>
<td>Crack width (mm) for 1% metal loss</td>
<td>Crack width (mm) for 1% metal loss</td>
<td>Crack width (mm) for 1% metal loss</td>
<td>Crack width (mm) for 1% metal loss</td>
</tr>
<tr>
<td>Group 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.008</td>
<td>0.01</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>3</td>
<td>0.008</td>
<td>0.01</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>4</td>
<td>0.01</td>
<td>0.01</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>5</td>
<td>0.02</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>6</td>
<td>0.01</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>7</td>
<td>0.02</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>8</td>
<td>0.01</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>9</td>
<td>0.01</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>10</td>
<td>0.02</td>
<td>0.02</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>Group 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.02</td>
<td>0.03</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>3</td>
<td>0.02</td>
<td>0.05</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>4</td>
<td>0.02</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>6</td>
<td>0.01</td>
<td>0.03</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>7</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>8</td>
<td>0.02</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>9</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>10</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>Group 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.05</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>3</td>
<td>0.05</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>4</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>5</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>6</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>7</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>8</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>9</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>10</td>
<td>0.04</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
</tbody>
</table>
CHAPTER V

CORROSION BEHAVIOR OF EPOXY-COATED REINFORCEMENTS AND POLYPROPYLENE FIBERS USED IN REINFORCED CONCRETE STRUCTURES

5.1 Introduction

Reinforced concrete is used in various infrastructures in corrosive environment (e.g. marine environment or de-icing salt environment). There is certain critical value for the pH and Cl⁻ concentration in concrete for the corrosion process of reinforcing steel to initiate. As mentioned in Chapter II, there are two general methods used widely in practice to prevent corrosion damage of reinforcing bars in RC structures – i) using various kinds of coated reinforcements and ii) adding fibers into plain concrete.

Epoxy coatings have been widely applied to surfaces of reinforcements to protect them since mid-1970s [97]. The usage of epoxy-coated reinforcements is thought of as one of the most effective methods. Such protection primarily depends on the integrity of the coatings. However, the coating surface may sustain damages during the handling of the epoxy-coated reinforcements. All the previous studies did not consider surface defects condition of epoxy-coated reinforcements before embedded into concrete. The effectiveness of epoxy-coated reinforcement with surface defects for corrosion prevention has not been fully investigated in the past.
Except using coated reinforcements, adding fibers into plain concrete is another way to improve the corrosion resistance of RC structures by reducing the crack width growing of cover concrete. As introduced in Literature Review (Chapter II) many studies have been conducted to characterize in the mechanical properties of reinforced concrete with fiber additives. According to these studies [75-83], most attention has been focused on polypropylene (PP) fibers because of its low cost, outstanding toughness and enhanced shrinkage cracking resistance of the concrete. Sanjuan et al. [98] added PP fibers into pure concrete cylinder specimens in carbonation corrosive environment to control the cracking and improve the durability of concrete. Low amounts of PP fibers were used in their study with a finding of beneficial effect of fiber addition on slowing down the corrosion rate. However, the corrosion resistance effectiveness of PP fibers in reinforced concrete structures under chloride corrosive environment has not been fully investigated in the past.

This chapter is to present the findings of accelerated corrosion tests on RC slabs with epoxy-coated reinforcements and PP fibers. In particular, the effect of surface defects of epoxy-coated reinforcements on corrosion resistance, when compared with specimens casted with black reinforcements, will be discussed. The corrosion mitigation effect on corrosion crack width expansion and metal loss of PP fibers added into concrete, when compared with test specimens casted without PP fibers, will also be presented. In this study, accelerated corrosion tests were carried out by means of passing the electrical current to the specimens and exposing a limited area of specimens to salt solutions. The electrical current values used to achieve 10%, 20% and 40% desired corrosion levels in a defined time period were determined using Faraday’s law. In the current experimental study,
emphasis was placed on specimens with the pre-existing cracks and sustained loading to more realistically simulate actual filed conditions [63, 84].

5.2 Methodology

After the introduction, the methodology should be designed before the experimental work. The detail of the methodology will be introduced in this section.

5.2.1 Material and specimen preparation

We purposely introduce surface defects to the epoxy-coated reinforcements. The percent of damage area is 5% of the total area, as show in Figure 5–2. Accelerated corrosion was induced by passing the required amount of electrical current provided by external DC power supply, as introduced in previous chapters. In this chapter, all the specimens were subject to two days wetting period and one day of drying cycles.

The quantitative of materials used to mix the concrete in the test program are summarized in Table 5-1. The physical characteristics of polypropylene fibers are listed in Table 5-2. The slab specimens dimension is same as the specimens used in Chapter 3 and Chapter 4. The details of specimen dimension and the connection with the external power are illustrated in Figure 3–2 and Figure 3–5.

![Figure 5–1 Sketches of surface defects on epoxy-coated reinforcement](image-url)
Table 5-1 The raw materials used in presented mixture design

<table>
<thead>
<tr>
<th>Material</th>
<th>$kg/m^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>148</td>
</tr>
<tr>
<td>Cement</td>
<td>335</td>
</tr>
<tr>
<td>Sand</td>
<td>777</td>
</tr>
<tr>
<td>Aggregate</td>
<td>1014</td>
</tr>
<tr>
<td>$w/c$</td>
<td>0.44</td>
</tr>
</tbody>
</table>

Table 5-2 Physical characteristics of polypropylene fibers [123]

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>$59 mg/cm^3$</td>
</tr>
<tr>
<td>Diameter</td>
<td>22 µm</td>
</tr>
<tr>
<td>Width crossing</td>
<td>Circular</td>
</tr>
<tr>
<td>Melting point</td>
<td>160 – 170 °C</td>
</tr>
<tr>
<td>Water absorption</td>
<td>0</td>
</tr>
<tr>
<td>Torsion resistibility</td>
<td>400–350 Mpa</td>
</tr>
</tbody>
</table>

In this experimental program, black reinforcements and epoxy-coated reinforcements were embedded in different RC test specimens, while the epoxy-coated reinforcements were prepared with surface defects. Two fiber quantitative ratios of $4.5 \ kg/m^3$ and $6 \ kg/m^3$, respectively, were used when fiber reinforced concrete specimens were
prepared. Furthermore, the test specimens were subjected to different experimental conditions: i) with pre-existing cracks and sustained loading, ii) without pre-existing cracks and sustained loading. The sustained loading level used in this chapter was 60% of the ultimate load carrying capacity of the specimens with un-corroded reinforcements.

Detailed information of the test program is presented in Table 5-3. Test specimen designation in Table 5-3 is as follows: the first letter “B” indicates black reinforcement and “E” indicates epoxy-coated reinforcement; the second letter “P” indicates plain concrete and “PP” indicates concrete with polypropylene fibers; the numerical value in the parenthesis indicates the quantitative ratios of polypropylene fibers mixed into the plain concrete; the numerical value -“10”, “20” and “40”- follows the parenthesis indicates the theoretical corrosion level in terms of metal loss as the percentage of original weight; the last numerical value -“0” and “60”- indicates the sustained loading in terms of percentage of the ultimate capacity in 3-point bending test of slab specimens with un-corroded reinforcements. For example, test specimen “B-P-20+0” indicates during accelerated corrosion test the RC specimen with black reinforcements without PP fiber additives at 20% desired corrosion level subjected to zero sustained loading. Test specimen “E-P-20+60” indicates during accelerated corrosion test the RC specimen with epoxy-coated reinforcements without PP fiber additives at 20% desired corrosion level subjected to 60% sustained loading level. It should be pointed out that the PP fibers added into plain concrete have good effect on increasing the tensile strength of concrete structures to control the crack width. Therefore, in order to simulate the severe corrosive environment, some test specimens with PP fibers were subjected to high electrical current to reach 40% desired
corrosion level. Figure 3–6 shows typical corrosion test setup and the detailed loading systems that were used during the corrosion process.

Table 5-3 Details of test specimens

<table>
<thead>
<tr>
<th>RC test specimen</th>
<th>Specimen identification</th>
<th>Number of the specimens</th>
<th>Wetting-drying cycles and total accelerated corrosion duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-P-20+0</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-P-20+0</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-PP(4.5)-20+0</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-PP(6)-20+0</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Group 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-P-10+60</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-P-20+60</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-P-10+60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-P-20+60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-PP(4.5)-20+60</td>
<td>2</td>
<td>Two days wetting and one day drying. Total corrosion duration is 21 days for each test specimen</td>
</tr>
<tr>
<td></td>
<td>B-PP(6)-20+60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-PP(4.5)-20+60</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-PP(6)-20+60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Group 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-PP(4.5)-40+60</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B-PP(6)-40+60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-PP(4.5)-40+60</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

5.2.2 Procedure for introducing pre-existing cracks on slab specimens

Same as the Chapter 3 and Chapter 4, to estimate the loading required to produce visible cracking on specimens we use both ACI 318R-95 [108] and Oh and Kang’s [109] equations, which provides the load needed to the first crack in the test specimen.
Usually, the smallest width of a visible crack under normal conditions is approximately 0.13 mm (0.005 in). The theoretical value according to ACI 318R-95 is around 13.3 kN (3000 lb). The load applied to the slabs in the test program to create the first crack was between 13.3 and 17.8 kN (3000 – 4000 lb) based on the on-site observation of first appearance of the visible crack.

5.2.3 Theoretical determination of electrical current to produce desired corrosion level

In this chapter, Faraday’s law, given in Equation (3.3), was still used to determine the theoretical electrical current necessary to produce the metal loss of reinforcements for a prescribed accelerated corrosion test duration.

In this chapter, the total accelerated corrosion duration of two weeks (1,209,600 seconds) was used. Three desired metal loss levels were selected for representing different severity of corrosion: 10%, 20% and 40%. The total length of the reinforcing bar in the RC slab specimens exposed to the salt solution was 1,118 mm (44 in). The applied current value determined from the Faraday’s law is shown in Table 5-4.

<table>
<thead>
<tr>
<th>Desired corrosion level</th>
<th>Metal loss</th>
<th>Current value (amperes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>111 grams (0.245 lb)</td>
<td>0.32</td>
</tr>
<tr>
<td>20%</td>
<td>222 grams (0.49 lb)</td>
<td>0.63</td>
</tr>
<tr>
<td>40%</td>
<td>444 grams (0.49 lb)</td>
<td>1.26</td>
</tr>
</tbody>
</table>

Note: The metal loss is represented by the percentage of the original rebar weight.

5.3 Results and discussion

After the experimental work, the entire test data were collected and analyzed. Based on collected data, several important results can be summarized.
5.3.1 Corrosion crack width

The corrosion products of steel have a larger volume than the original steel, and as a result, the pressure is applied on the cover concrete, which eventually leads to cracking of surrounding concrete. The expansion pressure keeps increasing within processing of corrosion, which would induce cracking on the surface of specimens exposed to salt solution when the stress exceed the tensile strength of the concrete.

The corrosion crack width expansion results for specimens containing black and epoxy-coated reinforcements in plain concrete as related to the corrosion duration in days for various test specimens are shown Figure 5–3. It can be seen that the crack expansion rate at beginning stage of corrosion for the specimens with epoxy-coated reinforcements is slower than that for the specimens with black reinforcements. However, as corrosion duration increasing, the crack expansion rate for specimens with epoxy-coated reinforcements exhibits same or even higher rate than that for the specimens with black reinforcements. This phenomenon can be explained by the defect introduced to epoxy coating. The defect allows chloride ions to penetrate through coating and reach the surface of rebar which then promote accelerate crack opening due to de-bonding of coating. In fact, after removing the outside concrete of specimens with epoxy-coated reinforcements after the corrosion test in this program, we can visibly see the de-bonding of coating occurrence, as shown in Figure 5–4. The epoxy coating of the reinforcements became brittle and can be peeled off from the inside steel bars after corrosion test. For an easier comparison, Table 5-5 lists the average crack expansion rate calculated as the final corrosion crack width divided by total corrosion test duration (21 days). It can be seen that E-P-20+0 has an average crack expansion rate of 0.055 mm/day, which is almost the same as the average
crack expansion rate of 0.051 mm/day for B-P-20+0. B-P-10+60 and E-P-10+60 have the same average crack expansion rate of 0.013 mm/day, while B-P-20+60 and E-P-20+60 also have similar average crack expansion rate of 0.018 mm/day and 0.022 mm/day, respectively. Thus, it seems that crack expansion rate due to corrosion is about the same for both black rebar and epoxy-coated rebar (with surface defect).

Figure 5–3 Corrosion crack width expansion for specimens without PP fibers additives with increasing corrosion level indicated by corrosion duration in days.
Figure 5–4 De-bonding of epoxy coating caused by corrosion

Table 5-5 Average crack expansion rate for all tested specimens (\textit{mm/day})

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Average crack expansion rate (\textit{mm/day})</th>
<th>Specimen designation</th>
<th>Average crack expansion rate (\textit{mm/day})</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-P-20+0</td>
<td>0.051</td>
<td>B-PP(4.5)-20+0</td>
<td>0.04</td>
</tr>
<tr>
<td>B-P-10+60</td>
<td>0.013</td>
<td>E-PP(6)-20+0</td>
<td>0.028</td>
</tr>
<tr>
<td>B-P-20+60</td>
<td>0.018</td>
<td>B-PP(4.5)-20+60</td>
<td>0.011</td>
</tr>
<tr>
<td>E-P-20+0</td>
<td>0.055</td>
<td>B-PP(6)-20+60</td>
<td>0.011</td>
</tr>
<tr>
<td>E-P-10+60</td>
<td>0.013</td>
<td>E-PP(4.5)-20+60</td>
<td>0.011</td>
</tr>
<tr>
<td>E-P-20+60</td>
<td>0.022</td>
<td>E-PP(6)-20+60</td>
<td>0.010</td>
</tr>
</tbody>
</table>

High corrosion design level (simulate severe corrosive environment)

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Average crack expansion rate (\textit{mm/day})</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-PP(4.5)-40+60</td>
<td>0.015</td>
</tr>
<tr>
<td>B-PP(6)-40+60</td>
<td>0.013</td>
</tr>
</tbody>
</table>

87
Figure 5–5 shows the experimental results of the specimens with PP fibers compared with that of the specimens without PP fibers. As shown in Figure 5–5, the PP fibers provide significant mitigation effect on the corrosion crack expansion rate. Figure 5–5a-b compare the experimental results of corrosion crack expansion between specimens containing black reinforcements without PP fibers and that of the specimens containing black reinforcements with PP fibers. It can be seen that PP fibers can reduce crack width expansion rate for both experiment conditions (without pre-existing cracks and sustained loading and with pre-existing cracks and sustained loading). The average crack expansion rates are listed in Table 5-5. On average, the use of PP fibers (e.g., B-PP(4.5)-20+0 and B-PP(4.5)-20+60) tends to reduce crack expansion rate to about 22% (without pre-existing cracks and sustained loading) and 38% (with pre-existing cracks and sustained loading) of that from specimens subjected to same experimental conditions but without PP fibers (e.g. B-P-20+0 and B-P-20+60). Figure 5–5c compares the experimental results of corrosion crack expansion for specimens containing epoxy-coated reinforcements without PP fibers with that of the specimens containing epoxy-coated reinforcements with PP fibers. It can be seen that PP fibers have same corrosion crack mitigation effect as specimens containing black reinforcements. Based on the average crack expansion rates listed in Table 5-5, the use of PP fibers (e.g., E-PP(6)-20+0 and E-PP(4.5)-20+60) in specimens containing epoxy-coated reinforcements tends to reduce crack expansion rate to about 50% for both experimental conditions of that from specimens without PP fibers (e.g., E-P-20+0 and E-P-20+60). Figure 5–5b-c also show the experimental results of corrosion crack expansion for specimens with PP fibers at 40% corrosion level. It can be seen that PP fibers can reduce crack expansion rate for both specimens containing black reinforcements and epoxy-coated
reinforcements. The specimens with PP fibers (e.g., B-PP(4.5)-40+60) at 40% corrosion level have smaller crack expansion rate than that from specimens without PP fibers at 20% corrosion level (e.g., B-P-20+60). Figure 5–5d shows a comparison of the crack width expansion rate to be similar between two fiber contents.

![Graphs showing crack width expansion rate](image)

(a) Specimens corroded without pre-existing cracks and sustained loading (20% desired corrosion level)

(b) Specimens corroded with pre-existing cracks and sustained loading (20% and 40% desired corrosion level)

(c) Specimens containing epoxy-coated reinforcements corroded with pre-existing cracks and sustained loading (20% and 40% desired corrosion level)

(d) Specimens containing different quantitative PP fibers corroded with pre-existing cracks and sustained loading (20% desired corrosion level)

Figure 5–5 Corrosion crack width expansion rate for specimens with and without PP fibers additives with increasing corrosion level indicated by corrosion duration in days

As explained previously in Figure 3–6a, the area of the specimens exposed to salt solution was divided into 10 segments (each segment is 50.8 mm in the length direction of
the specimen) to assess the spatial distribution of crack width along the length of the slab. Figure 5–6 shows that the specimens with pre-existing cracks and subjected to sustained loading conditions exhibit spatial variations of crack width along the length of the slab, with the maximum corrosion crack width in the central segment where pre-existing crack exists. The specimens corroded without pre-existing cracks and no sustained loading exhibit the same crack width for all segments. Because the specimens containing different materials subjected to the same experimental conditions have exhibited the same crack width variation characteristics. Therefore, to avoided repetition, we only show in Figure 5–6 the representative results from specimens containing epoxy-coated reinforcements without PP fibers and specimens containing black reinforcements with PP fibers.

Figure 5–6 Representative spatial distribution of crack width along the length of the slabs

5.3.2 Mass loss of reinforcements

This section discusses the metal loss for various specimens conducted in the test program. The corroded reinforcements were cleaned by acid solution after corrosion test and cut into small coupons (each coupon is 50.8 mm in length) corresponding to the
segments division shown in Figure 3–6a. The percentage metal loss of a rebar coupon, $Q_{gi}$, was calculated from the following equation:

$$Q_{gi} = \frac{m_u - m_i}{m_u} \times 100 \quad (5.1)$$

where $m_u$ is the metal mass per segment (50.8 mm) prior to corrosion; and $m_i$ is the residual metal mass per segment (50.8 mm) after corrosion.

The average metal loss values of all rebar coupons from all test specimens (one or more test specimens subjected to same experimental conditions, as shown in Table 5-3) are summarized in Table 5-6. For a better comparison, the experimental results of the specimens subjected to the same experimental conditions are discussed below:

1) The specimens without PP fibers additives have shown similar or nearly the same average mass loss when they were subjected to same experimental conditions (e.g., same corrosion level and with or without pre-existing cracks and sustained loading). For example, B-P-20+0 and E-P-20+0 have shown same average mass loss of 14%, B-P-10+60 and E-P-10+60 have same average mass loss of 14% and the difference between B-P-20+60 and E-P-20+60 is only 1%. This comparison results demonstrate that the surface defects of epoxy-coated reinforcement have impacted significantly on corrosion resistance of epoxy-coated reinforcements in terms of metal loss.

2) The specimens containing PP fibers additives also have similar or same average mass loss (e.g., same corrosion level and with or without pre-existing cracks and sustained loading). For example, B-PP(4.5)-20+60 and E-PP(4.5)-20+60 have shown similar average mass loss of 15% and 14%, respectively, while B-PP(6)-20+60 and E-PP(6)-20+60 also have shown similar average mass loss of 14% and 13%, respectively. This
comparison results also demonstrate that the surface defects of epoxy-coated reinforcement reduce its corrosion resistance.

3) The specimens with PP fibers additives were subjected to 40% corrosion level to represent severe corrosion condition. Nevertheless, even at this high corrosion level, the experimental results of average metal loss were still similar between the specimens containing black reinforcements and epoxy-coated reinforcements with surface defects. For example, B-PP(4.5)-40+60 and E-PP(4.5)-40+60 have shown similar average mass loss of 22% and 20%, respectively, while B-PP(6)-40+60 and E-PP(6)-40+60 also have shown similar average mass loss 20% and 19%, respectively.

4) Increasing the quantity of PP fibers in plain concrete from 4.5 kg/m\(^3\) to 6 kg/m\(^3\) does not increase the corrosion resistance in terms of metal loss. For example, B-PP(4.5)-20+60 and B-PP(6)-40+60 have shown same average metal loss of 15%; while E-PP(4.5)-20+60 and E-PP(6)-40+60 also shown similar average metal loss.

Because the metal loss of each rebar coupon (50.8 mm) corresponding to each segment shown in Figure 3–6a was measured in this program, we can present the spatial distribution of metal loss along the length of the slab, as shown in Figure 5–7. Figure 5–7a-b show the metal loss distribution of the test specimens without PP fibers additives (e.g., B-P-10+60, B-P-20+60 and E-P-10+60, E-P-20+60) subjected to the pre-existing cracks and sustained loading experiment conditions. The pre-existing cracks and sustained loading conditions induced the maximum mass loss in the central segments where pre-existing crack is located. While the specimens corroded without pre-existing cracks and sustained loading conditions (e.g., B-P-20+0 and E-P-20+0) shown in Figure 5–7c have
shown the uniform mass loss distribution along the length of the slab. Figure 5–7d–e show
the spatial distributions of metal loss for the test specimens with and

Table 5-6 Average metal loss (%) for all tested specimens

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Average metal loss (%)</th>
<th>Specimen designation</th>
<th>Average metal loss (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-P-20+0</td>
<td>14</td>
<td>B-PP(4.5)-20+0</td>
<td>12</td>
</tr>
<tr>
<td>B-P-10+60</td>
<td>12</td>
<td>E-PP(6)-20+0</td>
<td>14</td>
</tr>
<tr>
<td>B-P-20+60</td>
<td>17</td>
<td>B-PP(4.5)-20+60</td>
<td>15</td>
</tr>
<tr>
<td>E-P-20+0</td>
<td>14</td>
<td>B-PP(6)-20+60</td>
<td>15</td>
</tr>
<tr>
<td>E-P-10+60</td>
<td>12</td>
<td>E-PP(4.5)-20+60</td>
<td>14</td>
</tr>
<tr>
<td>E-P-20+60</td>
<td>16</td>
<td>E-PP(6)-20+60</td>
<td>13</td>
</tr>
<tr>
<td>High corrosion design level (simulate severe corrosive environment)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-PP(4.5)-40+60</td>
<td>22</td>
<td>E-PP(4.5)-40+60</td>
<td>20</td>
</tr>
<tr>
<td>B-PP(6)-40+60</td>
<td>20</td>
<td>E-PP(6)-40+60</td>
<td>19</td>
</tr>
</tbody>
</table>

without PP fibers additives (e.g., B-P-20+0, B-PP(4.5)-20+0 and B-P-20+60, B-PP(4.5)-
20+0) subjected to the same experimental conditions. The pre-existing cracks and sustained
loading conditions also produced maximum mass loss in the central segments even with
PP fibers additives. The experimental results of the test specimens (e.g., B-PP(4.5)-20+60,
B-PP(6)-20+60 and B- PP(4.5)-40+60, B-PP(6)-40+60) with different quantity of PP fibers
additives (4.5 kg/m³ and 6 kg/m³) shown in Figure 5–7e–f indicate increasing the
quantity of PP fibers did not have significant effect on the spatial variations of metal loss
along the length of the slab.
Figure 5–7 Spatial distribution of metal loss along the length of the slabs with different materials and different experimental conditions
5.4 Conclusions

Corrosion performance of RC slab specimens with pre-existing defects on the surface of epoxy-coated reinforcements as well as specimens with PP fibers under in-service corrosive environment was experimentally investigated by means of accelerated corrosion tests. The pre-existing cracks prior to corrosion tests and sustained loading conditions during corrosion test were the main focus of test program. Further, during corrosion process, wetting and drying cycles application was incorporated to realistically simulate corrosion environment experienced by bridge deck with de-icing salt. The corrosion-induced crack width was measured during each drying day to evaluate the crack width at different stage of corrosion test. After completing accelerated corrosion test at the end of 21 days, the corroded reinforcements were further evaluated for the metal loss. The following conclusions can be drawn from this chapter:

1) The experimental results in this chapter revealed that the epoxy coating may become ineffective to retard corrosion-induced crack expansion if the surface defects of coating exceed 5% in terms of the percentage of defect area to the total surface area of rebar.

2) PP fibers additives in concrete have shown to provide beneficial effects on reducing the corrosion-induced crack width. But PP fibers additives in concrete do not shown the beneficial effects on reducing the corrosion-induced metal loss.

3) Specimens containing PP fibers and epoxy-coated reinforcements with surface defects did not exhibit significant difference in terms of metal loss when compared to specimens with black rebar and PP additives. Thus, PP additives do not lessen the detrimental effect of defect on epoxy-coated rebar.
4) Increasing PP fibers quantity from 4.5 $kg/m^3$ to 6 $kg/m^3$ do not seem to significantly enhance the beneficial effect of PP fibers. Thus, a normal dosage of PP in the range of 4.5 $kg/m^3$ seems appropriate quantity for enhancing crack resistance.

5) Experimental conditions that incorporate the pre-existing cracks and sustained loading have shown to exert effects on the crack width and metal loss spatial distributions along the length of the slab. The specimens containing the pre-existing cracks and subjected to sustained loading have shown maximum crack width and metal loss in the central segments, where the pre-existing cracks is located.
6.1 Introduction

Corrosion of reinforcing steel bars embedded in concrete is a serious reason to the deterioration of reinforced concrete (RC) structures serviced in de-icing salt using or marine region [119]. Generally, corrosion is caused by the destructive attack of chloride ions penetrating by diffusion from the outside corrosive environment. When corrosion begins, the corrosion products are deposited around the corroding steel. Corrosion products need larger volume than the original steel, resulting in tensile stresses applied on the surrounding concrete and surface cracking of the cover concrete at certain stage. The ultimate capacity of a corroding RC structure is therefore reducing by the loss in the cross section area of reinforced bars or the loss in the bond between the corroded steel and the surrounding concrete [6, 8, 50, 52, 55, 58, 63, 65, 84, 85, 90, 119, 124].

As mention in Chapter II, most previous ultimate capacity prediction models of corroded RC structures used either the average gravimetric mass loss or the maximum pit metal loss of corroded reinforced bars [6, 8, 9]. Usually, the corrosive environment along the structures cannot be exact same which means the corrosion rate is changing place to place, especially for the structures contacted with de-icing salts. If the corrosion rate varies along the corroding length, which will cause the variation of the mass loss along
the corroding reinforced bars. Different researchers have different opinions on developing prediction model by using average gravimetric mass loss or using maximum local metal loss. Azad et al. [8] shew that at high desired corrosion level (20%), average gravimetric mass loss overestimates the residual ultimate capacity of corroded beams. Because Azad et al. [8] did not considered the hook design in his experiment specimens, the authors attributed the large reduced ultimate capacity of beams to the loss in the bond between the steel and the concrete. Torres-Acosta et al. [6] also showed that average radius loss calculated by average gravimetric mass loss had a substandard relation with the ultimate capacity of corroded RC beams. Azad et al. [8] also concluded that the average section loss can be used to predict residual capacity of tested specimens with low desired corrosion level. Fasl J. et al. [20] adopted the corroded reinforcing bars taken from real RC slabs with new concrete. Based on their test, the localized section loss of a single bar did not control the slab behavior; instead, slab flexural behavior was governed by the average section loss of all the reinforcing bars. A lot of researchers [10-20] did studies on mechanical properties of corroded reinforced bars. The general conclusion was that the corroded reinforcing steels exhibited less ductility compared with un-corroded ones and the ultimate strain was significant decreases with the corrosion section loss [10, 11].

Because different researchers used different experimental specimens and setup, it is hard to find a widely accepted prediction models from literatures to predict the ultimate capacity loss in this study. Furthermore, previous models did not consider the yield strength reduction of corroded reinforcing bars.
6.2 Objective

This chapter is to develop an accurate prediction methodology to predict the ultimate capacity loss of corroded specimens mentioned in previous chapters. All the tested ultimate capacity loss of corroded specimens including specimens with black reinforced bars and plain concrete, and specimens with epoxy-coated reinforced bars or adding PP fibers are summarized and used to validate the developed prediction methodology. In this chapter, the developed prediction methodology used the net area of corroded reinforced bars with decreased yield strength of corroded reinforced bars to predict the ultimate capacity loss of corroded specimens. The corrosion crack patterns detected from different specimens are assessed and related with the effective cross section of the corroded specimens used in prediction methodology.

6.3 Experiment program

The major variables used in this experimental program were i) sustained loading, 60% of un-corroded specimen’s ultimate capacity; ii) 2 days wetting and 1 day drying cycles; iii) five different corrosion desired levels (1%, 5%, 10%, 20% and 40%). Randomly picked up several un-corroded specimens were used to do collapse test and measure the ultimate capacity. The typical failure mode of un-corroded specimens is shown in Figure 3–3, which shows the typical flexural failure with several flexural cracks. The remaining specimens were divided into different groups for final comparison. The detail description for different specimens groups is listed in Table 6-1. The experimental condition listed in Table 6-1 is the identification of the test specimens used in this chapter. Test specimen designation in Table 6-1 is as follows: the numerical value -“1”, “5”, “10”, “20” and “40”- indicates the desired corrosion level in terms of metal loss as the percentage of original weight; the
following letter “e” indicates the specimens subject to pre-existing cracks condition; the last numerical value -“60”- indicates the sustained loading level in terms of percentage of the ultimate capacity of un-corroded specimens. For example, “1+0” indicates the testing specimens subjected to 1% desired corrosion level and zero sustained loading. While “1e+60” indicates the testing specimens subjected to 1% desired corrosion level and pre-existing cracks with 60% sustained loading level.

6.3.1 Specimen and material details

The detail information of the test specimens has been introduced in previous chapters. The quantities of all casting materials and curing process after casting have been also mentioned in previous chapters. The reinforcement was two #4 deformed bars in the tension surface. There were not compression and shear reinforced bars used in this program. Figure 6–1 shows the reinforced bars’ bending configuration and dimensions. The overlap part shown at the top-right corner (102 mm) of reinforced bars was welded to enhance the hook condition.
<table>
<thead>
<tr>
<th>Materials</th>
<th>Experimental conditions</th>
<th>Specimens number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black reinforced bars and plain</td>
<td>1+0</td>
<td>3</td>
</tr>
<tr>
<td>concrete</td>
<td>1e+60</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>5+0</td>
<td>2</td>
</tr>
<tr>
<td>Black reinforced bars with PP</td>
<td>5e+60</td>
<td>2</td>
</tr>
<tr>
<td>fibers (4.5 kg/m³) concrete</td>
<td>10+0</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>10e+60</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>20+0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>5</td>
</tr>
<tr>
<td>Black reinforced bars with PP</td>
<td>20+0</td>
<td>1</td>
</tr>
<tr>
<td>fibers (6.0 kg/m³) concrete</td>
<td>20e+60</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>2</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars and</td>
<td>20+0</td>
<td>1</td>
</tr>
<tr>
<td>plain concrete</td>
<td>10e+60</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with</td>
<td>20+0</td>
<td>1</td>
</tr>
<tr>
<td>PP fibers (4.5 kg/m³) concrete</td>
<td>20e+60</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with</td>
<td>20+0</td>
<td>1</td>
</tr>
<tr>
<td>PP fibers (6.0 kg/m³) concrete</td>
<td>20e+60</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: Experimental condition means the desired corrosion level, pre-existing cracks and sustained loading condition.
6.3.2 Test setup for accelerated corrosion induction

As mentioned in previous chapters, ACI 318R-95 [108] and Oh and Kang’s [109] formulas were used in this study to guild the pre-cracking produce to prevent the bending damage of experiment samples before corrosion duration. For testing specimens corroded without pre-existing cracks and sustained loading, the experimental setup is shown in Figure 3–6a. The tank used to contain the NaCl solution was fixed in the top surface of the testing specimen. The experimental setup of testing specimens corroded with pre-existing cracks and sustained loading is shown in Figure 3–6b. The sustained loading is applied with hydraulic jack from the bottom of the test specimens, so the top surface of the testing specimen is tensile surface and the bottom surface is compression surface.

Five different desired corrosion levels were used in this experiment - 1%, 5%, 10%, 20% and 40%. The applied current value decided from the Faraday’s law is shown in Table 4-2 and Table 5-4. Because the total accelerated corrosion duration of all sets is same, specimens with higher desired corrosion level have higher external current value.
6.4 Test result and discussion

All the corroded specimens were eventually tested to failure in three-point bending under a universal testing machine. The embedded reinforced bars are designed with hook ending in this study, which can help the corroded reinforced bars to keep enough bonding with surrounding concrete and avoid the bond failure. Based on the bending test in this program, all the corroded specimens were failed with the yield of the tensile reinforced bars.

6.4.1 Corrosion crack and metal loss of steel

One telltale sign of corrosion of reinforcements in concrete slabs/beams is the development of surface cracks. A commonly accepted explanation is that a higher corrosion level induces a larger amount of corrosion products, which in turn causes a larger crack width as a result of volume expansion of the corrosion products. The corrosion crack width was monitored and recorded during the drying time of corrosion duration. Because different desired corrosion levels and experimental conditions, the following observations can be made:

1) For specimens with plain concrete were corroded with high desired level (≥ 10% in this program) and without pre-existing cracks and sustained loading conditions, the longitudinal crack width are almost equal everywhere along the tensile reinforced bar and was also fully penetrate on both of tensile and compression surface, as shown in Figure 6–2a-b. Two side-parts concrete almost separate from the center part of the test specimens.

2) The specimens with plain concrete corroded with pre-existing cracks and sustained loading had the maximum longitudinal corrosion crack width within the middle span
of specimens, while the crack width is smaller within other areas and the crack did not penetrate the full length of the specimens’ surface. The corrosion cracks detected on the compression surface only penetrate short length within middle span of specimens. The typical corrosion crack patterns are shown in Figure 6–2c-d.

3) PP fibers have effect to mitigate the concrete crack width, so the specimens with PP fibers had smaller crack width comparing with that from specimens without PP fibers. The specimens with PP fibers corroded without pre-existing cracks and sustained loading did not have fully crack on the specimens’ surface and the compression surface also did not suffer the cracks as detected from the specimens without PP fibers, as shown in Figure 6–3. The specimens with PP fibers corroded with pre-existing cracks and sustained loading conditions have similar corrosion crack patterns with that of specimens with plain concrete shown in Figure 6–2c-d, but the crack width is much smaller. It is obviously shown in Figure 6–3 that the side concrete did not separate from the center part.

After the corrosion duration, all the corroded specimens were moved to do flexural test and measure the residual ultimate capacity. As introduced in previous, the crack patterns caused the different failure modes of the corroded specimens. Typical failure modes and illustration of effective cross section of corroded specimens with different experimental conditions are show in Figure 6–4. Figure 6–4a indicated that the two side parts of the specimens with high desired corrosion level (≥ 10% in this program) and without pre-existing cracks and sustained loading conditions were separate from the
Figure 6–2 The typical corrosion crack patterns of specimens without PP fibers

(a) Corrosion crack on the tensile surface of specimen without pre-existing cracks and sustained loading
(b) Corrosion crack on the compression surface of specimen without pre-existing cracks and sustained loading

(c) Corrosion crack on the tensile surface of specimen with pre-existing cracks and sustained loading
(d) Corrosion crack on the tensile surface of specimen with pre-existing cracks and sustained loading

Figure 6–3 Representative corrosion crack patterns of specimens with PP fibers

(a) Corrosion crack on the tensile surface of specimen without pre-existing cracks and sustained loading
(b) Corrosion crack on the compression surface of specimen without pre-existing cracks and sustained loading

Figure 6–3 Representative corrosion crack patterns of specimens with PP fibers
center part, which can be detect at the very early stage of the bending test. Figure 6–4b indicated that the cross section of the specimens corroded with pre-existing cracks and sustained loading conditions still kept as a whole even at the end stage of the bending test. The specimens with PP fibers or the specimens corroded with low desired corrosion level (<5% in this program) were failed with the same failure mode shown in Figure 6–4b. The illustration of the effective cross section during the bending test based on the failure modes are shown in Figure 6–4c-d. Based on the different desired corrosion levels and experimental conditions, only the half section (middle shaded area in Figure 6–4) or the full section should be considered in theoretical prediction model.

Following the flexure test, slabs were broken to get corroded steel bars and measure the gravimetric mass loss. Corroded reinforced bars were mechanically cleaned using a motorized wire brush and then dipped in a solution of hydrochloric acid. To correct for any possible loss in the base metal due to the cleaning process, replicate un-corroded control bars were cleaned using the same procedure being used on the corroded steel bars. Steel bars were then cut into equal coupons of 50.8 mm in length as shown in Figure 6–5. Each small steel coupon is corresponding with the segments divided on the surface of specimens shown in Figure 6–2. The percentage gravimetric mass loss of a steel coupon, \( Q_{gi} \), was calculated from the Equation (5.1).

The metal loss of all corroded reinforcing bars are measured and listed in Table 6-2. As shown in Table 6-2, the specimens corroded with pre-existing cracks and sustained loading conditions had larger difference between average metal loss and maximum metal loss, while the specimens corroded without pre-existing cracks and sustained loading conditions had similar average metal loss and maximum metal loss.
c) Illustration of the effective cross section of the testing specimens

Figure 6–4 The typical failure modes and the illustration of the effective cross section of the testing specimens

b) The typical failure mode of specimens with PP fibers

Figure 6–5 The coupons of cleaned corroded reinforced bars
Table 6-2 Metal loss of all corroded specimens

<table>
<thead>
<tr>
<th>Material</th>
<th>Test condition*</th>
<th>Average metal loss</th>
<th>Maximum metal loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black reinforced bars and plain concrete</td>
<td>1+0</td>
<td>1.3%</td>
<td>1.7%</td>
</tr>
<tr>
<td></td>
<td>1e+60</td>
<td>3.0%</td>
<td>4.8%</td>
</tr>
<tr>
<td></td>
<td>5+0</td>
<td>5.3%</td>
<td>5.9%</td>
</tr>
<tr>
<td></td>
<td>5e+60</td>
<td>8.8%</td>
<td>12.0%</td>
</tr>
<tr>
<td></td>
<td>10+0</td>
<td>7.7%</td>
<td>8.8%</td>
</tr>
<tr>
<td></td>
<td>10e+60</td>
<td>12.2%</td>
<td>17.6%</td>
</tr>
<tr>
<td></td>
<td>20+0</td>
<td>14.1%</td>
<td>16.6%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>17.9%</td>
<td>25.8%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars and plain concrete</td>
<td>20+0</td>
<td>15.8%</td>
<td>16.5%</td>
</tr>
<tr>
<td></td>
<td>10e+60</td>
<td>11.8%</td>
<td>22%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>16.8%</td>
<td>24.4%</td>
</tr>
<tr>
<td>Black reinforced bars with PP fibers (4.5 kg/m³) concrete</td>
<td>20+0</td>
<td>14.5%</td>
<td>25.8%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>13.5%</td>
<td>16.6%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>21.6%</td>
<td>36.7%</td>
</tr>
<tr>
<td>Black reinforced bars with PP fibers (6.0 kg/m³) concrete</td>
<td>20e+60</td>
<td>13.1%</td>
<td>25.3%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>21.4%</td>
<td>35.0%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with PP fibers (4.5 kg/m³) concrete</td>
<td>20e+60</td>
<td>14.1%</td>
<td>21.2%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>20%</td>
<td>37.1%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with PP fibers (6.0 kg/m³) concrete</td>
<td>20+0</td>
<td>12.1%</td>
<td>17%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>13.2%</td>
<td>22.9%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>18.7%</td>
<td>35.9%</td>
</tr>
</tbody>
</table>

Note: "*" the identification of the experimental condition is same in the Table 6-1 and introduced in the article.

6.4.2 Ultimate capacity prediction method of corroded specimens

Usually, the formula used to convert the failure load to ultimate moment capacity is based on the statics theory [55] and ACI 318-14 [125]. However, the serviced slab
components of bridges servicing in corrosive environment, the yield strength of corroded reinforced bars keep decreasing with the corrosion time. The ultimate capacity of RC structures keeps deteriorating with the reduction of the yielding strength and the area section of reinforced bars.

Based on the requirement of ACI, the maximum concrete stain limit value is $\varepsilon_{cu} = 0.003$, and the limit strain of steel should be larger than $\varepsilon_t = 0.004$ to make sure the ductile failure mode of the RC structure. The statistic bending moment design formula is as follow:

$$M_n = A_s \cdot f_y \cdot (d - \beta_u \cdot c) = \alpha f'_c b c$$  \hspace{1cm} (6.1)

where this formula is simplified for this program (only one layer tensile rebar and no compression rebar), $A_s$ is total area of the tensile rebar; $f_y$ is the tensile strength of the rebar; $d$ is the effect depth; $\beta_u = \beta_1/2$ is concrete stress block parameters, $\beta_1 = 0.85 - 0.05 \cdot \frac{f'_c - 4000}{1000}$; $c$ is the neutral axis depth $c = \frac{A_s f_y}{0.85 f'_c b \beta_1}$; $b$ is the width of the section; $f'_c$ is the peak concrete compressive strength; $\alpha = 0.85 \beta_1$.

Several basic assumptions made in Equation (6.1) as follows:

1) The equivalent rectangular compressive stress distribution to replace parabolic shape of compress stress distribution.

2) Perfect bond between steel and concrete.

3) The failure mode of beams is yielding of steel in tension before crushing of concrete in compression.

However, the ACI equations always give lower theoretical ultimate capacity of RC structure, which have considered the safety factor. The peak compression stress of concrete and yield strength of reinforcing steel used in this study were measured from the specimens
directly, which are larger than standard value used in ACI code. And a parabolic stress-strain relation as suggested by Desayi and Krishnan [126] used in this study is as follows:

\[ f_c = \frac{2f'_c \varepsilon}{\varepsilon_0 \left(1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^2\right)} \]  

(6.2)

where \( f_c \) is the stress in concrete at a concrete strain of \( \varepsilon \); \( f'_c \) specified compression strength of concrete measured by test or from design code; \( \varepsilon_0 \) concrete strain at the peak compression stress, may be assumed equal to 0.002.

ACI code use bi-linear curve to describe different grades steel, as show in Figure 6–6. The relation between yield strain and yield stress can be expressed as \( \varepsilon_y = f_y/E_s \). Usually the real yield strength of Grade 60 steel rebar used in this industry project is larger than the value of ACI code. Several reinforced bars are picked from the same batch of reinforced bars used in the test specimens. As shown in Figure 6–6, the typical yield strength of reinforcing steel bars used in this study is 522 Mpa.

![Figure 6–6 Stress-strain curves for reinforced bars](image)

Figure 6–6 Stress-strain curves for reinforced bars
The trial and error method combined with the parabolic stress-strain relation of concrete and the real compression strength of the concrete and the yield strength of the reinforcing steel bar were adopted here to calculate the ultimate capacity of un-corroded and corroded specimens. The basic steps are listed as following:

1) Based on the tested compression strength $f'_c$ and Equation (6.2), the crushing concrete strain $\varepsilon_{cu}$ and corresponding $\alpha_u, \beta_u$ can be calculated with following equations:

$$\varepsilon_{cu} = 0.00421 - \left(\frac{f'_c - 1450}{5800}\right) * 0.00138$$  \hspace{1cm} (6.3)

$$\alpha_u = \frac{1}{f'_c} \int_0^1 f_c(y)dy = \frac{\varepsilon_0}{\varepsilon} \left[ \ln \left( \frac{\varepsilon}{\varepsilon_0} \right)^2 + 1 \right]$$ \hspace{1cm} (6.4)

$$\beta_u = 1 - \frac{\int_0^1 y f_c(y)dy}{\int_0^1 f_c(y)dy} = 1 - \frac{2 \left[ 1 - \frac{\varepsilon_0}{\varepsilon} \tan^{-1} \left( \frac{\varepsilon}{\varepsilon_0} \right) \right]}{\ln \left( \frac{\varepsilon}{\varepsilon_0} \right)^2 + 1}$$ \hspace{1cm} (6.5)

2) Based on the strain diagram of the RC structures section and choosing the initial value of neutral axis $c$:

$$\varepsilon_s = \varepsilon_{cu} \frac{d-c}{c}$$ \hspace{1cm} (6.6)

Comparing $\varepsilon_s$ to the yield strain $\varepsilon_y$ to decide the strength of reinforcing steel bar at the ultimate stage. If $\varepsilon_s < \varepsilon_y$, then $f_s = \varepsilon_s * E_s$; else if $\varepsilon_s > \varepsilon_y$, then $f_s = f_y$.

3) The compression force at the concrete:

$$C = \alpha_u f'_c bc$$ \hspace{1cm} (6.7)

where $\alpha_u$ is calculated from Equation (6.4); $b$ is the width of the slab.

4) The tensile force at the steel:

$$T = A_s f_s$$ \hspace{1cm} (6.8)
where $f_s$ is the actual strength based on the actual service steel strain $\varepsilon_s$ and modulus of elasticity of steel $E_s$.

5) Using trial and error method to check the compression force $C$ and tensile force $T$, if $C \neq T$, then revising the neutral axis depth $c$ and repeat step 2) to 5). Until the chosen value of $c$ makes $C = T$ to satisfy the force equilibrium requirement, the ultimate moment value is as follow:

$$M_n = A_s f_s (d - \beta_u * c) \quad (6.9)$$

6.4.2.1 Yield strength reduction of corroded reinforced bars

So far, many researchers investigation into the residual strength and mechanical properties of corroded reinforced bars [13-15, 17, 127, 128]. The tensile test results show that the tensile ductility of corroded reinforced bars decreased with the increasing of corrosion level. All the existing data were collected and using non-linear fitting equation to present the deterioration of yield tensile strength of corroded reinforced bars against with corrosion level (metal loss), as shown in Figure 6–7. Because different manufacturers provide the different yield strengths of steel bar based on quality control, the dimensionless value - decrease percentage was adopted in this study to calculate the reduction of corroded reinforcing steel bars based on original yield strength. The fitting equation is as following:

$$f_{yc} = -0.0053\Delta m^2 + 1.4629\Delta m + 0.3946 \quad (6.10)$$

where $f_{yc}$ is the reduced percentage of yield strength of corroded reinforced bar; $\Delta m$ is the metal loss after corrosion.
6.4.2.2 Prediction methodology

A theoretical prediction methodology was proposed in this study to predict the ultimate capacity loss of corroded specimens and the detail flow chart of procedures is shown in Figure 6–8, which will be validated with testing results in the following section. As introduced in the previous sections, the crack width and crack patterns of specimens in this program are needed to be inspected and considered before predicting the ultimate capacity. The fully cracking on both of tensile and compression surfaces caused the reduction of the effective section used to do theoretical prediction. The decreased yield strength of the corroded reinforced bars and the reduced cross section of reinforced bars were both considered to calculate the ultimate capacity loss of the corroded specimens.

6.4.3 Theoretical Validation

The prediction methodology introduced in previous section was validated with the testing results of this program by considering the reduced yield strength of corroded
reinforced bars, the section loss of corroded reinforced bars and the effective cross section of the specimens induced by different corrosion patterns. The specimens used black reinforced bars and epoxy-coated reinforced bars with surface defects were almost suffered same metal loss if specimens were corroded with same experimental condition. Furthermore, the detected failure modes of specimens with epoxy-coated reinforced bars were same with that of specimens with black reinforced bars under same desired corrosion levels and experimental conditions.

The tested ultimate capacity loss of corroded specimens was presented by percentage of the un-corroded specimens’ ultimate capacity. The prediction results are listed in Table 6-3 and compared with test results. The prediction results based on the average and maximum metal loss are all presented. In the previous section, the corrosion crack patterns should be considered to decide the effective cross-section of corroded specimens used to predict the ultimate capacity loss. Based on comparison several major results can be drawn:

1) For the specimens corroded with low desired level (≤ 5% in this study) under different experimental conditions, the prediction results with shaded in the Table 6-3 calculated with full section of the specimens, as shown in Figure 6–4b, and the average metal loss, are more accurate than that from maximum local metal loss compared with testing results.

2) For the specimens corroded with high desired corrosion level (≥ 10% in this study) and without pre-existing cracks and sustained loading, only considering the middle section, as shown in Figure 6–4a, and average metal loss is more accurate to predict the ultimate capacity loss, while using full section gave smaller ultimate capacity loss.
3) For the specimens corroded with high desired corrosion level (≥ 10% in this study) and with pre-existing cracks and sustained loading, the prediction results with shaded in Table 6-3 calculated by full section area and average metal loss are accurate by comparing with tested results.

Figure 6–8 The flow chart of proposed prediction methodology
<table>
<thead>
<tr>
<th>Material</th>
<th>Test condition</th>
<th>Tested ultimate capacity loss</th>
<th>Full section</th>
<th>Half section</th>
<th>Model error*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Consider the section loss and reduction of the yield strength of corroded reinforced bars</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Full section</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Half section</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Based on average metal loss</td>
<td>Based on maximum metal loss</td>
<td>Based on average metal loss</td>
</tr>
<tr>
<td>Black reinforced bars</td>
<td>1+0</td>
<td>0.8%</td>
<td>0.8%</td>
<td>2%</td>
<td>/</td>
</tr>
<tr>
<td></td>
<td>1e+60</td>
<td>2.8%</td>
<td>3.2%</td>
<td>7%</td>
<td>/</td>
</tr>
<tr>
<td></td>
<td>5+0</td>
<td>10%</td>
<td>10.8%</td>
<td>11.8%</td>
<td>/</td>
</tr>
<tr>
<td></td>
<td>5e+60</td>
<td>17.8%</td>
<td>17.2%</td>
<td>23.7%</td>
<td>/</td>
</tr>
<tr>
<td></td>
<td>10+0</td>
<td>19%</td>
<td>13.8%</td>
<td>15.7%</td>
<td>19.4%</td>
</tr>
<tr>
<td></td>
<td>10e+60</td>
<td>27%</td>
<td>27.5%</td>
<td>36.5%</td>
<td>31.3%</td>
</tr>
<tr>
<td></td>
<td>20+0</td>
<td>40.9%</td>
<td>33.2%</td>
<td>34.3%</td>
<td>38.2%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>32%</td>
<td>34.2%</td>
<td>53.5%</td>
<td>38.6%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars</td>
<td>20+0</td>
<td>39.1%</td>
<td>33.7%</td>
<td>34.3%</td>
<td>37.5%</td>
</tr>
<tr>
<td></td>
<td>10e+60</td>
<td>25%</td>
<td>26.6%</td>
<td>43.4%</td>
<td>30.6%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>35.9%</td>
<td>36.2%</td>
<td>52.1%</td>
<td>39.2%</td>
</tr>
</tbody>
</table>

Note:** Model error is calculated between test results and the prediction results with shade in the table.

Table 6-4 also shown the results of the specimens with PP fibers were corroded with different experimental conditions. Even with high desired corrosion level, the specimens with PP fibers still keep the whole section at the end stage of flexural testing. The model error in the Table 6-4 indicates that using full section and reduced yield strength in theoretical prediction is reasonable.
Table 6-4 Validation results of specimens with PP fibers

<table>
<thead>
<tr>
<th>Materials</th>
<th>Test condition</th>
<th>Tested ultimate capacity loss</th>
<th>Predicted ultimate capacity loss based on average metal loss</th>
<th>Model error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black reinforced bars with PP fibers (4.5 kg/m³)</td>
<td>20+0</td>
<td>21.4%</td>
<td>28.6%</td>
<td>7.2%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>19.6%</td>
<td>25.1%</td>
<td>5.5%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>38.4%</td>
<td>43.4%</td>
<td>5%</td>
</tr>
<tr>
<td>Black reinforced bars with PP fibers (6.0 kg/m³)</td>
<td>20e+60</td>
<td>21.2%</td>
<td>23.4%</td>
<td>2.2%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>40%</td>
<td>43.3%</td>
<td>3.3%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with PP fibers (4.5 kg/m³)</td>
<td>20e+60</td>
<td>19.7%</td>
<td>23.7%</td>
<td>4%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>38.9%</td>
<td>40.9%</td>
<td>2%</td>
</tr>
<tr>
<td>Epoxy-coated reinforced bars with PP fibers (6.0 kg/m³)</td>
<td>20+0</td>
<td>20.5%</td>
<td>25.4%</td>
<td>4.9%</td>
</tr>
<tr>
<td></td>
<td>20e+60</td>
<td>19.6%</td>
<td>23.5%</td>
<td>3.9%</td>
</tr>
<tr>
<td></td>
<td>40e+60</td>
<td>36.3%</td>
<td>38.8%</td>
<td>2.5%</td>
</tr>
</tbody>
</table>

6.5 Conclusion

Based on experimental results that were obtained in this program, the following conclusions were drawn:

1) The corrosion crack patterns induced by different experimental conditions have influence on the effective cross section used to calculate the ultimate capacity loss. The specimens corroded with high desired corrosion level (≥ 10% in this study) and without pre-existing cracks and sustained loading had fully longitudinal crack on both tensile and compression surface, which caused the effective section reduced to half of the original section during prediction.
2) The relation between the reduction of yield tensile strength of corroded reinforced bars and the corrosion level is developed. Quadratic equation was used in this study to fit the relation based on reference data. The prediction methodology was developed in this study, which considered the section loss and yield tensile strength reduction of corroded reinforced bars. The validation based on the experimental testing shown that the average section loss of corroded reinforced bars gave more accurate prediction results rather than assuming the maximum section loss controls the ultimate capacity loss.

3) PP fibers can mitigate the corrosion crack expansion process, which can help the specimens to keep the effective section even with high corrosion level. Adding PP fibers into plain concrete can improve the ductility of corroded specimens to mitigate the ultimate capacity loss, which can be proved by comparing the testing results of specimens with and without PP fibers suffered to same experimental conditions.
CHAPTER VII
PREDICTING THE TIME FOR THROUGH-CRACKING IN CONCRETE COVER
CONSIDERING UNIFORM AND NON-UNIFORM CORROSION CONDITIONS

7.1 Introduction

In reinforced concrete structures, corrosion of steel reinforcements is a worldwide problem. Corrosion can result in loss of cross-section of steel reinforcements, loss bond between the reinforcements and concrete, and cracking and spalling of concrete cover [129]. Cracking of cover concrete caused by corrosion of steel reinforcement can shorten the service life of structural component. Concrete cover cracking has been adopted as a warning sign of reduced serviceability of structure members and the need for taking the required maintenance actions, such as corrosion prevention and crack repair. The ability to predict the cracking time of cover concrete induced by corrosion of steel reinforcements can help engineers select an appropriate time to take the maintenance actions, thereby minimizing or avoiding further deterioration and potential safety hazard.

As introduced in the Chapter 2, extensive experimental studies, theoretical models and finite element simulations have been devoted to predict serviceability limitation of corroded RC structures. However, most theoretical models [22, 27, 29, 130] were based on the elastic theory or the elastic-plastic theory and often neglect the tension softening characteristics (fracture energy) of concrete. Furthermore, it appears that past experimental programs and theoretical models have been developed based on the
assumption of uniform corrosion pattern, i.e. corrosion product around the perimeter of rebar is uniform [2, 21-27, 29, 30, 56, 95, 96, 99, 130]. It has been recently observed that non-uniform corrosion pattern could exist in the real-world built RC structures exposed to field corrosion environment. Non-uniform distribution of expansion pressure may cause adverse effects to the cracking behavior of concrete cover due to the fact that higher pressure is concentrated at the outer part of rebar toward concrete cover.

As mentioned in Chapter 1, there are several limitations of doing experiment and developing theoretical models to simulate the corrosion-induced cracking in RC structures. For example, the experiment is time consume and hard to simulate the different environment conditions. The theoretical models are hard to consider the realistic material properties of concrete. Finite element (FE) method has been used as an effective tool to study the effect of steel reinforcement corrosion in concrete [31-39]. However, it is difficult to promise the simulation results of FEM modeling can exactly match with test results. Different researchers had different explanations for the poor agreement between FEM analysis and test results. Williamson and Clark [40] offered some insights on the poor agreement of FEM in modeling corrosion process of RC structures by point out the importance of considering the fracture energy of the concrete. While Val et. al. [34] used different testing methods of tensile strength of concrete to explain the poor agreement results. Usually, the previous FE models [34-36] can only adopted the stress distribution and path to represent the corrosion crack path of RC structure. The advantage of extend finite element method (XFEM) [100] offers a better simulation approach to evaluate cracking behavior of RC structure due to corrosion with cracking of the elements.
This chapter is to develop a numerical model by combing with XFEM modeling results and theoretical model to predict the cracking time of cover concrete, and to study the implication of non-uniform corrosion pattern on the cracking time in cover concrete. The XFEM model was validated using existing experiment data in the literature for uniform corrosion pattern. From which the XFEM model was used to study the effect of non-uniform corrosion pattern on time to through-crack in concrete cover. A new predictive equation was suggested to compute time to through-crack for uniform and non-uniform corrosion pattern.

7.2 Review of previous predictive models

The thick-walled cylinder model for concrete cover, initially proposed by Tepfers [131] to analyze the splitting bond strength of reinforcing bars in concrete, had been used widely to develop predictive models of concrete cracking due to reinforcement corrosion [2, 22, 23, 30, 36, 132].

7.2.1 Predictive model for critical expansion pressure

Maaddawy [25] assumed the uniform corrosion pattern in the concrete and adopted the elastic theory to predict the critical expansion pressure $P_{cr}$ as follows:

$$P_{cr} = \frac{2C}{D} f_{ct}$$  \hspace{1cm} (7.1)

where $C$ is the cover thickness of the concrete, $D$ is the diameter of the reinforcement, $f_{ct}$ is the tensile strength of concrete.

In 2006, Zhao [133] considered the non-uniform corrosion pattern within cover concrete and elastic-plastic properties of concrete and derived an expression for the critical expansion pressure $P_{cr}$ expression was follows:
\[ P_{cr} = 0.6 \left( 0.5 + \frac{C}{D} \right) f_{ct} \]  

(7.2)

where \( C, D \) and \( f_{ct} \) are defined the same as in Equation (7.1).

In 2011, Zhao [30], by adopting the damage mechanics theory to deal with the cracking along the radial direction of the concrete cover, have divided the cylinder model into two parts: a cracked part and an intact part. The radial pressure \( q_{Re} \) at the interface between the intact and cracked cylinders can be expressed as:

\[ q_{Re} = \frac{E_c \varepsilon_t (b^2 - R_c^2)}{1 + \nu_c} \frac{1}{1 - 2\nu_c + b^2/R_c^2} \]  

(7.3)

where \( E_c \) is the elastic modulus of concrete; \( \varepsilon_t \) is ultimate tensile strain of concrete, which can be calculated \( \varepsilon_t = f_{ct}/E_c \); \( b = R + C \), \( R \) is the radius of the reinforcement, \( C \) is the cover thickness; \( R_c \) is the radial distance from the center of reinforcement to the front point of the crack; \( \nu_c \) is Poisson ratio of concrete.

The critical expansion pressure \( P_{cr} \) can be expressed as follows:

\[ P_{cr} = q_{Re} \left( \frac{R_c}{R} \right)^{2} \frac{2\sin \phi}{1 + \sin \phi} + c \cos \phi \left[ \left( \frac{R_c}{R} \right)^{2} \frac{2\sin \phi}{1 + \sin \phi} - 1 \right] \]  

(7.4)

where \( c \) is the cohesive strength of the concrete; \( \phi \) is the friction angle of concrete. If \( R_c = b \), which means the crack front has reached the surface of cover concrete, then \( P_{cr} \) is the critical expansion pressure to induce the fully through crack of the cover concrete.

7.2.2 Predictive model for time to through-cracking due to rebar corrosion

Faraday’s law can be used to predict the metal loss and decide the accelerated current applied in experimental works [22, 28, 48, 54, 56, 130, 134]. The equation of Faraday’s law is given in Equation (3.3)

Based on the volume equilibrium between expansion of the corrosion rust and the deformation at the steel and concrete interface, following expression can be established:
\[
\frac{M_r}{\rho_r} - \frac{M_{\text{loss}}}{\rho_s} = \frac{\pi}{4} \left\{ \left[ D + 2(\delta_r - \delta_l) \right]^2 - D^2 \right\}
\]  \hspace{1cm} (7.5)

where \(M_r\) is the mass of corrosion rust, \(M_{\text{loss}}\) is the mass of steel consumed to produce \(M_r\); \(\rho_r\) is the density of the corrosion rust; \(\rho_s\) is the density of the original steel; \(\delta_r\) is the thickness of the rust, \(\delta_r = \delta_l + \delta_o + \delta_c\), where \(\delta_o\) is the thickness of the porous zone at the interface between reinforcement and the surround concrete, \(\delta_c\) is the deformation of the concrete under the expansion pressure force; \(\delta_l\) is the thickness of steel lost to form rust.

Based on the Faraday’s law and volume equilibrium equations, Maaddawy [25] developed the predictive model for time needed to cause through crack due to rebar corrosion:

\[
t_1 = \frac{170820(D+2\delta_o)(1+\nu_c+\psi)}{I_{\text{corr}}} \left[ \frac{2Cf_{\text{ct}}}{D} + \frac{2\delta_o E_{\text{ef}}}{(1+\nu_c+\psi)(D+2\delta_o)} \right]
\]  \hspace{1cm} (7.6)

where \(\psi = \frac{(D+\delta_o)^2}{2C(D+D+2\delta_o)}\); \(E_{\text{ef}}\) is effective elastic modulus of concrete that is equal to \([E_c/(1 + \phi_{cr})]\), \(\phi_{cr}\) is the concrete creep coefficient (2.35 used in [25]); \(\delta_o = 0.001 \text{ mm}\).

### 7.3 Critical expansion pressure calculate from extend finite element method

There are many methods can be used to calculate the critical expansion pressure. The extend finite element method is applied in this thesis.

#### 7.3.1 Extend finite element method

Extend finite element method (XFEM) was originally proposed by Belytschko and Black [100] in 1999 to help alleviate shortcomings of the standard finite element in modeling the propagation of various discontinuities in structures. A key advantage of XFEM is that it alleviates the classical FEM in terms of mesh dependence therefore
requires highly refined mesh or frequent re-meshing to capture and discontinuity propagation. The physical means of XFEM can be illustrated in Figure 7–1 [135]. Figure 7–1a) depicts the actual physical process of the splitting of a rectangular piece of material into two parts. As we can see, the top row of Figure 7–1a only shows the vertical stress-free crack appeared in the middle of the material. Then the material is subjected to a relative motion of two parts (middle row in Figure 7–1a), and finally the right part is compressed in the direction parallel to the crack (bottom row). In Figure 7–1b the solid lines illustrate the ‘final’ XFEM approximation. As seen from Figure 7–1b, the XFEM approximation reproduces quite well both the separation and the independent deformation of one part.
The enriched displacement field in XFEM consists of two parts: a continuous and a discontinuous part. The continuous part is the standard displacement field corresponding to the situation without any crack. The discontinuous displacement field is based on local partitions of unity and enables the element to include a discontinuity. In general the enriched displacement approximation takes the form [135]:

Figure 7–1 The illustrated of actual physical process and XFEM cracking
\[ u(x) = \sum_{i \in I} N_i(x) u_i + \sum_{j \in E} N_j(x) \psi(x) e_j \]  

(7.7)

where \( I \) is the set of nodes in the system, \( N_i(x) \) is the shape function at node \( i \) and \( u_i \) are the nodal displacements, \( i = 1,2, \ldots N_{nod} \), \( E \subset \{1,2, \ldots m\} \) is the set of enriched nodes, \( j = 1,2, \ldots m \), \( N_j(x) \) is the nodal shape functions for the enriched nodes, \( e_j \) are additional degrees of freedom and \( \psi(x) \) is enrichment function.

7.3.2 Critical expansion pressure results

In this study, XFEM solution package in ABAQUS 6.13 was adopted to compute the critical expansion pressure to induce the through cover cracking. XFEM considers the tension softening properties of concrete by using fracture energy \( G_f \) of the concrete. In this study, only Mode I crack opening induced by the corrosion pressure, which is caused by stresses normal to the crack face, is considered in the modeling using XFEM.

7.3.2.1 Basic assumptions and limitations

In developing the extend finite element model for simulation of cracking behavior of concrete cover due to rebar corrosion, the following four basic assumptions in this study are: i) the concrete around the steel reinforcement is modeled as a thick-walled cylinder and the wall thickness is equal to the thinnest concrete cover; ii) the steel reinforcement is not modeled in this study; however, the expansion of corrosion rust is modeled by converting it into the internal expansion pressure; iii) the porous zone around the steel reinforcing bar is not modeled; iv) the radial cracks will develop from the cylinder’s inner surface to outside. Additional inherent limitations of XFEM are as follows: i) An enriched element cannot be intersected by more than one crack; ii) A crack is not allowed to turn more than 90° in one increment during an analysis; iii) Adaptive re-meshing is not supported; iv) only quadratic element (2D) can be enriched.
7.3.2.2 Extend finite element model (XFEM) description

In XFEM, the boundary conditions and initial crack locations need to be carefully modeled in order to capture the real behavior of the simulated problem. We adopted the experimental findings from Patnaik et al [106] in establishing our XFEM models. Based on observation of the sections of the experimental specimens, they have noticed that, most of the specimens have three obvious cracks with $120^\circ$ angle apart. The photos of these crack patterns in the cut thin-section are shown in Figure 7–2.

(a) Typical slice of corroded specimen       (b) Crack pattern around the reinforcement

Figure 7–2 Typical corrosion crack patterns

In this study crack initiation and propagation are considered as 2D problem-plain strain formulation in XFEM modeling to simplify the computations. Four-node bilinear quadrilateral finite-elements with reduced integration and hourglass control are used. The total model elements are enriched, thus allowing the crack to go through the elements. Based on the experimentally observed crack patterns [106], three initial cracks locations were embedded in the mesh with three fixed boundary points applied, as show in Figure 7–3. In order to minimize the effect of the initial cracks on the crack propagation process, the length of initial crack is purposely chosen to be less than $\frac{1}{150} C$ ($C$ is the cover thickness
of the concrete). The sweep mesh control in the ABAQUS software was used in establishing the entire mesh. Furthermore, the element length is controlled to be no larger than the initial cracks’ length (2 mm used in this study). The typical mesh result is shown in Figure 7–3.

![Figure 7–3 FE simulation of the cylinder model](image)

7.3.2.3 Model parameters used in XFEM

In order to evaluate the sensitivity of the initial cracks number to the critical expansion pressure, three initial cracks, one initial cracks and without initial crack conditions were all simulated with XFEM in ABAQUS, the final results of critical expansive pressure had small difference (within 5%). Except the basic geometry parameters used to establish XFEM models based on literature. There are several important parameters need to be decided to capture the real behavior of the cracking propagation.
Elastic modules $E_c$ and Poisson’s ratio $\nu_c$ of concrete are used to decide the elastic properties of the concrete material in ABAQUS, which can be decided from literature. The maximum principal stress damage criterion is chosen as the limitation of element cracking. The fracture damage parameters are tensile strength $f_{ct}$ and fracture energy $G_f$. Tensile strength $f_{ct}$ can be decided from literature. Europe design code [136] provides the fracture energy $G_f$ based on the tensile strength $f_{ct}$ of the concrete and the maximum aggregate size.

### 7.3.2.4 Critical expansion pressure with uniform induced expansion

In order to assess the accuracy of XFEM model, we have collected available experimental data from literature [22, 28, 48, 54, 56, 130, 134] for comparison purpose. Table 7-1 presents the key experimental parameters of the selected test specimens in literature. All the collected reference data are based on uniform corrosion-induced expansion condition; therefore, the internal expansion pressure is uniform as well, as shown in Figure 7–4a. The stress distribution in the radial direction is uniform after the internal expansion pressure was applied to the model, as shown in Figure 7–4b. The uniform internal expansion pressure is applied incrementally until critical expansion pressure is reached based on the crack propagation to the surface of concrete cover. Figure 7–4c and d show the crack propagation from initial condition to the through-crack condition. The uniform expansion pressure induced the through-crack achieved at the same time in three initial crack directions. Table 7-2 presents the critical expansion pressure calculated by the predictive model outlined Section 7.2.1 and as well as by the current XFEM model. As can be seen from Table 7-2, the critical expansion pressure $P_{cr}$ calculated by XFEM model is smaller than that calculated by the Maaddawy’s model [25] using elastic theory and larger than that calculated by Zhao’s model [30, 133] using damage and
elastic-plastic theory. The elastic theory is too idealization to represent the cracking behavior of concrete material. The damage theory considered in [30] is more realistic to describe cracking behavior of concrete material; however, the accuracy of developed predictive model in [30] is mainly depended on the key parameters $c$ and $\varphi$, which are hard to decide and there are not widely accepted standard values. The fracture energy and tension softening properties of concrete material have been used widely in many articles and there is code to decide these parameters. XFEM model with reasonable fracture energy and tensile strength values is realistic and accurate to detect the cracking behavior of the concrete material.
Table 7-1 Basic experimental parameters

<table>
<thead>
<tr>
<th>Reference</th>
<th>$E_c$ (Mpa)</th>
<th>$D$ (mm)</th>
<th>$f_{ct}$ (Mpa)</th>
<th>$C$ (mm)</th>
<th>$i_{corr}$ ($\mu$A/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[22]</td>
<td>27000</td>
<td>16</td>
<td>3.3</td>
<td>25</td>
<td>3.75*</td>
</tr>
<tr>
<td></td>
<td>27000</td>
<td>16</td>
<td>3.3</td>
<td>51</td>
<td>2.41*</td>
</tr>
<tr>
<td></td>
<td>27000</td>
<td>16</td>
<td>3.3</td>
<td>70</td>
<td>1.79*</td>
</tr>
<tr>
<td></td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>70</td>
<td>10*</td>
</tr>
<tr>
<td></td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>[48]</td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>30</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>70</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>24400</td>
<td>16</td>
<td>3.7</td>
<td>29.5</td>
<td>100</td>
</tr>
<tr>
<td>[28, 130]</td>
<td>24400</td>
<td>16</td>
<td>3.7</td>
<td>29.5</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>24400</td>
<td>16</td>
<td>3.7</td>
<td>19.5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>20000</td>
<td>16</td>
<td>3.06</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td>[134]</td>
<td>20000</td>
<td>16</td>
<td>3.06</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>29500</td>
<td>16</td>
<td>3.76</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>29500</td>
<td>16</td>
<td>3.76</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>[56]</td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>[54]</td>
<td>28000</td>
<td>16</td>
<td>4.9</td>
<td>33</td>
<td>150</td>
</tr>
</tbody>
</table>

Note: The '*' means the experimental condition is long-term corrosion test ($i_{corr} < 10 \mu$A/cm$^2$) and others are short-term accelerated test ($i_{corr} > 100 \mu$A/cm$^2$).
Table 7-2 Critical expansive pressures comparison between existing models and XFEM results

<table>
<thead>
<tr>
<th>Reference</th>
<th>Maaddawy’s model $P_{cr}$ (Mpa)</th>
<th>Zhao’s model (2006) $P_{cr}$ (Mpa)</th>
<th>Zhao’s model (2011) $P_{cr}$ (Mpa)</th>
<th>XFEM $P_{cr}$ (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[22]</td>
<td>10.3</td>
<td>4.1</td>
<td>5.4</td>
<td>11.7</td>
</tr>
<tr>
<td></td>
<td>21.0</td>
<td>7.3</td>
<td>10.6</td>
<td>16.6</td>
</tr>
<tr>
<td></td>
<td>28.9</td>
<td>9.7</td>
<td>14.2</td>
<td>21.4</td>
</tr>
<tr>
<td></td>
<td>31.1</td>
<td>10.4</td>
<td>14.2</td>
<td>23.1</td>
</tr>
<tr>
<td></td>
<td>8.9</td>
<td>3.72</td>
<td>4.5</td>
<td>9.8</td>
</tr>
<tr>
<td>[48]</td>
<td>13.3</td>
<td>5.1</td>
<td>6.5</td>
<td>13.1</td>
</tr>
<tr>
<td></td>
<td>22.2</td>
<td>7.7</td>
<td>10.4</td>
<td>17.6</td>
</tr>
<tr>
<td></td>
<td>31.1</td>
<td>10.4</td>
<td>14.2</td>
<td>23.1</td>
</tr>
<tr>
<td></td>
<td>13.6</td>
<td>5.2</td>
<td>6.3</td>
<td>12.7</td>
</tr>
<tr>
<td>[28, 130]</td>
<td>13.6</td>
<td>5.2</td>
<td>6.3</td>
<td>12.7</td>
</tr>
<tr>
<td></td>
<td>9.1</td>
<td>3.8</td>
<td>4.3</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>3.8</td>
<td>5.4</td>
<td>10.0</td>
</tr>
<tr>
<td>[134]</td>
<td>19.2</td>
<td>6.7</td>
<td>10.4</td>
<td>16.3</td>
</tr>
<tr>
<td></td>
<td>11.8</td>
<td>4.7</td>
<td>5.4</td>
<td>11.8</td>
</tr>
<tr>
<td></td>
<td>23.5</td>
<td>8.2</td>
<td>10.4</td>
<td>19.3</td>
</tr>
<tr>
<td>[56]</td>
<td>8.9</td>
<td>3.8</td>
<td>4.4</td>
<td>9.8</td>
</tr>
<tr>
<td>[54]</td>
<td>20.2</td>
<td>7.5</td>
<td>7.1</td>
<td>14.6</td>
</tr>
</tbody>
</table>
7.3.2.5 Critical expansion pressure with non-uniform corrosion induced expansion

Non-uniform corrosion expansion occurs in real structures due to the fact that chlorides are often reaching the rebar with non-uniform diffusion path. Figure 7–5 shows the cases of uniform and non-uniform corrosion around a rebar. $D_0$ in Figure 7–5 is the initial diameter of rebar before corrosion and $D_t$ represents the diameter of rebar at the time after losing some steel cross-sections due to corrosion. For the case of non-uniform corrosion shown in Figure 7–5b, it may be reasonably assumed that the corrosion product thickness is linearly decreasing from the top of the rebar to the bottom of the rebar. The value of $\alpha$ is defined as the ratio of the maximum non-uniform corrosion product thickness.
to the uniform corrosion product thickness. It was reported from the experiments of Gonzalez [137] that the value of $\alpha$ can range from 4 to 8 in actual field conditions and 5 to 13 in accelerated corrosion testing environment. Jang et al [35] compared stress induced by the uniform corrosion expansion and the non-uniform corrosion expansion by standard finite element results. They used the values of $\alpha(=1,2,4,8)$ to explore the effects of non-uniform corrosion expansion. Jang [35] indicated that the cracking pressure for non-uniform corrosion $\alpha = 4$ and $\alpha = 8$ is about 40%-60% of that for uniform corrosion case. For non-uniform corrosion pattern $\alpha = 2$, the critical expansive pressures of Jang’s [35] results was slightly larger than that for uniform corrosion case, which is not reasonable.

(a) Uniform corrosion pattern
Figure 7–5 Basic description for uniform and non-uniform corrosion patterns

Due to lack of laboratory experimental data related to cracking under non-uniform corrosion expansion; therefore, the use of XFEM modeling techniques in this paper can be effective approach to investigate the difference between uniform and non-uniform corrosion expansion on cracking behavior in concrete cover. In the numerical simulation, we adopted $\alpha = 2$ to model the non-uniform expansion pattern, as depicted in Figure 7–6a. After the non-uniform inner pressure is applied to the model, as depicted in Figure 7–6b, the radial stress as computed by XFEM was also non-uniform, as shown in Figure 7–6c. Figure 7–6d shows that one crack propagates toward the concrete surface, which occurred was much earlier than other two cracks.
Figure 7–6 Non-uniform corrosion pattern

The XFEM computed critical expansion pressure for $\alpha = 2$ case us summarized in Table 7-3. In order to avoid the duplicate simulation and following analysis purpose, only part experimental references in Table 7-1 were selected to do comparison. It can be seen that the critical expansion pressure difference between uniform corrosion pattern and non-uniform corrosion pattern can be up to 35%. Also shown in Table 7-3 is that the critical
expansion pressure decreases slightly as cover thickness increases, referring to reference [48] data, which is similar to the findings in [25].

### Table 7-3 Cracking pressures of XFEM for uniform and non-uniform corrosion conditions

<table>
<thead>
<tr>
<th>Reference</th>
<th>$E_c$ (Mpa)</th>
<th>$D$ (mm)</th>
<th>$f_{ce}$ (Mpa)</th>
<th>$C$ (mm)</th>
<th>Uniform corrosion condition $P_{cr}$ (Mpa)</th>
<th>Non-uniform corrosion condition $P_{cr}$ (Mpa)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>[48]</td>
<td>22000</td>
<td>16</td>
<td>3.55</td>
<td>20</td>
<td>9.8</td>
<td>6.4</td>
<td>34.6%</td>
</tr>
<tr>
<td>[22]</td>
<td>27000</td>
<td>16</td>
<td>3.3</td>
<td>51</td>
<td>16.6</td>
<td>12.5</td>
<td>25%</td>
</tr>
<tr>
<td>[134]</td>
<td>29500</td>
<td>16</td>
<td>3.76</td>
<td>25</td>
<td>11.9</td>
<td>7.9</td>
<td>34%</td>
</tr>
<tr>
<td>[54]</td>
<td>28000</td>
<td>16</td>
<td>4.9</td>
<td>33</td>
<td>14.6</td>
<td>9.9</td>
<td>31%</td>
</tr>
</tbody>
</table>

#### 7.4 New prediction model

After the reference experimental results and theoretical knowledge are introduced previously, the new prediction model is proposed.

#### 7.4.1 Modification based on existing model

Maaddawy [25] used the concrete creep coefficient in his prediction model, as given in Equation (7.6). The concrete creep is defined as deformation of concrete structure under sustained load. However, in most accelerated corrosion experiments, the duration of test may not be long enough to have significant contribution from creep. Therefore, the first
modification to Equation (7.6) is to use concrete modulus $E_c$ in XFEM to calculate the

critical expansion pressure.

Most researches have considered that the corrosion product can fill crack before
causing volume expansion and subsequent crack propagation. Even in the accelerated tests,
there still a portion of corrosion product may be fill into the crack before the complete
through-crack occurs on concrete cover. We proposed to add time for corrosion rust to
filling crack open space in calculating total time for developing through-crack in concrete
cover. We proposed to use Lu and Liu [28, 130] proposed expression for the time $t_2$ for
corrosion product to fill into crack, as follows:

$$
t_2 = k \frac{C}{D} t_1
$$

(7.8)

where $k$ is a modified coefficient ($k = 0.9$ for long-term test; $k = 0.2$ for short-term test);
$t_1$ is calculated time from Equation (7.6).

The total time of cover through crack to develop, considering the rust filling time,
can be expressed as:

$$
t_{tot} = t_1 + t_2 = \left(1 + \frac{C}{D} k\right) \left[\frac{170820(D+2\delta_o)(1+\nu_c+\psi)}{l_{corr}E_c} \right] \left[P_{cr} + \frac{2\delta_o E_c}{(1+\nu_c+\psi)(D+2\delta_o)}\right]
$$

(7.9)

where all the parameters defined the same as in Equation (7.6).

7.4.2 Model Validation and Discussion

After the theoretical of the model is introduced in previous sections, the accuracy of
the model will be validated in the following section.

7.4.2.1 Uniform corrosion condition

Based on the experimental parameters given in the original reference, XFEM
package in ABAQUS computer program was used to determine the critical expansion
pressure. With input of this computed critical expansion pressure, along with other relevant
parameters in Equation (7.9), we can compute the total time for through-crack occurring. The computed time for each case study is given in Table 7-4. The predicted time calculated by Maaddaway’s model [25] is also listed in Table 7-4. The model error computed as the difference between the predicted time and the observed time in the experiment is also summarized listed in Table 7-4 for both of new model and Maaddaway’s model. Table 7-4 shows the comparison between predicted through-crack time and observed time in the experiment. As we can see, the new model is more accurate to predict the through-crack time than Maaddaway’s model.

7.4.2.2 Non-uniform corrosion condition

Table 7-5 listed the predicted cover cracking time based on the developed model with input of critical expansion pressure of non-uniform corrosion pattern computed from XFEM computed results. As can be seen in Table 7-5, the cover concrete cracking time of non-uniform corrosion condition was up to 26% shorter than that of uniform corrosion condition for the selected test condition in Table 7-5. It is noted that the cover thickness, tensile strength and elastic modulus of concrete also have influence on the time to reach through-crack in concrete cover between uniform and non-uniform corrosion conditions. The difference between uniform and non-uniform conditions also decreases slightly with an increase of cover thickness (see the results based on reference[48]). Because the main focus in this study is the effect of the non-uniform corrosion pattern, the sensitivity analysis of the geometry and material properties was not included here, which has been well discussed in [25].
<table>
<thead>
<tr>
<th>Reference</th>
<th>Observed time ((h \ or \ y))</th>
<th>Predicted time with developed model (t_{\text{tat}} \ (h \ or \ y))</th>
<th>Model error</th>
<th>Predicted time with Maaddawy’s model (t \ (h \ or \ y))</th>
<th>Model error</th>
</tr>
</thead>
<tbody>
<tr>
<td>[22]</td>
<td>0.72*</td>
<td>0.64*</td>
<td>11%</td>
<td>0.25*</td>
<td>65%</td>
</tr>
<tr>
<td></td>
<td>1.84*</td>
<td>1.90*</td>
<td>3%</td>
<td>0.58*</td>
<td>68%</td>
</tr>
<tr>
<td></td>
<td>3.54*</td>
<td>3.86*</td>
<td>9%</td>
<td>0.98*</td>
<td>72%</td>
</tr>
<tr>
<td></td>
<td>0.30*</td>
<td>0.32*</td>
<td>7%</td>
<td>0.1*</td>
<td>67%</td>
</tr>
<tr>
<td></td>
<td>113</td>
<td>112.9</td>
<td>0.1%</td>
<td>84.9</td>
<td>24.9%</td>
</tr>
<tr>
<td>[48]</td>
<td>151</td>
<td>143.6</td>
<td>4.9%</td>
<td>105.5</td>
<td>45.5%</td>
</tr>
<tr>
<td></td>
<td>208</td>
<td>202.9</td>
<td>2.4%</td>
<td>148.1</td>
<td>28.8%</td>
</tr>
<tr>
<td></td>
<td>264</td>
<td>283.1</td>
<td>7.2%</td>
<td>191.5</td>
<td>27.5%</td>
</tr>
<tr>
<td></td>
<td>147.5</td>
<td>140</td>
<td>5.1%</td>
<td>100.3</td>
<td>32%</td>
</tr>
<tr>
<td>[28, 130]</td>
<td>91.1</td>
<td>93.3</td>
<td>2.4%</td>
<td>66.8</td>
<td>26.7%</td>
</tr>
<tr>
<td></td>
<td>112</td>
<td>107</td>
<td>4.2%</td>
<td>80.9</td>
<td>27.8%</td>
</tr>
<tr>
<td></td>
<td>134</td>
<td>131</td>
<td>2%</td>
<td>92</td>
<td>31.3%</td>
</tr>
<tr>
<td>[134]</td>
<td>194.7</td>
<td>205</td>
<td>5.6%</td>
<td>142.2</td>
<td>27%</td>
</tr>
<tr>
<td></td>
<td>116</td>
<td>115</td>
<td>0.6%</td>
<td>82.3</td>
<td>29%</td>
</tr>
<tr>
<td></td>
<td>155.7</td>
<td>176</td>
<td>12.8%</td>
<td>124.2</td>
<td>20.2%</td>
</tr>
<tr>
<td>[56]</td>
<td>96.4</td>
<td>112.9</td>
<td>17%</td>
<td>84.9</td>
<td>11.9%</td>
</tr>
<tr>
<td>[54]</td>
<td>95</td>
<td>96</td>
<td>1.1%</td>
<td>78.9</td>
<td>16.9%</td>
</tr>
</tbody>
</table>

Note: The time in the table with ‘*’ has the unit year and other time with the unit hour.
Table 7-5 Comparison of predicted cracking time between uniform and non-uniform corrosion conditions

<table>
<thead>
<tr>
<th>Reference</th>
<th>Predict time with modified model-uniform corrosion $t_{tal} \ (h \ or \ y)$</th>
<th>Predict time with modified model-non-uniform corrosion $t_{tal} \ (h \ or \ y)$</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>[48]</td>
<td>112.9</td>
<td>88.3</td>
<td>21.8%</td>
</tr>
<tr>
<td></td>
<td>143.6</td>
<td>114.6</td>
<td>20.2%</td>
</tr>
<tr>
<td>[22]</td>
<td>202.9</td>
<td>160.8</td>
<td>20.7%</td>
</tr>
<tr>
<td></td>
<td>283.1</td>
<td>235.6</td>
<td>16.8%</td>
</tr>
<tr>
<td>[134]</td>
<td>1.90*</td>
<td>1.58*</td>
<td>16.8%</td>
</tr>
<tr>
<td>[54]</td>
<td>115</td>
<td>87.3</td>
<td>24.2%</td>
</tr>
<tr>
<td></td>
<td>96</td>
<td>71.0</td>
<td>25.8%</td>
</tr>
</tbody>
</table>

Note: The time in the table with ‘*’ has the unit year and other time with the unit hour.

7.5 Conclusion

In this chapter, crack propagation in cover concrete due to corrosion of reinforcing steel was investigated using XFEM analysis, from which critical expansion pressure was determined when the crack front has reached to the surface of concrete cover. Furthermore, a predictive equation for determining the time from corrosion initiation to through-crack in the cover concrete was developed. The non-uniform and uniform corrosion patterns were compared in the XFEM studies to highlight the importance of considering non-uniform expansion pressure distribution around the perimeter of rebar in computing time to through-crack. The following conclusions can be drawn from this study:

1) The critical expansion pressure values based on XFEM simulation have been compared with the existing theoretical models. The results of XFEM, in which concrete’s tension softening properties (fracture energy) were considered, were shown to be smaller than that computed by the theoretical model using elastic theory, but shown to be larger than
that of numerical models using plastic and damage theories. Thus, it is important to use appropriate modeling techniques in studying cracking behavior of concrete.

2) The developed prediction model for cover concrete through-crack time was validated with the existing experimental data. The prediction model accounts for the time required for corrosion products to fill into the crack in computing the critical total cover cracking time. The comparison between model prediction and experimental data has shown that the differences between predicted and measured through-crack time is less than 10%.

3) The non-uniform corrosion product distribution around the perimeter of a rebar can be easily simulated by XFEM. The critical expansion pressure to cause through-cracking of concrete cover under non-uniform corrosion condition ($\alpha = 2$) was much smaller than that of uniform corrosion condition. Specifically, for the case studied, the critical cracking pressure can be decreased by 35% when computed in a non-uniform corrosion pattern. This means that a local corrosion at the outer face of rebar can cause the failure of concrete cover at relatively low expansion pressure. The predicted cover concrete through-crack time of non-uniform corrosion pattern was less than that of uniform corrosion pattern.
CHAPTER VIII
SUMMARY AND CONCLUSIONS

8.1 Summary of Work Accomplished

An accelerated corrosion experimental program had been developed in this research, which considered different experimental conditions and different additive materials to assess the corrosion damage. This research encompassed both physical experiment program and theoretical prediction models. The physical experiment program adopted the pre-existing cracks and sustained loading conditions and compared with specimens without these experimental conditions. Furthermore, the epoxy-coated reinforcements with surface defects and PP fibers were considered in physical experimental program to assess their corrosion resistance properties. On the theoretical methodology side, the reduced yield strength and net area of corroded reinforcements used to predict the ultimate capacity loss of tested specimens. The XFEM was adapted to calculate the critical inner expansive pressure and used to prediction the cracking of cover concrete, which is the serviceability limitation. Specific contributions obtained from this physical experimental program and theoretical methodologies included the following:

1) The state-of-the-art literature review clearly supported the need for conducting this research, which revealed that there was no accelerated corrosion using slab specimens by considering pre-existing cracks and sustained loading. It was observed that in the
2) past, most of the research used the beam specimens with shear reinforcements. The results and theoretical models gotten from the experimental program could not represent the real behavior of corroding RC structures.

3) Identified the effects of pre-existing cracks and sustained loading conditions on the longitudinal and transverses corrosion crack expansion process, which induced the longitudinal corrosion crack width variating as parabola curve with the maximum value near the center segments.

4) Identified the effect of pre-existing cracks and sustained loading conditions on metal loss of corroded specimens. Developed the empirical relationship between corrosion crack width and metal loss, which can be applied in practical to help the engineers make a judgment of corrosion damage condition.

5) Epoxy coated reinforcements with surface defects did not improve corrosion resistance in terms of metal loss. PP (polypropylene) fiber did not help corrosion resistance in terms of metal loss. Pre-existing cracks and sustained loading experimental conditions had similar effects on crack patterns and metal loss for specimens with epoxy-coated reinforcements/PP fibers as observed results from specimens with black reinforcements and plain concrete.

6) A theoretical prediction methodology had been developed to predict the ultimate capacity loss of corroded specimens used in this program. The reduced yield strength and net area of corroded reinforcements were considered in the developed prediction methodology. Furthermore, the experimental conditions and corrosion levels had influence on the effective of cross section area after accelerated corrosion duration. PP
fibers mitigated the corrosion crack and improve the ductility of corroded specimens, which could mitigate the ultimate capacity loss.

7) Developed the numerical prediction model to predict the serviceability of the RC structures based on the time of corrosion crack penetration. The non-uniform condition can be easily simulated by XFEM and had been compared with uniform condition, which can represent the realistic condition defected in the serviced RC structures.

8.2 Conclusions

The main conclusions of this study are summarized as follows.

1) Based on comparison, the sustained loading has effect on the longitudinal and transverse corrosion crack width development and the wetting-drying cycle periods do not have very significant effect on longitudinal and transverse corrosion cracks.

2) Transverse cracks have different effects on longitudinal corrosion crack expansion rate: transverse cracks can induce severe corrosion damage by providing clear channel for chloride ion at the initial corrosion stage; however, transverse cracks also provide more space for corrosion product to fill and come out from inside to release the hoop tensile stress at later stages of the corrosion test.

3) The pre-existing cracks and sustained loading conditions have effects metal loss, which make the metal loss variating as parabola curve with the maximum value close to the middle span of specimens. And specimens corroded with pre-existing cracks and sustained loading have larger average metal loss than specimens corroded with other experiment conditions.
4) RC slab specimens corroded under different experiment conditions have shown different crack patterns. Type I is longitudinal crack along the tensile side of the specimen. Type II is a combination of transverse crack and longitudinal crack on the tensile surface of the specimen. Type III is longitudinal crack on the compression surface of the specimen. Different crack patterns developed due to combined interaction of the pre-existing cracks and applied sustained loading during corrosion.

5) Based on the comparison, the pre-existing cracks and sustained loading conditions have effect on the relationship of corrosion crack width and metal loss. The empirical relationship between metal loss and crack width developed from specimens corroded with pre-existing cracks and sustained loading conditions for all corrosion levels remain the same value. It was therefore recommend that 1% mental loss corresponds to a maximum crack width 0.03 mm (0.001 in), which can be referred in practical.

6) The experimental results in this chapter revealed that the epoxy coating may become ineffective to retard corrosion-induced crack expansion if the surface defects of coating exceed 5% in terms of the percentage of defect area to the total surface area of rebar.

7) PP fibers additives in concrete have shown to provide beneficial effects on reducing the corrosion-induced crack width. But PP fibers additives in concrete do not shown the beneficial effects on reducing the corrosion-induced metal loss. Increasing PP fibers quantity from 4.5 kg/m³ to 6 kg/m³ do not seem to significantly enhance the beneficial effect of PP fibers. Thus, a normal dosage of PP in the range of 4.5 kg/m³ seems appropriate quantity for enhancing crack resistance.
8) Specimens containing PP fibers and epoxy-coated reinforcements with surface defects did not exhibit significant difference in terms of corrosion induced metal loss when compared to specimens with black rebar and PP additives. Thus, the PP additives do not lessen the detrimental effect of defects on epoxy-coated rebar.

9) The yield tensile strength of reinforcing bars is decreasing with the increasing of corrosion level. Using the average net area of steel and decreased nominal yield strength to predict the residual ultimate capacity of corroded specimens is accurate by comparing with test results in this study. The crack width and crack patterns have effect on the effective cross section of the corroded slab specimens, which should be considered to calculate the ultimate capacity loss of corroded specimens. PP fibers mitigate the corrosion crack (effective cross section) and improve the ductility of corroded specimens, which can mitigate the ultimate capacity loss of corroded specimens.

10) The results of XFEM, in which concrete’s tension softening properties (fracture energy) were considered, were shown to be smaller than that computed by the theoretical model using elastic theory, but shown to be larger than that of numerical models using plastic and damage theories. Thus, it is important to use appropriate modeling techniques in studying cracking behavior of concrete.

11) The developed prediction model for cover concrete through-crack time was validated with the existing experimental data. The prediction model accounts for the time required for corrosion products to fill into the crack in computing the critical total cover cracking time. The comparison between model prediction and experimental data has
shown that the differences between predicted and measured through-crack time is less than 10%.

12) The non-uniform corrosion product distribution around the perimeter of a rebar can be easily simulated by XFEM. The critical expansion pressure to cause through-cracking of concrete cover under non-uniform corrosion condition ($\alpha = 2$) was much smaller than that of uniform corrosion condition. Specifically, for the case studied, the critical cracking pressure can be decreased by 35% when computed in a non-uniform corrosion pattern. This means that a local corrosion at the outer face of rebar can cause the failure of concrete cover at relatively low expansion pressure. The predicted cover concrete through-crack time of non-uniform corrosion pattern was less than that of uniform corrosion pattern.

8.3 Recommendations for Future Research

The major development efforts of this study were concentrated on physical experimental program related with accelerated corrosion damage of RC slab, as well as the theoretical prediction model and finite element simulation. And also, the following experimental conditions and unconsidered additive materials were used in this research: i) pre-existing cracks and sustained loading experimental conditions, ii) epoxy-coated reinforcements with surface defects and different quantities PP fibers additive, iii) the reduced yield strength and net area of corroded reinforcements were used to predicted the ultimate capacity loss of tested specimens; and iv) extent finite element were applied to predict the critical inner expansive pressure. The following recommendations for future research are suggested:
1) Develop large scale experimental program to assess the corrosion damage of RC structures.

2) Monitor the real serviced bridge structures from the beginning to the end of ultimate and serviceability limitation stage and compare with experimental results to proposal more practical guideline for engineers.

3) Consider multiple layers reinforcements and compression reinforcements to simulate two-way slab.

4) Develop 3D finite element in ABAQUS to simulate the experimental specimens and serviced structures.

5) Use different types of coating reinforcements and fibers in specimens to assess their corrosion resistance proprieties under different experimental conditions.
REFERENCES


108. Institute A.C., Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95), 1995, American Concrete Institute: WASHINGTON, D.C.


125. Institute A.C., Standardization I.O.f. Building code requirements for structural concrete (ACI 318-14) and commentary. 2014. American Concrete Institute.


135. Jirásek M., Belytschko T. Computational resolution of strong discontinuities. in Proceedings of Fifth World Congress on Computational Mechanics, WCCM V, Vienna University of Technology, Austria. 2002.
